Start Historical

2.1 GEOGRAPHY AND DEMOGRAPHY

2.1.1 SITE LOCATION AND DESCRIPTION

2.1.1.1 Specification of Location

The Susquehanna SES is located on the west bank of the Susquehanna River in Salem Township, Luzerne County, Pennsylvania with additional recreational and agricultural lands located on the east bank of the rivers in Conyngham and Hollenback Townships.

It is four miles south of Shickshinny and five miles northeast of Berwick. The nearest village is Beach Haven on the southeast edge of the main station site.

The Universal Transverse Mercator Coordinates for the center point between Susquehanna SES Units 1 and 2 reactors are 4,549,300 meters north and 403,800 meters east, Zone 18. These correspond to 41°05'30" north latitude and 76°08'55" west longitude and are also equivalent to the Pennsylvania Coordinate System (PCS) Coordinates 341,175 feet north and 2,442,025 feet east respectively. The PCS is used throughout this report.

The portion of the site in Salem Township is about 1,574 acres, which includes the property on the flood plain and agricultural land to the west of Confers Lane (Township Road T-438). The main station site area within the security fence is approximately 230 acres. An additional 717 acres of land on the east bank has woodlands, farming, reaction, etc.

Topography in the site vicinity ranges from relatively flat flood plains to gently rolling hills. Elevations range from 500 feet to 1,600 feet above mean sea level (msl). In an east-west direction, the site is essentially flat from the river to U.S. Route 11 and is 530 ft above mean sea level. Elevation increases sharply to the west from U.S. Route 11 to the station site, rising from about 530 feet (mean sea level) to about 700 feet (msl) in the station area (see Figure 2.1-21). Continuing to the westerly edge of the site, the land is relatively flat and at about the same elevation as the main station buildings. In a south-north direction, the site rises gradually from about 650 ft (msl) on the south boundary to about 900 ft (msl) on the north.

The main station buildings are located on a terrace above the flood plain approximately 4000 feet west of the Susquehanna River in Salem Township (see Figure 2.1-22). The land around the main station buildings is relatively open with trees on the steeper slopes. It was formerly under cultivation for farm and orchard crops and is slowly reverting back to woodlands.

2.1.1.2 Site Area Map

A map of the site area including major structures and facilities is provided as (Figure 2.1-22). In addition to the site property in Salem Township the Licensee owns 717 acres of recreational land on the east side of the river in Conyngham and Hollenback Townships.

The exclusion area as defined in 10CFR100 (Ref. 2.1-1), is a circle of radius 1800 feet with the center at the common release point. Radiation dose limits at this location are regulated by

10CFR50.67, Accident Source Term. The coordinates of the common release point are N341,175 and E 2,441,970.5. Aside from transit through the exclusion area, there are no activities permitted within this zone other than those related to station operation. There are no residences within the exclusion area.

Roads that traverse the site are:

a) *On the north - Beach Grove Road (T-419) which is approximately 1600 ft. from the center of the exclusion area and approximately 500 ft. from the nearest vital structure. (Salem Township)*

b) *On the west - Confers Lane (T-438) which is approximately 2000 ft. from the center of the exclusion area and approximately 1400 ft. from the nearest vital structure. (Salem Township)*

c) *On the south - Bell Bend (T-456) which is approximately 1800 ft. from the center of the exclusion area and approximately 1600 ft. from the nearest vital structure. (Salem Township)*

d) *On the east - U.S. Route No. 11 which is approximately 2600 ft. from the center of the exclusion area and approximately 2500 ft. from the nearest vital structure. (Salem Township)*

e) *On the floodplain on the east bank – Route 239 is located approximately 7,100 feet from the center of the exclusion area. (Conyngham Township).*

Railways that traverse the site are:

A rail line owned by the Commonwealth of Pennsylvania traverses the flood plain. It is operated by the North Shore Railroad Co. and is located approximately 2,700 feet east of the center of the exclusion area. This line is exclusively used by the Licensee. The section of the line north of the site is not being used at present. An access spur from the main line of the railroad onto the site permits rail service to the station.

The Susquehanna River flows from north to south separating the site from recreational lands on the east side of the river. The river is navigable only by small pleasure and fishing boats because of shallow water and obstructions in the vicinity of the site.

2.1.1.3 Boundaries for Establishing Effluent Release Limits

The exclusion area is the area within a radius of 1,800 feet from the common release point of both reactors, (Figure 2.1-22). The distance from the gaseous effluent release points to the boundary is at least 1,800 feet. The station liquid effluent release point is located at the river approximately 4,000 feet from the common release point. See Section 2.1.2.3 for arrangements for traffic control.

2.1.2 EXCLUSION AREA AUTHORITY AND CONTROL

2.1.2.1 Authority

PP&L owns the entire plant exclusion area (except for Township Route T-419) in fee simple and, therefore, has complete authority to regulate any and all access and activity within that area.

Minimum distance to exclusion area boundary is discussed in Subsection 2.1.1.2.

2.1.2.2 Control of Activities Unrelated to Plant Operation

The only area within the exclusion area in which activities unrelated to plant operation may or will occur is Township Route T-419. See Section 2.1.2.3 for traffic control arrangements.

The location of this area within the exclusion area is shown in Figure 2.1-22. The exclusion area outside the protected area fence will be posted and, with the exception of the township route, will be closed to persons who have not received authorization to enter the property.

PP&L normally will not control passage along Township Route T-419 within the exclusion area although the Emergency Plan provides for execution of control over passage in the event of emergency conditions at the plant.

2.1.2.3 Arrangements for Traffic Control

PP&L has made arrangements with the Salem Township Supervisors and with the Pennsylvania State Police for control of traffic on Township Route T-419 in the event of an emergency. In addition, Pennsylvania, Luzerne County, and Columbia County Emergency Management Agencies have incorporated traffic control in their emergency procedures.

2.1.2.4 Abandonment or Relocation of Roads

Approximately 0.25 mile of Township Road T-419 was relocated approximately 250 feet south to improve the grade and lessen the number of curves. This road was relocated on property owned by PP&L. Access to the plant by a rail spur was also improved through this area.

2.1.3 POPULATION DISTRIBUTION

The population in the vicinity of the Susquehanna SES is sparsely distributed. The steep sloped ridges and the prevailing land use, agriculture, combine to yield a low population density outside of the communities. Table 2.1-33 indicates a decline in total population of counties within 20 miles of the site between the years 1950 and 2000. There was a decrease in population of about 111,000 people or 15.7% (Ref. 2.1-2 and 2.1-3). Between 1990 and 2000 in these counties there was a decrease in population by about 7,800 people or 1.3%. Sullivan County had the greatest percentage increase (7.4%) and Luzerne County had the greatest percentage loss (2.7%) in population (Ref. 2.1-4). The nearest major populated area, the Wilkes-Barre/Scranton corridor, is 15 to 30 miles northeast of the site.

Definitions for urban and rural areas changed in 2000 and, therefore, it is difficult to compare growth trends. For example, in 1990 urban and urbanized areas had population densities as low as 1,000 persons per square mile and in 2000 this definition decreased to 500 persons per

square mile. The changes in urban and rural populations listed in Table 2.1-34 from 1990 to 2000 may only reflect the changes in definition and not major population shifts from urban to rural areas or vice versa (Ref. 2.1-5 and 2.1-6).

Transient populations were considered in calculating the population distribution in the site vicinity. Variations in the transient population occur from 30 to 50 miles away from the site. This subject is discussed in greater detail in Subsection 2.1.3.3.

Population projections are based upon U.S. Bureau of Census projections for the nation (Ref. 2.1-7). These projections are "stepped-down" to the local level by a ratio technique. The U.S. Census projection series for the nation is based upon fertility assumptions. For the projections used in this report, two basic fertility assumptions are made; one is that the completed cohort fertility is 2.7 children per woman, which is characteristic of a growing population, and the other is 2.1 children per woman, which is characteristic of a replacement population growth. In 1990 the 2.1 completed fertility generates birth rates were comparable to national birth rates experienced in 1970-1974 (Ref. 2.1-8). It is reasonable to assume that the 2.1 completed fertility rate applies to the site area. The U.S. Census projections virtually assume a closed population. That is, on the national level migration of persons into or out of the U.S. is considered negligible. However, such a situation does not exist for the Pennsylvania area. For example, between 1980 and 1990 the U.S. population increased by 9%; however, Pennsylvania's population increased by less than 1% (Ref. 2.1-9). These figures indicate that Pennsylvania experienced out-migration. To include migration in the projections, the trends established in the 1960 to 1970 decade were projected out to the years 2000 to 2020 (Ref. 2.1-10 and 2.1-11). The results of these projections are given in Tables 2.1-35 through 2.1-38 and Figures 2.1-23 through 2.1-26.

2.1.3.1 Population Within 10 Miles

The population within 10 miles of the site was sparsely distributed in 2000, and may be characterized as a declining rural population with a few small communities scattered through the area. As shown in Figure 2.1-27 and Table 2.1-39, the bulk of the population was located in the WSW, NNE, NE, SE, N and SSE sectors (Ref. 2.1-4 and 2.1-12). Seasonal population data were included in this table and figure. These sectors contain all or part of several small communities (Table 2.1-40); however, none of these communities exceeded 25,000 people, and none qualifies as a population center (10CFR100) (Ref. 2.1-1).

The rest of the area is agricultural; however, the number of farms between 1969 and 1997 declined by 26% for Luzerne County and 32% for Columbia County (Ref. 2.1-12). This decline combined with a decline in farmland (24% for Luzerne and 22% for Columbia) indicates a decline in agriculture in the vicinity of the site (Ref. 2.1-13).

Population projections for the area from 0 to 10 miles from the site for 2010 and 2020 are given in Tables 2.1-35, 2.1-37 and Figures 2.1-23, 2.1-25. Between 2010 and 2020 there is a projected decline in population of approximately 2,500 people.

2.1.3.2 Population Between 10 and 50 Miles

As shown in Figure 2.1-28 and Table 2.1-41, the major focus of the population between 10 and 50 miles is in the NE and SE sectors. Seasonal population data were included in this table and

figure. The cities contained in these sectors (Scranton, Wilkes-Barre, and Hazleton) form the nucleus of the Scranton/Wilkes-Barre/Hazleton Metropolitan Area. Between 1990 and 2000 the population decreased from 734,175 to 624,776 in this metropolitan area (Ref. 2.1-14).

Population projections for the area from 10 to 50 miles from the site for years 2010 and 2020 are given in Tables 2.1-36, 2.1-38, and Figures 2.1-24, 2.1-26. These projections are based on the 1990 Census since new projections are not available. After a slight increase in projected population from 2000 to 2010 there is a moderate but steady decline in population between 2010 and 2020.

2.1.3.3 Transient Population Between 0 and 50 Miles

The transient population around the site are of three types:

- *a. Seasonal*
- *b. Daily*
- *c. Transportation*

A seasonal population is dependent upon the time of year. Examples of seasonal dependency are tourists at resort areas and migrant workers. Commuters, a daily population, for example, may be present in an area 40 hours out of the 168 hours in a week. Another example of a daily population is the visitors to the mountains or beaches for the day. Finally, a transportation population is associated with some mode of transportation. For example, several thousand vehicles pass a particular location on a highway during a day; however, the persons in these vehicles may be in the site vicinity a few minutes only. Furthermore, many of the vehicles counted may be those of local residents going to and from work, or running errands. The seasonal and daily populations are of interest to the location of a nuclear power station and are discussed in more detail below.

2.1.3.3.1 Seasonal Population Between 0 to 50 Miles

Within a 30-mile radius of this site there are all or part of eleven counties (Table 2.1-42). Table 2.1-42 lists current population (2000), seasonal potential population and seasonal population. Luzerne and Columbia Counties have a seasonal population of 8,586 or less than 2.2% of the seasonal potential population in these two counties (Ref. 2.1-14). Within 10 miles of the station assuming seasonal population is 2.2% of the total population then there are approximately 1179 people who are considered seasonal.

From 30 to 50 miles the seasonal population maintains the same general concentration as it does within 30 miles. Table 2.1-42 shows that Pike, Sullivan, Monroe and Wayne counties have rather high concentrations of seasonal population. These counties are NW, NE, and E of the site. The seasonal population for the area defined by a 50 mile radius from the site was weighted and incorporated in the Population Distribution (Tables 2.1-39 and 2.1-41 and Figures 2.1-27 and 2.1-28) and Projections (Tables 2.1-35, 2.1-36, 2.1-37, 2.1-38, and 2 Figures 2.1-23, 2.1-24, 2.1-25, 2.1-26), Ref. 2.1-15.

Other sources of seasonal populations are daily visitors to attractions such as parks, wildlife refuges and national forests. It is difficult to weigh the population due to these attractions since the length of stay is usually unknown. Furthermore, persons who visit a park and hike or swim for the day are often from within the study area. Thus, instead of there being a net increase of

persons in the study area, there may only be a redistribution of persons. The station recreation area is estimated to have a peak daily attendance of 1,000 persons and a daily average of 300 persons according to recreation personnel. The nearest recreational area of a significant size is Ricketts Glen State Park, located approximately 15 miles north-northwest of the site and Nescopeck State Park about 12 miles east both in Luzerne County. (Ref. 2.1.16). Attendance at these parks was not included because of possible counting of local residents who visit the parks.

2.1.3.3.2 Daily Transient Population

Persons who work at locations which are different than their residences constitute shifts in population during working hours. Especially large employers or urban centers can result in substantial shifts in local population. The Susquehanna site experiences daily shifts in population both into and away from the site area. However, the resident population presents a conservative (high) estimate of the distribution of persons in the vicinity of the station since employment opportunities away from the site area (i.e., further than 5 miles) are greater than those within it. Population shifts into the area occur as workers commute to the 13 industries which occur within 5 miles of the site. These industries employ a total of 1,746 persons some of whom would be expected to reside in the site area (see Table 2.1-43, Ref. 2.1-17). When one weighs the proportion of working hours to total hours in a week (40/160) the residential population increased by only 437 persons.

Daily shifts in population away from the site would be expected to be greater than noted above due to the presence of a number of urban areas located nearby. In Luzerne County, the City of Wilkes-Barre, located 21 miles to the northeast, is a major urban and employment center. Valmont Industrial Park, located near the City of Hazleton 15 miles southeast of the site, is another important employment center. In Columbia County, Berwick Borough, located 5 miles to the southwest, and the Town of Bloomsburg, located approximately 18 miles to the west-southwest, are the major employment and urban centers.

2.1.3.3.3 Low Population Zone

The distance of the Low Population Zone (LPZ), established for the Susquehanna SES in accordance with 10CFR100, is a three-mile radius from the center of exclusion area. Radiation dose limits at LPZ are regulated by 10CFR50.67, Accident Source Term. The estimated population in the LPZ in 2000 was 2,133 (Table 2.1-39). Projected and existing population distribution data for distances up to 50 miles from the plant are on Tables 2.1-36, 2.1-38, and 2.1-41, and Figures 2.1-24, 2.1-26 and 2.1-28.

No schools, hospitals, state or municipal parks are located within the LPZ. The plant recreation area is within the LPZ. Five industrial plants, Leggett and Platt, Castek, PMC Lifestyle, and MP Metals and Tech Packaging are located within the LPZ. These plants employ a total of 254 persons who would contribute to the peak daily transient population. Seasonal population and daily transient populations are discussed in Subsection 2.1.3.3.1.

Some of the facilities and institutions beyond the low population zone within five miles of the station which because of their nature, may require special consideration when evaluating emergency plans include: campgrounds, at state gamelands, public schools, municipal buildings, swimming and boating operations, a miniature golf course, and a number of

industries. Industries within five miles are identified in Table 2.1-43. The State and county emergency management agencies have plans for notification of facilities and institutions within the 10-mile Emergency Planning Zone (EPZ) in the event of an emergency. Also, State and local police departments will direct traffic in the event of an emergency. It should be noted that portions of communities outside this 10-mile EPZ are included in emergency planning evacuation plans.

2.1.3.4 Population Center

The nearest population center as defined in 10CFR100 is the City of Wilkes-Barre, located about 21 miles northeast, which had a 1980 population of 55,551 and a 1990 population of 47,523 and a 2000 population of 43,123 (Ref. 2.1-4). It is part of the Scranton/Wilkes-Barre/Hazleton Metropolitan Area. See Section 2.1.3.2 for additional information.

Subsection 2.1.3.3 discusses transient population.

Tables 2.1-35, 2.1-36, 2.1-37, 2.1-38 and Figures 2.1-23, 2.1-24, 2.1-25, and 2.1-26 show population projections for the population around the Susquehanna site.

Berwick, Pennsylvania is not likely to exceed 25,000 people and become the population center within the next 40 years. Using population figures provided by the Columbia County Planning Commission (Ref. 2.1-18), Berwick's population is projected to decrease by 1,071 persons between 2000 and 2020 (Table 2.1-44). The 2000 projection of 10,395 was within 379 people according to the 2000 census.

2.1.3.5 Population Density

Tables 2.1-45 and 2.1-46 show the comparison of cumulative population for the initial year of operation and final year of operation versus a cumulative population from a uniform population density of 500 people/sq. mile and 1000 people/sq. mile respectively.

2.1.4 REFERENCES

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End Historical

TABLE 2.1-33

PAST POPULATION CHANGES OF COUNTIES WITHIN 20 MILES OF THE SITE*

County	1950	1960	1970	1980	1990	2000	% Change 1990 to 2000
Luzerne	392,241	346,972	341,956	343,079	328,149	319,250	-2.7
Columbia	53,460	53,489	55,114	61,967	63,202	64,151	1.5
Sullivan	6,745	6,251	5,961	6,349	6,104	6,556	7.4
Schuylkill	200,577	173,027	160,089	160,630	152,585	150,336	-1.5
Carbon	57,558	52,889	50,573	53,285	56,846	58,802	3.4
Total	710,581	632,628	613,693	625,310	606,886	599,095	-1.3

Source: Refs. 2.1-2 and 2.1-3 and 2.1-4

* Population includes entire county even area outside 20 mile radius from site.

TABLE 2.1-34

POPULATION BY RESIDENCE FOR COUNTIES WITHIN 20 MILES OF THE SITE
1980 TO 2000

County	1980		1990		2000	
	Urban	Rural	Urban	Rural	Urban	Rural
Luzerne	253,336	89,743	239,215	88,834	254,164	65,086
Columbia	23,576	38,391	23,418	39,784	35,730	28,421
Sullivan	0	6,349	0	6,104	0	6,556
Schuylkill	76,995	83,675	63,560	89,025	95,497	54,839
Carbon	30,665	22,620	29,795	27,051	29,109	29,693

Source: Ref. 2.1-5 and 2.1-6

TABLE 2.1-35

POPULATION DISTRIBUTION
2010
0-10 Miles
Distance (Miles)

Sector	0-1	1-2	2-3	3-4	4-5	5-10	10 Mile Total
N	40	15	48	1,112	2,157	1,200	4,572
NNE	0	75	29	0	0	1,600	1,704
NE	0	0	162	191	369	9,000	9,722
ENE	0	15	37	67	94	1,600	1,813
E	0	56	22	41	72	1,200	1,391
ESE	24	26	63	175	172	2,300	2,760
SE	44	245	147	143	13	3,000	3,592
SSE	40	154	37	89	46	3,700	4,066
S	36	147	29	137	10	1,900	2,259
SSW	4	245	44	83	120	1,000	1,496
SW	4	15	335	89	1,520	2,000	3,963
WSW	4	60	331	111	4,073	12,700	17,279
W	4	30	59	92	182	1,500	1,867
WNW	12	41	44	99	260	700	1,156
NW	16	72	92	0	94	800	1,074
NNW	70	19	15	0	23	1,000	1,127
TOTAL	298	1,215	1,494	2,429	9,205	45,200	59,841
CUMULATIVE TOTAL	298	1,513	3,007	5,436	14,641	59,841	

Source: Refs. 2.1-8 and 2.1-10

Table Rev. 0

TABLE 2.1-36

POPULATION DISTRIBUTION
2010
0-50 Miles
Distance (Miles)

Sector	10 Mile Total	10-20	20-30	30-40	40-50	50 Mile Total
N	4,572	3,700	1,800	7,700	4,800	22,572
NNE	1,704	17,200	17,100	20,100	8,900	65,004
NE	9,722	75,200	101,000	164,800	40,900	391,622
ENE	1,813	30,200	34,800	12,000	9,600	88,413
E	1,391	12,500	1,400	7,900	33,700	56,891
ESE	2,760	11,600	3,100	7,900	32,600	57,960
SE	3,592	37,200	26,500	34,600	211,500	313,392
SSE	4,066	10,700	23,400	11,500	57,400	107,066
S	2,259	9,000	28,300	38,200	31,100	108,859
SSW	1,496	5,200	42,200	12,600	21,500	82,996
SW	3,963	1,700	18,400	23,700	14,700	62,463
WSW	17,279	21,000	23,200	27,700	40,900	130,079
W	1,867	4,400	4,700	20,500	16,300	47,767
WNW	1,156	4,000	2,300	8,900	43,600	59,956
NW	1,074	1,600	1,500	2,000	1,900	8,074
NNW	1,127	1,500	800	3,000	7,500	13,927
TOTAL	59,841	246,700	330,500	403,100	576,900	1,617,041
CUMULATIVE TOTAL	59,841	306,541	637,041	1,040,141	1,617,041	

Source: Refs. 2.1-8 and 2.1-10

Table Rev. 0

TABLE 2.1-37

POPULATION DISTRIBUTION
2020
0-10 Miles
Distance (Miles)

Sector	0-1	1-2	2-3	3-4	4-5	5-10	10 Mile Total
N	41	15	47	1,056	2,010	1,200	4,369
NNE	0	76	29	0	0	1,400	1,505
NE	0	0	160	181	359	7,400	8,100
ENE	0	15	36	63	92	1,600	1,806
E	0	57	22	39	70	1,200	1,388
ESE	24	26	62	166	168	2,400	2,846
SE	45	246	145	136	13	3,100	3,685
SSE	41	155	36	84	44	4,000	4,360
S	37	147	29	130	10	1,800	2,153
SSW	4	246	44	78	117	1,000	1,489
SW	4	15	330	84	1,480	2,000	3,913
WSW	4	60	327	106	3,966	12,100	16,563
W	4	30	58	88	177	1,500	1,857
WNW	12	42	44	94	254	700	1,146
NW	16	72	91	0	92	800	1,071
NNW	20	19	15	0	22	1,000	1,076
TOTAL	252	1,221	1,475	2,305	8,874	43,200	57,327
CUMULATIVE TOTAL	252	1,473	2,948	5,253	14,127	57,327	

Source: Refs. 2.1-8 and 2.1-10

TABLE 2.1-38

POPULATION DISTRIBUTION
2020
0-50 Miles
Distance (Miles)

Sector	10 Mile Total	10-20	20-30	30-40	40-50	50 Mile Total
N	4,369	3,600	2,000	8,200	4,500	22,669
NNE	1,505	18,300	18,800	21,400	9,900	69,905
NE	8,100	65,600	91,200	151,700	37,400	354,000
ENE	1,806	29,200	31,700	13,600	9,800	86,106
E	1,388	14,800	1,500	8,900	35,600	62,188
ESE	2,846	11,000	3,100	8,300	33,700	58,946
SE	3,685	32,500	23,600	34,700	225,600	320,085
SSE	4,360	9,600	21,200	12,000	62,700	109,860
S	2,153	7,900	25,200	38,600	33,700	107,553
SSW	1,489	4,800	37,800	11,800	21,600	77,489
SW	3,913	1,600	17,000	20,700	13,900	57,113
WSW	16,563	21,400	22,800	25,900	43,600	130,263
W	1,857	4,400	4,700	19,200	16,600	46,757
WNW	1,146	4,000	2,100	8,700	42,100	58,046
NW	1,071	1,500	1,400	1,900	1,800	7,671
NNW	1,076	1,500	700	2,700	7,500	13,476
TOTAL	57,327	231,700	304,800	388,300	600,000	1,582,127
CUMULATIVE TOTAL	57,327	289,027	593,827	982,127	1,582,127	

Source: Refs. 2.1-8 and 2.1-10

TABLE 2.1-39

POPULATION DISTRIBUTION
2000
0-10 MILES
DISTANCE (MILES)

Sector	0-1 Miles	1-2 Miles	2-3 Miles	3-4 Miles	4-5 Miles	5-10 Miles	10 Mile Total
N	12	29	49	711	937	1,575	3,313
NNE	13	28	46	176	83	3,460	3,806
NE	17	28	46	64	126	3,347	3,628
ENE	16	28	46	67	133	1,840	2.130
E	14	27	66	109	139	1,552	1,907
ESE	16	35	79	111	142	2,288	2,671
SE	22	34	77	111	144	3.873	4.261
SSE	29	48	61	90	161	2,505	2,894
S	30	70	61	85	122	1,076	1,444
SSW	30	88	69	85	110	1,015	1,397
SW	30	89	120	418	1,043	771	2,471
WSW	30	89	143	403	3,583	12,590	16,838
W	29	54	63	137	167	2.019	2.469
WNW	16	29	48	67	105	822	1,087
NW	13	29	48	67	86	1,077	1,320
NNW	12	29	48	67	91	1.384	1.631
TOTAL	329	734	1.070	2.768	7.172	41.194	53.267
53,267	329	1,063	2,133	4,901	12,073	53,267	

Ref. 2.1-4 and 2.1-12

TABLE 2.1-40

COMMUNITIES WITHIN 10 MILES OF THE SITE, 2000

COMMUNITIES[1]	POPULATION		DIRECTIONAL SECTOR	RADIAL DISTANCE	% CHANGE
	1990	2000			
Shickshinny	1,108	959	N	5 to 10	-13.45
Briar Creek	616	651	WSW	5 to 10	5.68
Berwick	10,976	10,774	WSW	5 to 10	-1.84
Nescopeck	1,651	1,528	SW	5 to 10	-7.45
Conyngham	2,060	1,958	SE	5 to 10	-4.95

Source: Ref. 2.1-3, 2,1-4, and 2.1-12

NOTES: [1]Boroughs

TABLE 2.1-41

POPULATION DISTRIBUTION
2000
0-50 MILES
DISTANCE (MILES)

Sector	0-10 Miles	10-20 Miles	20-30 Miles	30-40 Miles	40-50 Miles	50 Mile Total
N	3,314	4,953	2,440	5,990	8,071	24,768
NNE	3,806	18,285	13,505	18,818	11,110	65,524
NE	3,629	119,400	78,944	147,035	40,712	389,720
ENE	2,130	13,926	5,178	21,330	35,341	77,905
E	1,909	8,346	7,131	35,232	52,694	105,312
ESE	2,671	13,938	17,073	27,333	56,689	117,704
SE	4,260	36,774	27,237	36,858	230,006	335,135
SSE	2,895	7,229	14,821	14,120	62,753	101,818
S	1,444	14,507	43,974	27,178	24,875	111,978
SSW	1,396	4,018	28,353	15,335	16,925	66,027
SW	2,470	3,511	21,747	18,465	15,064	61,257
WSW	16,839	25,498	18,138	34,811	38,435	133,721
W	2,470	5,868	6,089	27,774	23,573	65,774
WNW	1,086	4,040	4,437	16,797	57,231	83,591
NW	1,318	3,510	2,156	3,354	3,888	14,226
NNW	1,630	3,843	2,524	5,352	10,376	23,725
TOTAL	53,267	287,646	293,747	455,782	687,743	1,778,185
CUMULATIVE TOTAL	53,267	340,913	634,660	1,090,442	1,778,185	

Ref. 2.1-4 and 2.1-12

TABLE 2.1-42

SEASONAL POPULATION OF COUNTIES IN STUDY AREA, 2000

Counties Within 30 Miles of Site	Current Population	Seasonal Potential Population	Seasonal Population	% Seasonal
Wyoming	28,080	30,892	2,812	9.1
Sullivan	6,556	12,672	6,116	48.3
Monroe	138,687	176,579	37,892	21.5
Lackawanna	213,295	217,984	4,689	2.2
Carbon	58,802	70,044	11,242	16
Schuylkill	150,336	152,745	2,409	1.6
Luzerne	319,250	325,270	6,020	1.9
Columbia	64,151	66,717	2,566	3.8
Montour	18,236	18,464	228	1.2
Northumberland	94,556	95,175	619	0.7
Lycoming	120,044	124,865	4,821	3.9
SUBTOTAL	**1,211,993**	**1,291,407**	**79,414**	
Counties Within 50 Miles of Site				
Susquehanna	42,238	51,108	8,870	17.4
Bradford	62,761	68,602	5,841	8.5
Wayne	47,722	72,509	24,787	34.2
Pike	46,302	80,804	34,502	42.7
Northampton	267,066	268,478	1,412	0.5
Lehigh	312,090	313,040	950	0.3
Berks	373,638	375,326	1,688	0.4
Lebanon	120,327	121,253	926	0.8
Dauphin	251,798	253,278	1,480	0.6
Snyder	37,546	38,617	1,071	2.8
Union	41,624	43,763	2,139	4.9
SUBTOTAL	**1,603,112**	**1,686,778**	**83,666**	
TOTAL FOR ALL COUNTIES	**2,815,105**	**2,978,185**	**163,080**	

Source: Ref. 2.1-4, 2.1-12 and 2.1-15

TABLE 2.1-43

INDUSTRIES WITHIN 5 MILES OF THE SITE
2000

Industry	Distance (miles) and Direction from the Site		Number of Employees	Products
Riverview Vibrated Block Co.	4.0	WSW	5	Concrete Block and Brick
Crispin Multiplex Mfg. Co. Inc.	4.75	WSW	45	Valve
Tech Packaging	1.5	WSW	130	Packaging Material
RAD Woodwork Co., Inc.	4.5	SW	60	Lumber Mill
Zeiser Vault Co.	4.25	SW	15	Concrete Products
Cooks Wholesale Food, Inc.	5.0	WSW	8	Package Food Products
Leggett & Platt	1.25	NNE	57	Carpet Underlay
Berwick Industries, Inc.	3.9	WSW	1300	Decorative bows and ribbons
Castek, Inc.	1.5	WSW	10	Plastic/Cement
PMC Lifestyle	1.5	SSW	55	Foam Products
MP Metals	2.5	SW	2	Metal Scrap
Dyco	4.5	SW	59	Packaging Machines
Audimation	3.2	S	4	Stereo Amplifiers

Source: Ref. 2.1-17

TABLE 2.1-44

POPULATION PROJECTIONS FOR BERWICK, PENNSYLVANIA

Year	Historical Trend[1]
1970	12,274
1980	11,850
1990	10,976
2000	10,395
2010	9,845
2020	9,324

[1] Calculated by TtNUS based on 1970-1990 population data supplied by the Columbia County Planning Commission (Ref. 2.1-8). Extrapolation based on 1970 to 1990 trend.

TABLE 2.1-45

CUMULATIVE POPULATIONS FOR 1990 AND 2000

Distance (mi)	1990	2000	500#/mile(sq)*
1	124	329	1,570
2	820	1,063	6,280
3	2,243	2,133	14,135
4	4,573	4,901	25,130
5	12,006	12,073	39,265
10	51,528	53,267	157,079
20	341,058	340,913	628,515
30	610,710	634,660	1,413,715
40	1,020,502	1,090,442	2,513,270
50	1,616,658	1,778,185	3,926,990

* This is the population that would occur if 500 persons per square mile were uniformly distributed over the study area.

Source: Refs. 2.1-12, 2.1-14 and 2.1-16

TABLE 2.1-46
CUMULATIVE POPULATIONS FOR 1990, 2000, 2010 AND 2020

Distance (miles)	1990	2000	2010	2020	1000/mi^{2}*
1	124	329	289	252	3,140
2	820	1,063	1,513	1,473	12,560
3	2,243	2,133	3,007	2,948	28,270
4	4,573	4,901	5,436	5,253	50,260
5	12,006	12,073	14,641	14,127	78,530
10	51,528	53,267	59,841	57,327	314,159
20	341,058	340,913	306,541	289,029	1,256,630
30	610,710	634,660	637,041	593,827	2,827,430
40	1,020,502	1,090,442	1,040,141	982,127	5,026,540
50	1,616,658	1,778,185	1,617,041	1,581,127	7,853,980

*This is the population that would occur if 1000 persons per square mile were uniformly distributed over the study area.

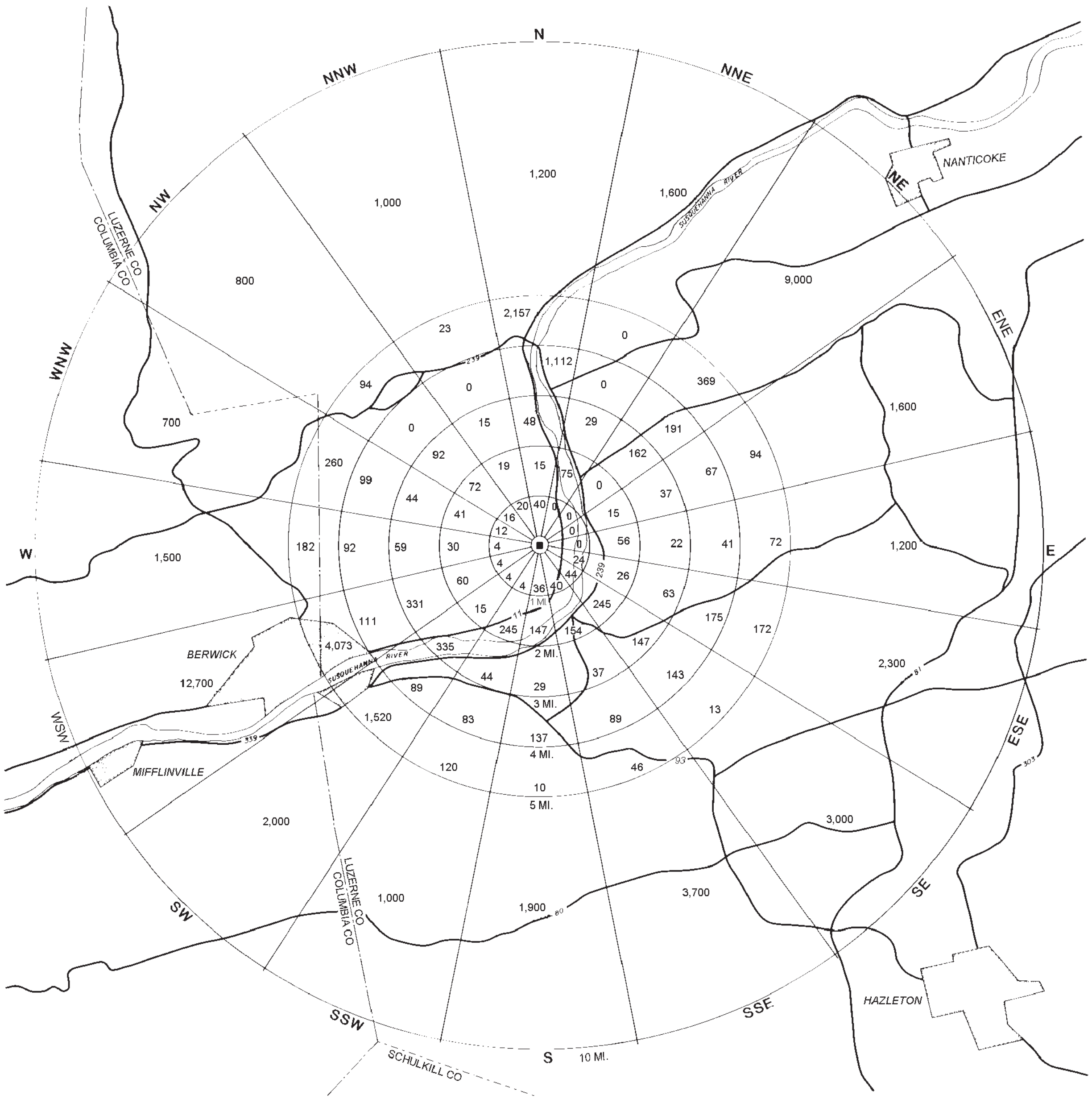
Source: Refs. 2.1-12, and 2.1-14 and 2.1-16

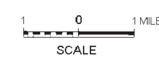
Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
SITE VICINITY MAP
FIGURE 2.1-21

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
SITE AREA MAP
FIGURE 2.1-22





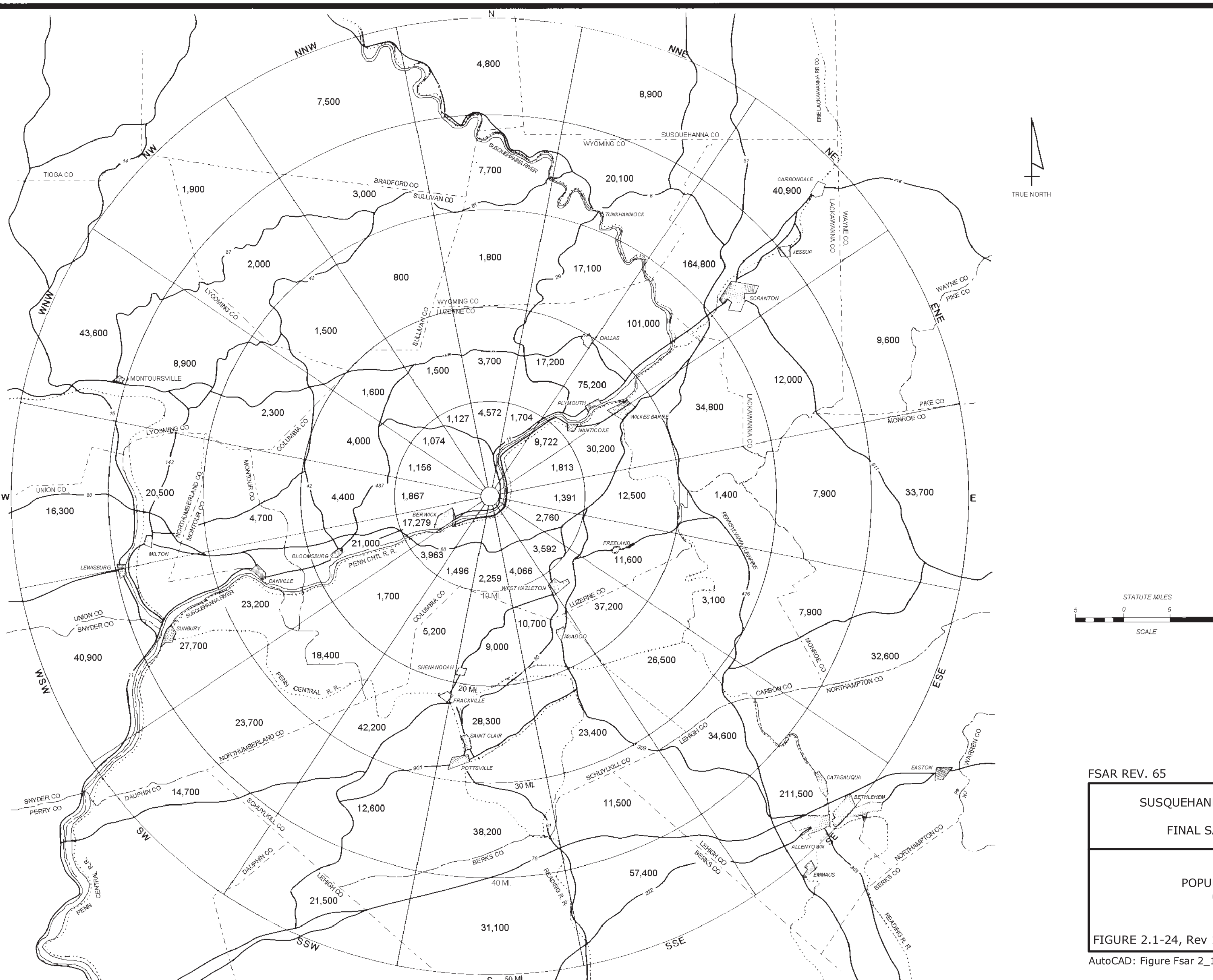
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

POPULATION DISTRIBUTION
0-10 MILES, 2010

FIGURE 2.1-23, Rev 1

AutoCAD: Figure Fsar 2_1_23.dwg



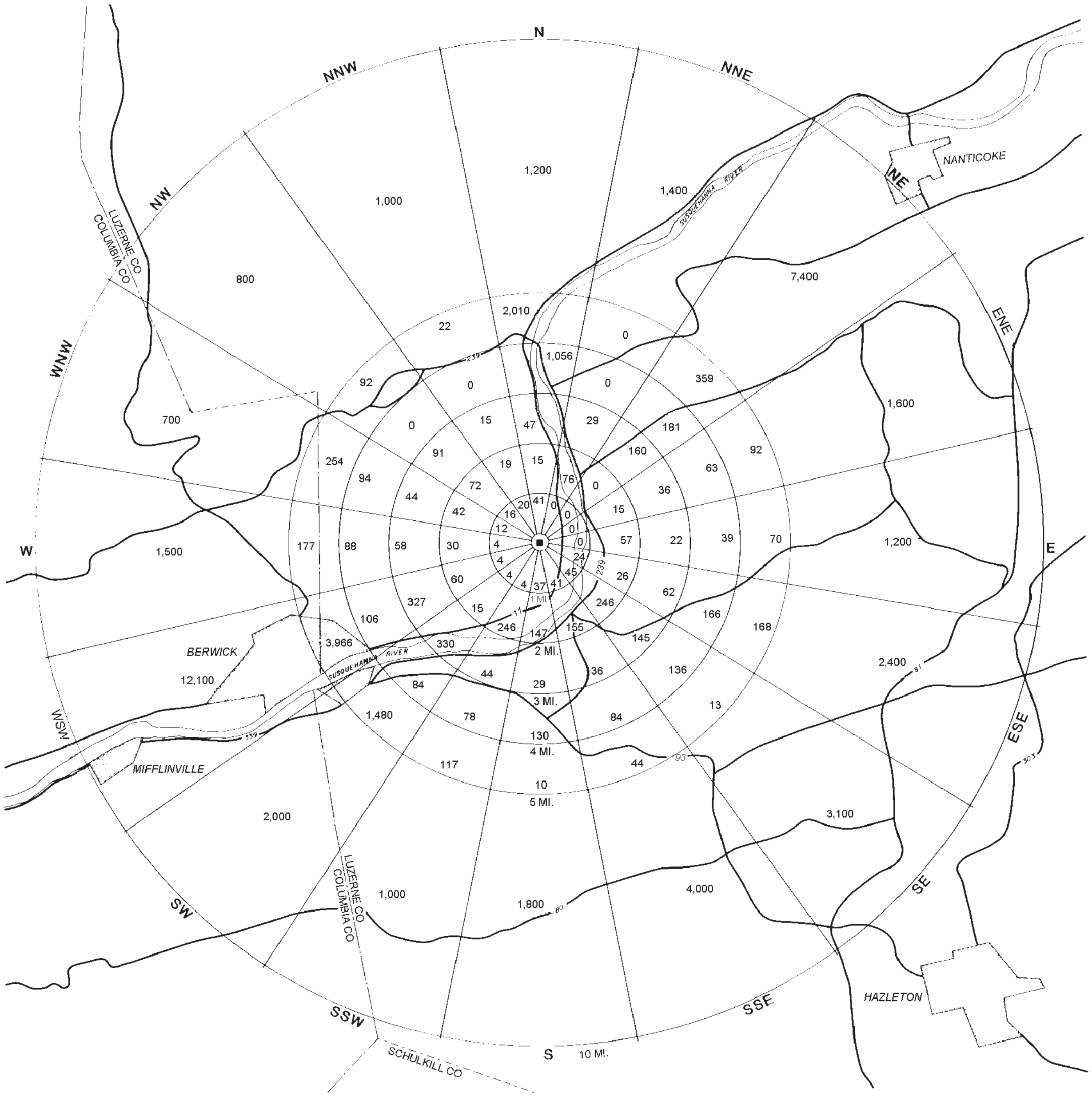
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

POPULATION DISTRIBUTION
0-50 MILES, 2010

FIGURE 2.1-24, Rev 1

AutoCAD: Figure Fsar 2_1_24.dwg



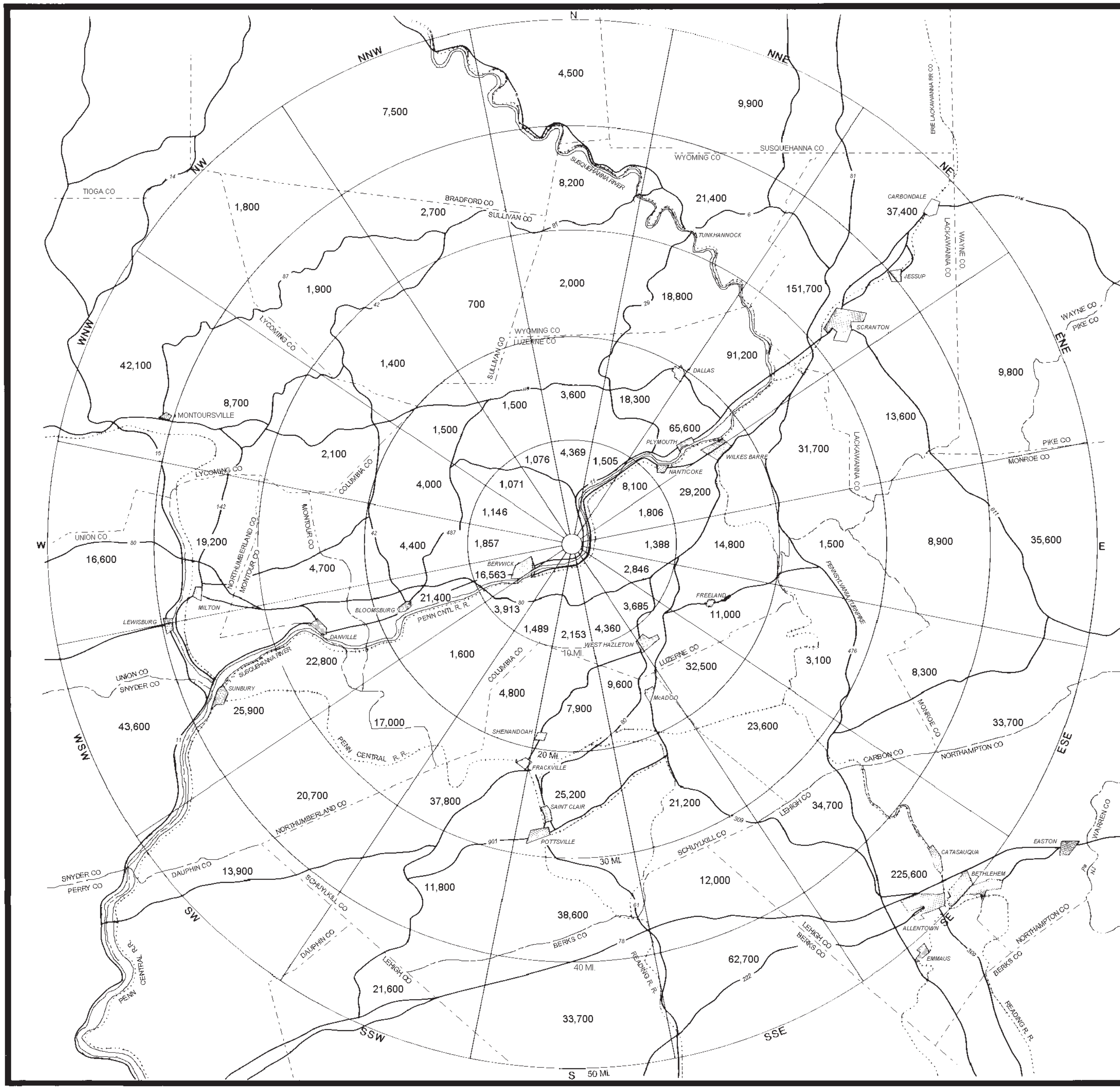
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

POPULATION DISTRIBUTION
0-10 MILES, 2020

FIGURE 2.1-25, Rev 1

AutoCAD: Figure Fsar 2_1_25.dwg



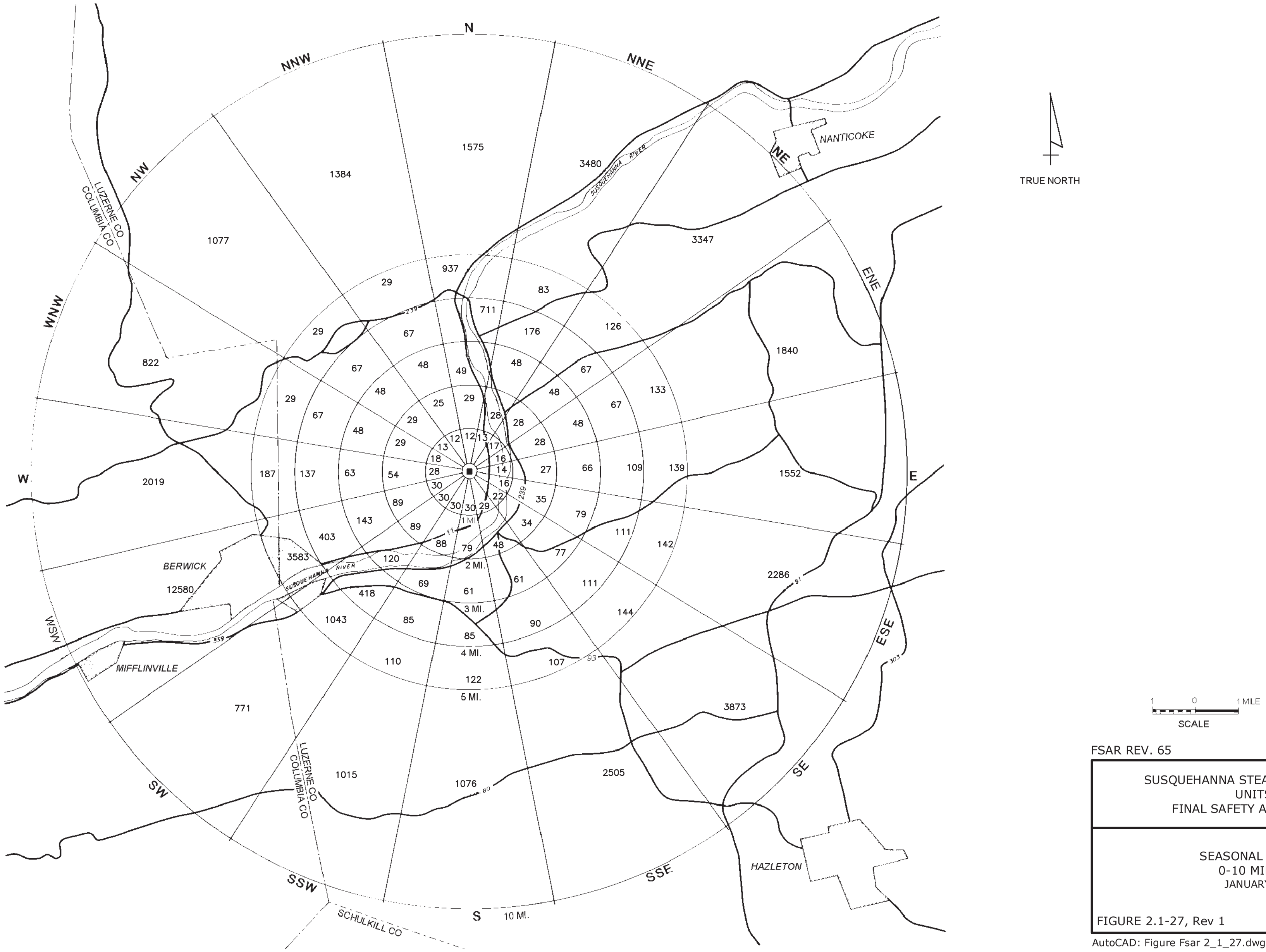
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

POPULATION DISTRIBUTION
0-50 MILES, 2020

FIGURE 2.1-26, Rev 1

AutoCAD: Figure Fsar 2_1_26.dwg



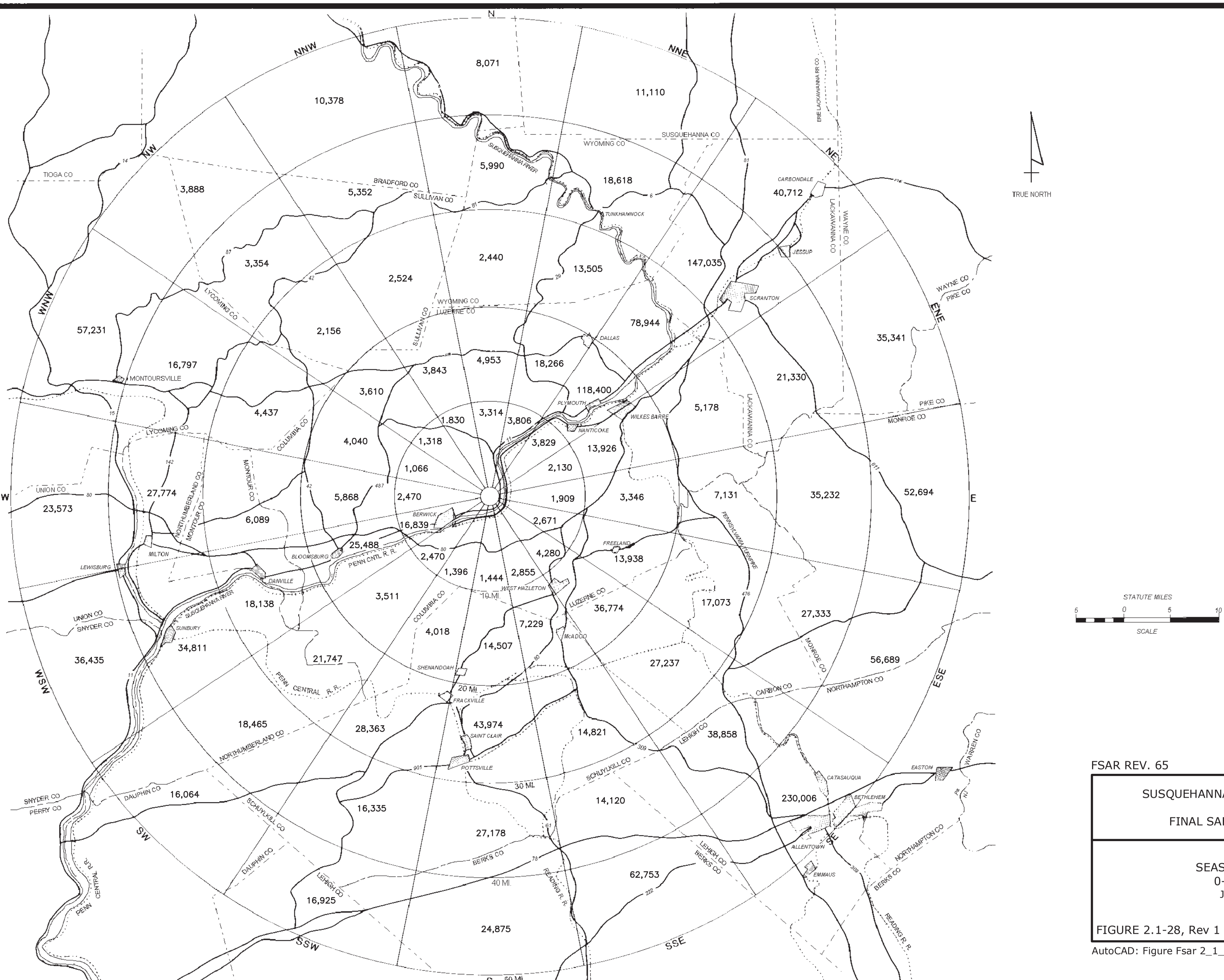
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SEASONAL POPULATION
0-10 MILES, 2000
JANUARY 15, 2003

FIGURE 2.1-27, Rev 1

AutoCAD: Figure Fsar 2_1_27.dwg



FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SEASONAL POPULATION
0-50 MILES, 2000
JANUARY 15, 2003

FIGURE 2.1-28, Rev 1

AutoCAD: Figure Fsar 2_1_28.dwg

Start Historical

2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

2.2.1 LOCATIONS AND ROUTES (See pages 2.2-2 and 2.2-16 for changes)

Security-Related Information
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Security-Related Information
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2.2.2 DESCRIPTIONS

2.2.2.1 Description of Facilities

Thirteen industries located within five miles of the Susquehanna SES are listed on Table 2.1-43 along with their products and number of employees (Ref. 2.1.17). None of these manufacturers stores any explosives or hazardous materials.

The nearest industry to the site is Leggett and Plat *(formerly Dura Bond) which is located 1.25 miles northeast of the site. The company does not use any explosive material. It has a water well field one mile north of the site. The Company employs 74 employees at this location (Ref. 2.2-3).*

There are approximately 25 industries located in Berwick Borough beyond the five mile limit, but within a distance of seven miles from the site. The two largest employers are Wise Potato Chip Company with 800 employees, and Deluxe Homes with 250 employees. The remaining industries are mainly manufacturers of apparel and other finished products. None of these industries located beyond five miles uses or stores any explosive or hazardous materials.

2.2.2.2 Descriptions of Products and Materials

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End Historical

2.2.2.3 Pipelines

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Security-Related Information
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Start Historical

2.2.2.4 Waterways

Navigation, except for recreational boating, is negligible on the Susquehanna River. Therefore, no commercial traffic occurs in the vicinity of the Susquehanna SES. Only recreational boating and sports fishing occur in the vicinity of the Susquehanna SES (Ref. 2.2-4).

2.2.2.5 Airports

The Hazleton Municipal Airport is located 12 miles southeast of the site is the closest airport. The airport serves private and corporate airplanes. The airport has one paved runway with a length of 4,988 feet with an east-west orientation. The flying pattern is a normal left hand pattern. The number of operations over the last 10 years have significantly decreased from earlier estimates because commercial flights were discontinued (Ref. 2.2-5).

The Wilkes-Barre Scranton Airport located 28 miles northeast of the site handles single wheel, dual wheel and dual tandem wheel airplanes. The types of commercial aircraft using the airport are Boeing 727s, BAC-111s and DC-9s. The airport has three asphalt paved runways. The length of the longest runway (Runway 4-22) is 7,500 feet. The restricted maximum weight is 110,000 pounds for single wheel loads, 169,000 pounds for dual wheel loads, and 300,000 pounds for dual tandem wheels. The lengths of the two remaining runways are 4,497 and 3,699 feet. The orientation of the runways are as follows: 4-22-SSW to NNE; 10-28-E to W; 16-34-NNW to SSE. The airport is so distant from the site that approach and holding patterns do not pose a hazard to the plant.

The number of operations conducted at the airport in 1990 was 67,200. Operations in 1990 were approximately 11,000 below the 1973 number and significantly below the 1975 airport forecast for 1995 total operations of 167,400 (Ref. 2.2-6).

Federal Vortac airways passing near the site are:

V-499 3.0 miles	*(West) – Lancaster, Pa./ Binghampton, NY*
V-106 3.5 miles	*(Southeast) – Wilkes-Barre-Scranton, Johnston, PA*
V-164 6.7 miles	*(Southwest) – Allentown/Williamsport, PA*
V-232 9 miles	*(South) – Newark, NJ/Cleveland, OH*
V-188/226 13 miles	*(North) – Wilkes-Barre-Scranton/ Williamsport, PA*

The distances to the site are measured from the map centerline of the route, as given in the standard aeronautical map (New York Sectional).

2.2.2.6 Projections of Industrial Growth

Commonwealth employment forecasts for textile and apparel manufacturing for the Bloomsburg - Berwick and Wilkes-Barre - Hazleton Labor Market Areas indicate negligible industrial expansion. Employment is projected to increase 5.1% in textile manufacturing and 14% in apparel manufacturing from 1970-1990 in the Berwick - Bloomsburg Labor Market Area. For the same period of time in the Wilkes-Barre - Hazleton Labor Market Area, employment is projected to decrease by 40% in textile manufacturing and to increase 6.2% in apparel manufacturing (Ref. 2.2-7).

2.2.3 EVALUATION OF POTENTIAL ACCIDENTS

Potential accidents in nearby transportation, industrial, and military activities are reviewed in this section to evaluate whether their effects at the site might be of serious consequence to nuclear safety, with an annual probability of occurrence exceeding 10^7. For events such as truck accidents where a probability has not been estimated, it is shown that should the event occur no consequence of critical magnitude with respect to nuclear safety would be induced at the plant.

End Historical

2.2.3.1 Determination of Design Basis Events

2.2.3.1.1 Explosions

Security-Related Information
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Security-Related Information
Text Withheld Under 10 CFR 2.390

2.2.3.1.1.1 Hydrogen Water Chemistry Storage Tank Farm

Security-Related Information
Text Withheld Under 10 CFR 2.390

2.2.3.1.2 Flammable Vapors

Security-Related Information
Text Withheld Under 10 CFR 2.390

2.2.3.1.2.1 Twelve-Inch Natural Gas Pipeline

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Security-Related Information
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2.2.3.1.2.2 Twenty-Four Inch Natural Gas Pipeline

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Text Withheld Under 10 CFR 2.390

Security-Related Information
Text Withheld Under 10 CFR 2.390

2.2.3.1.2.3 Thirty-Six and Forty-Two Inch Natural Gas Pipeline

Security-Related Information
Text Withheld Under 10 CFR 2.390

2.2.3.1.2.4 Propane Carriers

Security-Related Information
Text Withheld Under 10 CFR 2.39.

2.2.3.1.2.5 Sixteen Inch Natural Gas Pipeline

*Security-Related Information
Text Withheld Under 10 CFR 2.390*

2.2.3.1.3 Toxic Chemicals

*Security-Related Information
Text Withheld Under 10 CFR 2.390*

Security-Related Information Text Withheld Under 10 CFR 2.390

Start Historical

2.2.3.1.4 Fires

Fires which could result in smoke clouds at the site may arise from brush and forest fires, oil spills from adjacent pipelines, and transportation accidents. A fire from a natural gas pipeline could result in a transient radiant heat flux of very short duration (a few seconds) if the flame front were as close as 1,500 feet. However, the condition is not sustainable and would become limited to about 2,000-3,000 feet from the point of pipeline rupture.

An oil fire from a pipeline rupture at the river, followed by ignition of a pool of floating oil could produce 1.5 Kg/second of particulates for each 1,000 barrels per hour of fuel consumed in open area burning. For pool or choked burning, i.e., sooting conditions, the particulate generation could reach 10 Kg/second. Maximum smoke concentration at the site could reach 250 milligrams/cubic meter. No radiant heat problem at the site would be expected, since firefighting equipment would normally be able to use the road between the site and river bank. However, the on-site fire brigade would respond to any fire at the intake location. The fire hydrant and hose located at the intake would be used to mitigate the effects of the potential radiant heat associated with an oil fire at the river.

The usual failure mode of oil pipelines, the distances to structures containing safety related equipment, and the nature of oil spills on rivers minimize the potential of an oil fire impacting Susquehanna SES. However, as a worst case, it could be assumed that the pipeline will continue to flow for one half hour after the rupture. Since the maximum flow rate in the Sun Pipeline (the closest oil pipeline to the site) is 800 barrels per hour, this would produce a spill of 400 barrels plus the amount remaining in the pipeline up to the points of shutoff in each direction. This distance would be about 3/4 of a mile in the near direction and about 8 miles in the far direction, if it is assumed that pipeline rupture occurs at the shutoff point closest to the site. For 6.625 inside diameter line, this gives a volume of approximately 1970 barrels. When added to the 400 barrels for the amount spilled before shutoff, we have a total worst case spill of 2370 barrels.

The fire would basically burn until the spill was shutoff, 1/2 hour under the worst case conditions. However, it may be that the spill, if it reaches the Susquehanna River, might spread out on the surface of the river and continue to burn until the spill thickness passes below some minimum which will no longer sustain combustion. Under the worst case circumstances, the thickness of the slick by the area over which the spill will spread can be estimated. A well-recognized formula for this spreading is:

$$A = 10^5 \, x \, v^{3/4} \qquad \text{(Ref. 2.2-14)}$$

where A is the spill area in square meters and V is the spill volume in cubic meters. The thickness is then estimated by dividing the volume by the spill area. For the aforementioned

worst case 2370-barrel spill, the formula gives a thickness (at maximum spread) of only 4.2 x 10^{-3} centimeters. At a typical burning rate of one inch per hour, this thickness would be consumed in less than 10 seconds. Therefore, it would appear that a spill from the Sun Pipeline would not be able to burn for much longer than the 1/2 hour maximum flow time until shutoff. This evaluation assumes the oil is spilled on a calm lake. The postulated exposure and the chance for ignition would be minimized by the river flow.

The gas line would not create any smoke problem, but could ignite brush or forest areas. Combustible cover to the northwest of the plant is heavy along Lee Mountain, 3,200 acres at about three miles distance, and over a low ridge north of the plant boundary, 250 acres at one mile. The smoke particulate load estimated from a fire consuming 40 acres per hour (low wind condition, associated with atmospheric stagnation) would be at 210 Kg total particulates per hectare (EPA-42, Factors for Atmospheric Emissions), 160 and 22 milligrams/cubic meter for fires at one and three miles, respectively.

2.2.3.1.5 Collisions with Intake Structure

The Susquehanna River is not used as a navigable waterway for other than small recreational boats, which do not constitute any hazard potential to the intake structure.

2.2.3.1.6 Liquid Spills

Petroleum spills could occur from a pipeline rupture near the Susquehanna River and float on the river surface at the river water makeup line intake. The intake is underwater, so oil would be excluded from entry into the intake line. The most severe condition would occur at the design low water condition with water surface at 483.5 feet MSL. The water intake would still be submerged one foot. Intake flow velocity would be between 0.5 and 0.7 foot/second, remaining below the value of about 1.0 foot/second, at which the surface layer in flow stagnation against the debris lip might begin to be drawn down into the intake. In Subsection 2.4.11, a more severe condition was considered; namely, the low flow level with strong wind blowing away from the intake. Under this latter hypothesis, however, floating oil would also be blown away from the intake.

2.2.3.1.7 Subsurface Gas and Waste Storage

The unconsolidated Quaternary deposits in the vicinity of the Susquehanna SES are unsuitable for storage or disposal. While storage in unconsolidated strata is feasible, the thickness and extent of the Quaternary strata in the site area is insufficient. For example, with regard to aquifer storage of natural gas, the minimum depth of overburden necessary to maintain adequate deliverability at the well head is about 500 ft., while depths in excess of 1500 ft. are desirable for an efficient operation.

As discussed in Subsection 2.4.13, none of the bedrock formations in the site vicinity have a high primary transmissivity. Both the primary porosity and permeability of these well consolidated rocks are generally low. Ground water utilization is dependent upon secondary permeability developed through tectonic fracturing and jointing or solution processes. Thus, while the anticlinal structure in the site vicinity may provide geometry suitable for aquifer storage or disposal, no suitable reservoir strata are known to be present.

Deep well injection into fracture porosity zones in impermeable rock might be considered as a potentially feasible method of waste disposal in the site vicinity. However, based on existing literature and considering current technology, this method of disposal is the least desirable. Reservoir strata with some degree of primary permeability are preferred (Ref. 2.2-15).

It is believed that the Precambrian basement, at depths in excess of 30,000 feet in the vicinity of the Susquehanna SES, does not contain the Fold and Thrust Belt Fracture System (Subsection 2.5.1.1.3). The nature and extent of any fracturing in these rocks is known. Recent advances in drilling technology suggest that the technical capability to construct a disposal well at depths in excess of 30,000 feet may be available in the near future. In the U.S. there has been at least one instance of disposal of chemical waste into Precambrian age crystalline rock (Ref. 2.2-16). However, rocks of this type with transmissibilities dependent solely on fracture porosity are not generally considered to be suitable storage or disposal reservoirs (Refs. 2.2-17, 2.2-18 and 2.2 19).

A discussion of the potential hazard resulting from a subsurface storage facility would be dependent upon the type of facility and the type of material being stored. In view of the low potential for the development of such a facility in the near vicinity of the Susquehanna SES, a discussion of potential hazard is unwarranted.

2.2.3.2 Effects of Design Basis Events

No offsite accidental conditions were identified as constituting a design basis event. The plant is sufficiently removed from local transportation to avoid critical problems with explosions, toxic gases, flammable vapors, and fire. The water intake is designed to exclude floating oil. There are no identifiable nearby industrial or military activities which constitute design basis hazards.

Potential hazard associated with onsite storage and usage of industrial gases has been considered in the control room ventilation design.

2.2.4 REFERENCES

2.2-1 *Telephone conversation with Mr. Ken Bradshaw, Penn Dot, District 4, September 4, 1991.*

2.2-2 *Telephone conversation with Mr. Vince Herman, Manager of Traffic and Expediting, Pennsylvania Power & Light Company, October 11, 1991.*

2.2-3 Leggett and Platt, Personal communication with T. V. Jacobsen, Ecology III on November 14, 2002

2.2-4 *Susquehanna River Basin Study, Appendix C, p. G XI-3.*

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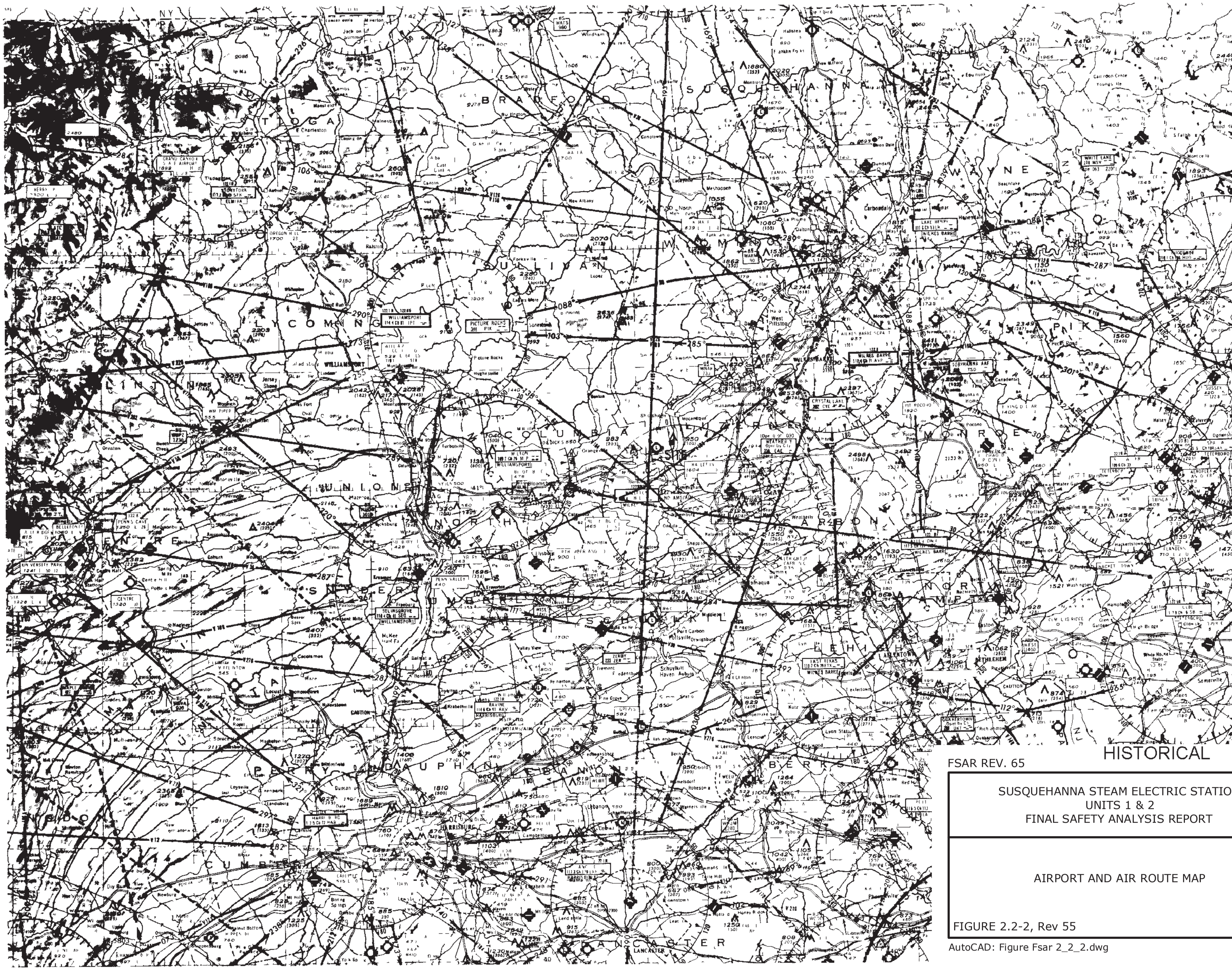
End Historical

Table 2.2-1
PIPELINES WITHIN FIVE MILES OF THE SITE

Security-Related Information
Table Withheld Under 10 CFR 2.390

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
MAJOR TRANSPORTATION ROUTES AND PIPELINES FIGURE 2.2-1



HISTORICAL

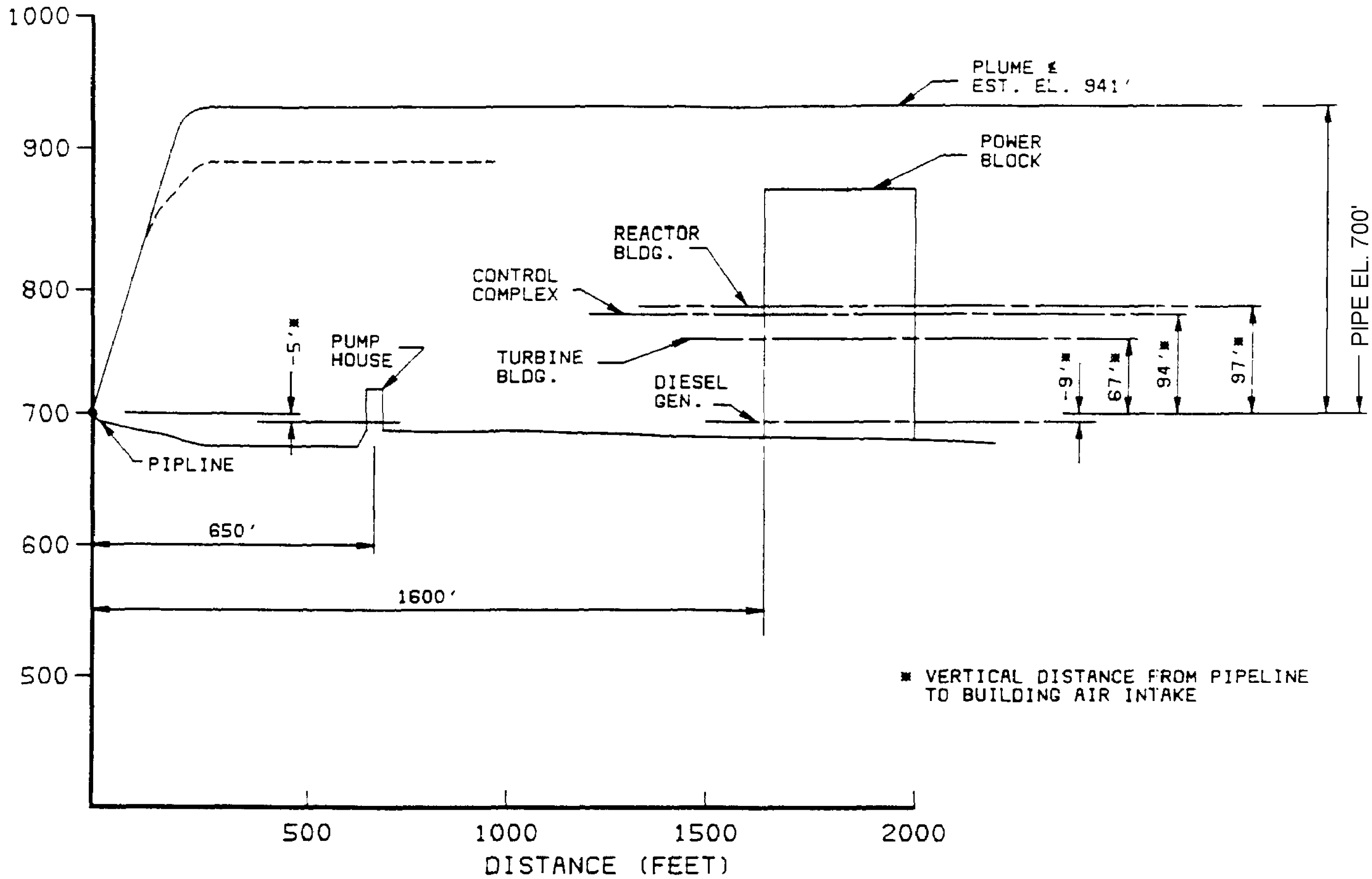
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

AIRPORT AND AIR ROUTE MAP

FIGURE 2.2-2, Rev 55

AutoCAD: Figure Fsar 2_2_2.dwg



HISTORICAL

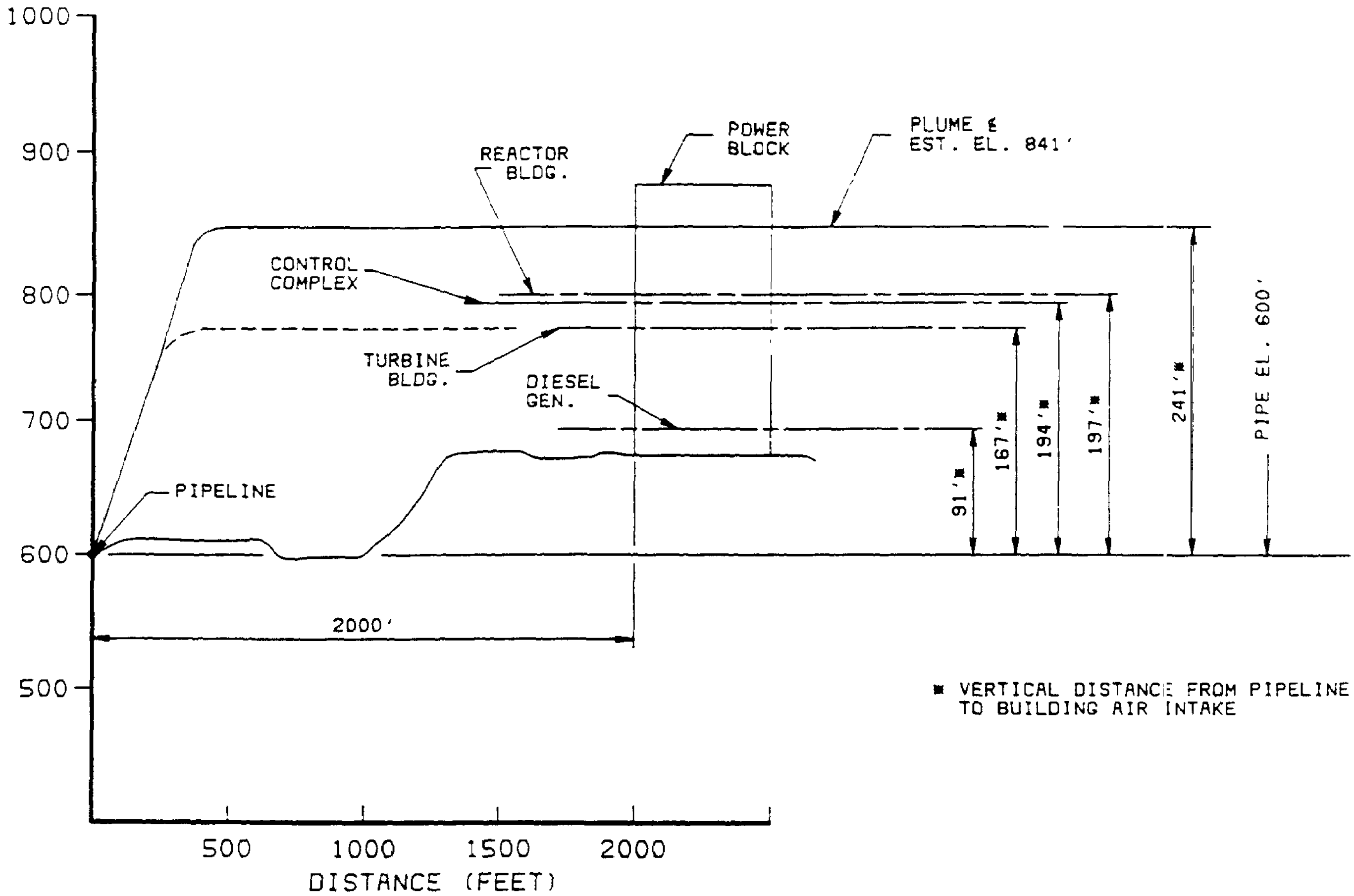
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

PIPELINE BREAK AT EL. 700'
(NEAREST PUMPHOUSE)
STEADY STATE BREAK FLOW

FIGURE 2.2-3, Rev 55

AutoCAD: Figure Fsar 2_2_3.dwg



HISTORICAL

FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
PIPELINE BREAK AT ELEVATION 600' STEADY STATE BREAK FLOW FIGURE 2.2-4, Rev 55

AutoCAD: Figure Fsar 2_2_4.dwg

2.3 METEOROLOGY

2.3.1 REGIONAL CLIMATOLOGY

2.3.1.1 General Climate

The climate of east central Pennsylvania is on the border of Koeppens' "snow forest" and temperate rainy climate (Ref. 2.3-1). There is considerable snow during the winter and relatively hot humid summers with precipitation distributed evenly throughout the year.

This region is repeatedly affected by interactions between warm, moist maritime tropical air masses and cool, dry continental polar air masses. The polar air masses are the dominant influence in the winter while tropical air masses predominate in the summer. Maritime polar air masses are also common in a highly modified form from the Pacific or directly from the North Atlantic. The North Atlantic air masses which are cool and humid are usually associated with approaching warm fronts and "back door" cold fronts.

The weather systems which affect east central Pennsylvania are generally of non-tropical origin. The storm tracks of less than 7% of all North Atlantic tropical cyclones enter Pennsylvania (Ref. 2.3-2). Systems which produce precipitation are divided into 3 groups. Cold fronts, trailing from cyclones passing to the north, occurring throughout the year, are the primary source of summer precipitation in the region. A second type of disturbance that produces precipitation in this area is the coastal low, originating in the Gulf of Mexico or in the Cape Hatteras region, which moves NNE along the coast. The greatest snowfalls in east central Pennsylvania are associated with this type of system. Major extra-tropical cyclones originating in the Gulf of Mexico, Texas Panhandle, or the lee of the Rockies which move northeast or east frequently give the region light or moderate snowfalls and rain. Tropical cyclones occasionally affect the region but very rarely retain hurricane force so far inland. Record rainfalls are often associated with decaying tropical cyclones.

Tornadoes seldom occur in Pennsylvania and those which cause severe damage or loss of life are rare.

The monthly average winds are westerly (Ref. 2.3-3). The wind in the region is constrained by the general direction (ENE-WSW) of the ridge and valley topography. Wind speeds in the region are light to moderate with monthly averages less than 10 mph.

Average temperatures range between 72°F in the summer and 25°F in the winter with extremes of 101°F and -21°F. Relative humidity is usually greater than 50%, often greater than 85% (Ref. 2.3-3a).

The Wilkes-Barre Scranton Airport at Avoca, Pennsylvania approximately 45 km northeast of the site, is the nearest National Weather Service Station. Based upon "STAR" summaries for the years 1971-1975 neutral stability conditions predominate at Avoca with Class C, D, and E occurring approximately 9, 59, and 13 percent of the time, respectively (Ref. 2.3-4).

The diffusion climatology of the region is generally good due to the prevalence of moderate wind speeds at most times. Occasional stagnant situations occur during the late summer and autumn when anticyclones stall over the northeastern U.S. It should also be noted that the plant's location

in the Susquehanna Valley can cause different stability conditions than those found concurrently at the top of the surrounding mountains or plateaus.

2.3.1.2 Regional Meteorological Conditions for Design and Operating Bases

2.3.1.2.1 Hurricanes

Hurricane winds seldom affect the area because of the rough terrain and the distance of the region from the ocean. Recently, Hurricane Agnes (June 1972) resulted in the worst natural disaster to hit the region because of the excessive precipitation it produced. Record flooding was recorded along the Susquehanna River. At Wilkes-Barre, 25 miles upstream from the site, the river crested on June 24 at a height of 40.91 feet, almost 8 feet above the previous record. Rainfall at Avoca, Pennsylvania for the period of Hurricane Agnes (June 21-22) was 3.10 inches. On August 18-19, 1955 the rainfall associated with Hurricane Diane was 4.58 inches (Ref. 2.3-3).

A tabulation of North Atlantic tropical cyclones with centers passing within 75 and 150 nautical miles of the Susquehanna site is presented in Table 2.3-1. The significance of these two distances is that points that lie within 75 to 150 nautical miles from the center of a hurricane may receive some heavy rainfall whereas points that lie between 0 and 75 nautical miles from the center of a hurricane are very likely to receive heavy rainfall. The frequency and recurrence interval of hurricane centers passing within 75 and 150 nautical miles is, respectively, 0.08 per year with an interval of 12 years, and 0.20 per year with an interval of 5 years (Ref. 2.3-2).

2.3.1.2.2 Tornadoes

The incidence of tornadoes in the site area is very low. Between the years 1950 and 1973 only 38 tornadoes were reported within 50 miles of the site. Tornado activity is at a maximum during the summer months with most tornadoes occurring in the late afternoon or evening. Figure 2.3-1 is a histogram for the years 1953-1962 showing tornado frequency by month, hour, and intensity within a 3° by 3° square which is centered on the site. The intensity categories are based on the Fujita tornado intensity classification (Ref. 2.3-5). From Figure 2.3-1 it can be seen that maximum tornado occurrence is in the summer. Diurnally, tornado frequency reaches a maximum during late afternoon, shortly after the period of greatest instability. For the period from 1950-1997, there were 5 tornadoes officially reported in Columbia County and 13 tornadoes officially reported in Luzerne County (Ref. 2.3-5a).

2.3.1.2.3 Thunderstorms

Thunderstorms in the area are usually of brief duration and concentrated in the warm months. They are responsible for most of the summertime rainfall which normally averages around 3.7 inches per month at Avoca, Pennsylvania. Based on a 19 year average at Avoca the mean number of "days with thunder heard" is 30 (Ref. 2.3-3). A monthly breakdown of the mean number of thunderstorm days that is representative of the site is shown in Table 2.3-2.

2.3.1.2.4 Lightning

There is neither documentation nor direct measurement of the occurrence of lightning other than the observation of associated thunder. Local climatological data tabulated by the National Weather Service (Ref. 2.3-3) does not provide information regarding the incidence, severity, or frequency of lightning occurrences. A thunderstorm can usually be heard unless the lightning causing the thunder is more than 15 miles away; therefore, thunder incidence can presumably be used to confirm the presence of some lightning.

The number of lightning strikes per square mile per year has been established by Uman (Ref. 2.3-6). The combined results of several studies summarized by Uman indicate that the number of flashes to the ground per square mile per year is between .05 and .80 times the number of thunderstorm days per year. The mean number of days with thunderstorms probably overestimates the actual occurrence of cloud-to-ground lightning since some thunderstorms probably contain only cloud-to-cloud lightning. Therefore, if the annual thunderstorm frequency at Avoca is used (30 days), the number of ground lightning strikes is between 2 and 24.

2.3.1.2.5 Hail

Hail in the site region sometimes falls from severe thunderstorms. Because hail falls in narrow swaths, only a small fraction of occurrences is recorded at regular reporting stations. The average annual number of days with hail at a point in the area is 23 (Ref. 2.3-7). The occurrence of large hail (greater than 3/4 inch diameter) averages 1 or 2 occurrences annually. According to Pautz (Ref. 2.3-7) the number of hailstorms with hail 3/4 inch and greater by 1-degree longitude-latitude squares was about 5 in the vicinity of the site for the period 1955-1967. For Avoca, Pennsylvania from 1973-75 there was one hailstorm each June, and one each in July of 1973 and 1974. In 1975 there was also one hailstorm in August and one in October. There were no occurrences of hail recorded in 1976 at Avoca (Ref. 2.3-3).

2.3.1.2.6 Extreme Winds

Strong winds occur in Pennsylvania as a result of the remnants or outer fringes of tropical systems, occasional hurricanes, thunderstorms, and tornadoes. The following is the fastest mile of wind and its associated direction, by month, at Avoca, Pennsylvania (1955-1976) (Ref. 2.3-3).

FASTEST MILE OF WIND

Month	mph	Direction	Month	mph	Direction
January	43	SE	July	42	NW
February	60	W	August	50	NE
March	49	S	September	38	SW
April	47	NW	October	38	E
May	40	NW	November	45	S
June	43	W	December	47	SW

The 50-year and 100-year mean fastest mile wind speeds for the site area are 75 miles per hour and 80 miles per hour, respectively (Ref. 2.3-8). According to Pautz, there were 8 windstorms 50 knots and greater for the 1 degree latitude-longitude square that includes the Susquehanna site for the period 1955-1967 (Ref. 2.3-7).

The gust factor was calculated as 1.3 from the following equation (Ref. 2.3-9).

$$G_F = \frac{G_f}{K_z(.00256V^2)}$$

Where

G_F is the gust factor to be applied to the fastest mile wind speed at 10 m above the ground.

K_Z is the velocity pressure coefficient at 10 m (.52).

V is the speed of the 100 year return period fastest mile wind (80 mph).

G_f is the velocity pressure (11.5).

2.3.1.2.7 Freezing Rain

Freezing rain can occur in the late fall, winter, and early spring. During the 50 years from 1919-1969 there were 4 occurrences of ice accumulation of 1 inch. The probability of an ice storm accumulating at least 1 inch in any year in the Northeast region of the U.S. is .24 (Ref. 2.3-10). At Avoca, Pennsylvania from 1973-1976 there were 57 days with freezing rain, 21 in January and 18 in February. There were nine occurrences each in March and December during that period. The duration of these phenomenon never exceeded 12 hours and was usually less than 3 hours (Ref. 2.3-11).

2.3.1.2.8 Duststorms

Because the soil in Pennsylvania is usually moist all year the likelihood of a duststorm is small (Ref. 2.3-12). There were no recorded duststorms for the period 1972-1976 at Avoca, Pennsylvania (Ref. 2.3-11).

2.3.1.2.9 High Air Pollution Potential

The meteorological conditions that are generally conducive to high air pollution potential are light winds, stable boundary layers, and near surface based inversions. Holzworth (Ref. 2.3-13) studied the episodic occurrence of several limited dispersion conditions at each of 62 upper air stations in the United States. He considered episode durations of at least 2 days and at least 5 days. Twelve different limited dispersion conditions were used to define each episode. Each condition was defined by a different combination of mixing height and wind speed. Intermediate limiting conditions of mixing heights less than or equal to 1500 m and wind speeds 4.0 m sec^{-1} or less with no significant precipitation during episodes lasting at least 2 days are of interest because such criteria have been used as criteria by the National Pollution Potential Forecasting Program (Ref. 2.3-13). The approximate number of episode-days at the site area is 25 in 5 years. This is much less than in the western half of the country and less than most of the East. Table 2.3-3 presents a

summary of the data at stations presented by Holzworth which are closest to the site. Days with high air pollution correlate to days with minimum low level atmospheric mixing and dispersion.

2.3.1.2.10 Snowpacks

Severe snowstorms are not frequent in the area. Snowfall averages between 40 and 50 inches a year in the site region. At Avoca, Pennsylvania the extreme 24-hour snowfall was 20.5 inches in November, 1971 but the greatest snowfall of record was 21.1 inches over a 29 hour period in January, 1964. The extreme seasonal snowfall was 76.8 inches in 1969-1970 (Ref. 2.3-3).

The 100 year mean recurrence interval snow load on the ground is 122.02 kgm^{-2} (25 lbs ft^{-2}) (Ref. 2.3-9). The 100 year mean recurrence snow depth for Avoca, Pennsylvania is 28.6 inches (Ref. 2.3-15).

Assuming the maximum probable winter precipitation falls on top of the 100 year mean recurrence interval snowload yields a conservative estimate of the maximum probable combined snowload.

Assuming that the 100 year mean recurrence interval snowload occurs during January, which has the lowest average monthly temperature and the greatest snowfall of record, the weight of the 48 hour probable maximum winter precipitation for January must be added to it. The weight of the 48 hour probable maximum winter precipitation for January is 287.0 kg m^{-2} (59 lbs ft^{-2}) (Ref. 2.3-14). Thus, the weight of the probable maximum combined snowload at ground level is 409.02 kg m^{-2} (84 lbs ft^{-2}).

2.3.1.2.11 Design Basis Tornado

The development of a Design Basis Tornado (DBT) follows the premise that the probability of occurrence of a tornado exceeding the DBT should be on the order of 10^{-7} per year (Ref. 2.3-16). The 10^{-7} per year design tornado was determined for a 3° latitude by 3° longitude area encompassing the site. The tornado path lengths and widths in the area of interest were used in the probable calculation.

The first step in the procedure is the computation of the geometric probability which is given by the following equation:

$$P_s = \overline{n}(a / A) \qquad \text{(Eq. 2.3-1)}$$

where P_s is the mean annual probability of a tornado striking a point, $\bar{n}$ is the mean number of tornadoes occurring within the area A per year, and "a" is the mean path area determined from the log-normal distribution.

The design basis wind speed is one which satisfies the condition $P_sP_i < 10^{-7}$ yr^{-1} where P_i is the acceptable intensity probability and is determined from the plot of cumulative F-scale intensity frequencies on log-probability paper. The F-scale is an estimate of tornadic wind speed range based upon damage inspection and has been compiled for the years of 1971 and 1972 (Ref. 2.3-17).

The average rate of pressure drop within the radius of maximum winds is determined by: (Ref. 2.3-19)

$$\frac{dp}{dt} = \frac{\Delta pT}{2r_m} \qquad \text{(Eq. 2.3-2)}$$

Where:

Δp	=	pressure change
t	=	time
T	=	translational speed
r_m	=	radius rotational wind speed = 150'

The total pressure drop, p, is determined by the application of the cyclostrophic wind equation:

$$\int_o^r \frac{\partial\rho}{\partial r} dr = \int_o^r 2\rho A \frac{V^2 m}{r_m} dr \qquad \text{(Eq. 2.3-3)}$$

where V_m is maximum rotational wind speed and is the density of air (1×10^{-3} gm/cm^3).

The region from which tornado path length and width statistics were selected was between 75° to 78° longitude and 40° to 43° latitude; approximately centered on the site location. Of the 63 tornadic events thirty four values of path length and width were found for the period 1950-1973 based upon the National Severe Storms Forecast Center's tornado tape.

The geometric probability is calculated by substituting the following parameters into Equation 2.3-1:

Where,

	a	=	0.388 mi^2
	A	=	32,265 mi^2
	$\bar{n}$	=	63/24yr = 2.625 yr^{-1}
	P_s	=	3.157×10^{-5} yr^{-1}
and	Pi	=	3.168×10^{-3} yr^{-1}

This results in a design wind speed of 260 mph for a probability of 10^{-7} yr^{-1} (Ref. 2.3-19). The value of "a" is conservative in comparison with a value of .26 mi^2 for the combined states of Pennsylvania and West Virginia and .37 mi^2 for New York state for the period 1953-1972 (Ref. 2.3-18). Although the value of "a" was based on only 34 of the 63 tornadoes it was conservatively assumed that all 63 tornadoes had a mean path area of 0.388 mi^2. In this region, the highest tornadic intensity was F 2 or 157 mph. Thus, the 260 mph design wind speed is conservative with respect to the local historical record.

The rate of pressure drop and the total pressure drop are determined directly from Equations 2.3-2 and 2.3-3, respectively. The maximum translational wind speed was interpolated from the Region

II and Region III values (Ref. 2.3-16). The design basis parameters calculated for the Susquehanna Steam Electric Station are (Ref. 2.3-19):

Total Maximum Wind Speed	Rotational Wind Speed	Maximum Translational Speed
260	205	55

Minimum Translational Speed	Total Pressure Drop	Rate of Pressure Drop
5 mph	1.9 psi	0.9 psi/sec

The actual design basis parameters that were used for the Susquehanna design are presented in Section 3.3.

2.3.1.2.12 Ultimate Heat Sink

An analysis of the ultimate heat sink is presented in Section 9.2 of the FSAR.

This analysis is based on 11 years of meteorological data collected on site as well as 34 years of meteorological data collected at Avoca Airport near Scranton, Pennsylvania.

A computer-aided search was done for both data bases to determine two periods of time for use as the Ultimate Heat Sink design meteorology. One was chosen such that the ability to cool sprayed water is minimized (minimum heat transfer case). The other was chosen such that the potential for water loss is maximized (maximum water loss case). The selection of this meteorology is discussed further in Subsections 9.2.7.3.5 and 9.2.7.3.7.

2.3.2 LOCAL METEOROLOGY

2.3.2.1 Normal and Extreme Values of Meteorological Parameters

2.3.2.1.1 Wind

The following data sources were used as the basis of this section: long-term data from Wilkes-Barre Scranton Airport at Avoca, Pennsylvania (Ref. 2.3-3), four years of data (1973-1976) collected at the 31.5 and 300 feet levels and five years of data (1999-2003) collected at the 10 m and 60 m levels of the Susquehanna meteorological tower located at the site.

The Avoca station is located about 30 miles northeast of the Susquehanna site. It is reasonably representative of the site due to their close proximity to one another and similar topography.

Table 2.3-6 is a summary of long-term wind data for Avoca (Ref. 2.3-3). It shows the annual average speed is 8.4 miles per hour and the prevailing direction is southwest. The monthly

average wind speeds are greatest in the spring (9.6 mph in April) and lowest in the late summer (7.2 mph in August). The prevailing wind direction is SW or WSW for every month except March when it is NW. Table 2.3-7 is a similar summary for the on-site data.

Lower level (31.5 feet) data from the Susquehanna site for the 4 year period show an average wind speed of 4.45 mph (1.99 m/sec). The prevalent direction over the 4 years was the WSW closely followed by W. The ENE direction presents a secondary maximum in frequency of occurrence.

Tables 2.3-8 through 2.3-16 provide wind persistence data for the Susquehanna site on an annual basis, at the lower level, for each stability class, all classes combined, and all stable classes.

The joint frequencies of wind speed, direction, and stability at both the lower and upper levels were updated in 2005 to use the 5 year period (1999 – 2003) are found in Tables 2.3-75 through 2.3-91.

The overall southwest to northeast orientation of topographic ridge lines in the SSES vicinity has a profound influence on the low level winds. At Avoca, PA, the mean annual wind direction is from the southwest. At the SSES site, the predominant wind directions measured at the 10-meter level are from the east-northeast and from the southwest. The Susquehanna River Valley orientation effectively funnels a localized, low level wind flow up or down the valley. The river valley environment is also favorable for stable meteorological conditions in the lowermost portion of the atmosphere characterized by little to no wind and the presence of fog. This is most prevalent during the overnight hours. The river valley influence on atmospheric stability at the SSES site makes stability conditions unique and often quite different when compared to stability conditions at Aroca.

2.3.2.1.2 Temperature and Atmospheric Water Vapor

Table 2.3-17 presents the long-term monthly average and extreme temperatures for Avoca, Pennsylvania. July is the warmest month with a long-term average maximum temperature of 82.2°F, an average minimum temperature of 61.8°F, and a mean of 72.0°F. The coolest month is January, having an average temperature range of 32.3°F to 17.7°F and a mean temperature of 25.0°F. The average annual diurnal variation is 18.1°F (Ref. 2.3-3a).

East Central Pennsylvania experiences the temperature extremes associated with mid-latitude traveling low pressure disturbances. The temperature extremes at Avoca, Pennsylvania are 101°F in July of 1966 and -21°F in January, 1994. Average Avoca, Pennsylvania dewpoint and relative humidity data are contained in Table 2.3-18 (Ref. 2.3-3a).

At the Susquehanna site during the period 1973-1976 dry bulb temperatures ranged from a high of 34.3°C (94°F) to a low of -20.9°C (-6°F). The average temperature was 9.3°C (49°F). July had the highest average temperature 20.3°C (69°F) while January had the lowest with -2.1°C (28°F). The average wet bulb temperature was 6.9°C (44°F) with the months of July and August averaging 17.7°C (64°F). The average relative humidity was 70% with the month of August averaging 82%. A summary of the site data is presented in Tables 2.3-19 through 2.3-32.

For the period of 1981-1996, the maximum SSES average hourly temperature of 37.8°C (100°F) occurred on July 16, 1996. The minimum average hourly temperature at SSES for this period was –30.8°C (-23.1°F) on January 21, 1994.

2.3.2.1.3 Precipitation

The region surrounding the Susquehanna site has a moderately moist climate averaging just over 36 inches of rainfall per year spread quite evenly over all months of the year. There is a slight maximum during the summer when there is a greater effect of tropical air masses and thunderstorms. The average monthly and maximum 24-hour precipitation for Avoca, Pennsylvania are given in Table 2.3-33. The greatest 24-hour rainfall amount reported at Avoca, Pennsylvania was 6.52 inches in September 1985, associated with the remnants of Hurricane Gloria. The greatest 24-hour snowfall, 20.5 inches occurred with the Thanksgiving Day storm of November 24-25, 1971, but the greatest snowfall of record was 21.1 inches over a 29 hour period on January 12-13, 1964 (Ref. 2.3-3).

Table 2.3-34 presents the expected rainfall by duration and recurrence intervals for the area around the Susquehanna site as compiled by the National Weather Service (Ref. 2.3-20). The probable maximum precipitation for various rainfall duration in East Central Pennsylvania by area size is presented in Table 2.3-35. Assuming 10 square miles is most representative of the power plant site, the probable maximum rainfall ranges from 25 1/2 inches in 6 hours to 36 1/2 inches in 72 hours (Ref. 2.3-21). The rainfall rate distribution curves are presented for Scranton, Pennsylvania in Figure 2.3-2. The 100 year return period rainfall rate is 2.5 inches for a 1 hour period.

Table 2.3-36 presents the summary of on-site precipitation data for the 4 year period. The site averaged a total of 47.83 inches annually with the greatest occurring in September (7.54 inches) and the minimum in December (2.21 inches). Data on the rainfall frequency, and duration of precipitation for the Susquehanna site are presented in Tables 2.3-37 through 2.3-49 by month and for the 4 year period. Precipitation wind roses are presented by month and for the total period in Tables 2.3-50 through 2.3-62.

2.3.2.1.4 Fog and Smog

At Avoca, Pennsylvania between the years of 1973-1976 there was an average of 86 days of haze and smoke reported. Most of the days were in the summer months. Over the same period, the three hourly observations of fog averaged 250 for a year. Fog was usually observed with rain or snow and most often in the early fall months. The average number of days with heavy fog for the period was 21. Table 2.3-63 presents the heavy fog occurrences at Avoca, Pennsylvania for recent years. Based on National Weather Service data from Avoca, Pennsylvania from a 22 year period, heavy fog (visibility 1/4 mile or less) occurs 24 times per year. No on-site data on fog, or haze, is available.

2.3.2.1.5 Stability

Atmospheric stability at the Wilkes-Barre Scranton airport based on STAR data for the period 1971-1975. The STAR data for the period 1971-1975 were selected because they represented the most recent five year period which was available at the time and the fact that a five year period of record is generally regarded as being representative of long-term meteorological conditions. The 1971-1975 period also shows the prevailing direction to be from the SW at an average speed of 8.5 mph. STAR data for the fire year period 1960-1964 also show that the prevailing wind direction is from the SW at an average speed of 6.7 mph. For a 22 year period of record the prevailing wind

direction was SW at an average speed of 8.4 mph. The relative stability distribution of these two five year periods are:

Pasquill Stability Class	1960-64	1971-75
A	0.41	0.32
B	5.27	4.84
C	10.04	8.92
D	54.27	58.64
E	12.36	13.09
F	13.05	11.16
G	4.60	3.01

The seasonal occurrence of E and F stabilities are given below for the 1971-75 period:

SEASONAL OCCURRENCE (%) OF E AND F STABILITIES

	Winter	Spring	Summer	Autumn
E	12.3	12.0	14.9	12.9
F	8.5	8.9	14.0	13.0

Tables 2.3-64 to 2.3-71 are annual stability summaries by wind speed and direction from Avoca, Pennsylvania data for the years 1971 through 1975 (Ref. 2.3-4). The analytical technique for classifying stability is based upon three hourly observations and is dependent primarily upon net solar radiation and wind speed. For the entire period neutral and sightly stable most often occur. The on-site stability summaries by wind speed and direction are presented in Tables 2.3-75 through 2.3-91.

Studies by Holzworth (Ref. 2.3-33) indicate that for Northeastern Pennsylvania unstable conditions (A, B, C) occur 16-25 percent of the time while neutral (D) conditions prevail 46-55 percent of the time and stable conditions (E, F, G) occur 26-35 percent of the time. For the 4 year period 1973-1976 the on-site data showed the following stability frequencies: Pasquill class A-16 percent, B-7.6 percent, C-4.2 percent, D-30.8 percent, E-26.2 percent, F-10.5 percent, G-4.5 percent. This indicates that the site is prone toward stable conditions (41.2%) rather than neutral conditions (30.8%).

Representative mixing heights on a seasonal and diurnal basis obtained by averaging data from Albany, and New York, New York; Pittsburgh, Pennsylvania; and Washington, D.C. (Ref. 2.3-13) are presented in Table 2.3-72.

Low level atmospheric stability is influenced by insolation. The relatively high latitude of the SSES site (approximately 41° North) has a profound impact on the length of daylight. At the winter solstice (around December 21), the time elapsed between sunrise and sunset is 9 hours, 11minutes. At the summer solstice (around June 21), the time elapsed between sunrise and sunset is 15 hours, 10 minutes, or a difference of 6 hours (Ref. 2.3-13a).

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

The expected characteristics and effects of water vapor plumes entering the atmosphere arising from the operation of two natural draft cooling towers have been evaluated.

The characteristics and effects associated with cooling tower operation were determined in terms of:

a) Monthly and annual frequency distributions of plume length with respect to distance and direction out to 20,000 ft.

b) Monthly and annual frequency distributions of ground level plumes (fogging) with respect to distance and direction.

c) Monthly and annual frequency distributions of ground level plumes accompanied by subfreezing temperatures (icing) by distance and direction.

d) Monthly and annual frequency distribution of increases in relative humidity and temperature with respect to distance and direction.

Simulations were obtained from a computerized diffusion model that simulates vapor plume length and the occurrence of ground level fogging or icing. The Gaussian plume theory distribution is assumed with buoyancy approximated by a dry plume rise equation. The computer program utilizes cooling tower performance data and on-site meteorological observations for 1976 (ambient temperature, dew point, wet bulb temperature, wind velocity, and atmospheric stability) to determine the downwind dispersion of water vapor at plume centerline and ground level.

The year 1976 was selected because it was the most conservative year with respect to atmospheric dispersion conditions of the four years of on-site data. It is also conservative with respect to long-term atmospheric conditions.

The model used was developed by Dames & Moore and has been used in previous submittals to NRC. The model was presented by Bowman, W. Alan and Biggs, W. Gale in their paper entitled "Meteorological Aspects of Large Cooling Towers" presented at a APCA Conference in Miami, Florida in June, 1972. The height at which each meteorological measurement input to the model was taken is given below:

Wind Speed..300 ft.
Wind Direction......................................300 ft.
Temperature...31.5 ft.
Relative Humidity ....31.5 ft.
Stability ....300 ft. - 31.5 ft.

Generally, the longer plume lengths occur more frequently in the winter months in the early morning hours when the relative humidities are high. The visible plumes were computed to extend laterally beyond 20,000 feet (4 miles) approximately 30 percent of the time in the sectors of maximum occurrence (NE and ENE) and 70 percent for all sectors. Visible plumes occurred least frequently in the WNW through NNW sectors with computed plume lengths beyond 4 miles occurring with a frequency of 2.1 percent to 2.5 percent annually. There were no computed occurrences of ground fogging. Relative humidity increases of 2.5 percent above ambient did not occur.

No occurrences of icing were computed. Likewise, no computed increases in surface temperature of 0.5^{o}C or greater were projected in the study sample.

In conclusion, the frequent (70%) long visible plumes are the primary meteorological effects to be experienced from the operation of the Susquehanna cooling towers. There is no fogging or icing expected. The inducement of other weather modification effects such as rainfall augmentation is unlikely due to the small percentage increase in atmospheric moisture introduced into the already moisture laden environment.

Further details of this analysis are provided in Subsection 5.1.4 of the Environmental Report.

The topography surrounding the site consists of ridges and valleys. Figure 2.1-11 shows the topography within a 5 mile radius of the site. The cross-sections of elevation centered on the plant along the 16 cardinal directions to a distance of 50 miles are shown in Figures 2.3-4-1, 2.3-4-2, 2.3-4-3, 2.3-4-4, 2.3-4-5, 2.3-4-6, 2.3-4-7, and 2.3-4-8.

2.3.2.3 Local Meteorological Conditions for Design and Operating Bases

All local meteorological and air quality conditions used for design and operating basis considerations and their bases, except for those conditions referred to in Subsections 2.3.4 and 2.3.5, are provided in Subsections 2.3.1.2.1 through 2.3.1.2.11. Current site meteorological information is documented on a regular basis in the SSES Annual Effluent and Waste Disposal report.

2.3.3 ON-SITE METEOROLOGICAL MEASUREMENTS PROGRAM

The on-site meteorological program is designed to provide a complete climatology of the site area, but most importantly to provide dispersion climatology for use in safety planning of radioactive effluent releases and as a means of determining the appropriately conservative meteorological parameters to be used in estimating the potential consequences of hypothetical accidents. Analysis of collected meteorological data permits an assessment of the diffusion parameters characteristic of the site.

2.3.3.1 Location and Description of the Tower Site

The site is about 8 km (5 mi.) ENE of Berwick, Pennsylvania. The primary meteorological tower, commonly referred to as the Primary Meteorological Tower, a 200 foot steel framed tower, is located about 340 m to the southeast of the cooling towers. The area is generally level, increasing slightly in elevation to the north and west. South and east of the tower the topography slopes down towards the Susquehanna River. Vegetation in the immediate vicinity is low weeds with some deciduous trees in a gully to the south. The deciduous trees are approximately 40 feet in height and are approximately 100 feet from the tower. In 1994, an ash facility was placed approximately 185 feet north of the Primary Meteorological Tower. The maximum height of this structure is approximately 30 feet.

In November 1972 three meteorological instrumentation platforms were constructed. The primary tower was erected on the Susquehanna nuclear power station site at the same altitude as the station (approximately 650' msl) between the station and the Susquehanna River. The purpose of the primary tower is to estimate the stability and movement of the air layer into which the effluent from the facility could be released as required by NRC Regulatory Guide 1.23 (Ref. 2.3-22). In addition to the primary tower, 75 foot and 10 foot instrumented poles were erected at site vicinity.

The 75 foot tower was at 1115' msl on a hill to the NW of the station and the 10 foot tower was below the station towards the river at 500' msl. The purpose of the 75 foot pole was to provide sensing of wind, temperature, and humidity parameters at an elevation comparable to the elevations of the cooling tower plumes. The 75 foot tower was removed on January 14, 1974 due to construction requirements. The data from the primary meteorological tower provides sufficient information for the cooling tower analysis. The 10 foot tower in the valley below the station, was removed on November 14, 1975, after three years of data had been collected. Figure 2.3-5 presents a schematic of the sites and the instrumentation.

In compliance with the requirements of NUREG-0654 a backup meteorological tower was erected in 1982. This tower is commonly referred to as the Backup Tower. The tower is a 30-foot instrumented utility pole located northeast of the station and across from the Training Center. The purpose of the backup tower is to provide sensing of wind parameters at the 10-meter level. Additionally, in 1985 two supplemental towers were installed in the river valley near the station to provide additional data to more accurately model the effects of surrounding terrain on atmospheric dispersion and transport. One tower is located UPRIVER approximately 1.2 miles NNE of the station off Route 11 towards Shickshinny; the second tower is located DOWNRIVER approximately 3.6 miles SW of the station off Route 93 just east of Nescopeck. Meteorological validation of the UPRIVER supplemental tower data was terminated on October 1, 1994 and the UPRIVER supplemental tower equipment was abandoned in place at that time .

Both The DOWNRIVER tower measures wind speed, wind direction and sigma theta at the 10 meter level. The DOWNRIVER tower also measures temperature at a height of approximately 6.6 feet.

The meteorological data collected from the DOWNRIVER tower continues to be validated and is used only to support assessment and restoration efforts in the event there is an accidental release of radioactive material from SSES.

2.3.3.2 Types of Measurements Made

The parameters which are monitored for conformance to NRC Regulatory Guide 1.23 (Ref. 2.3-22) commitments are wind speed, wind direction, temperature, delta temperature, dewpoint temperature and precipitation. Delta temperature accuracy criteria is monitored for conformance to AEC Safety Guide 123 (Ref. 2.3-22a). The parameter, heights, and number of sensors installed at the Susquehanna site are listed in Table 2.3-73.

2.3.3.3 Description of Instruments

The wind sensor consists of a 3 cup anemometer and coupled drive shaft that responds to wind and rotates a multi-section light beam chopper. Rotation of the chopper alternately masks and exposes a phototransistor to a miniature light source. The phototransistor responds to the light passing through the chopper wheel and generates an electrical output which has a frequency proportional to wind velocity. This signal is then sent to a translator for further conversion. The accuracy required for the wind speed measurement is ±0.5 mph for speeds less than 5 mph and ±10% for speeds above 5 mph. This requirement is met by the instrumentation used on the primary tower.

The wind direction sensor is comprised of a counterbalanced lightweight vane coupled to a precision potentiometer assembly by the drive shaft, causing the potentiometer wiper to directly follow movements of the wind vane. The position of the vane is sensed by the potentiometer and is sent to a translator as a DC voltage. The accuracy requirement for the wind direction measurement is ±5° of azimuth with a starting threshold of less than 1 mph. This requirement is met by the instrumentation used on the primary tower.

On the primary tower the temperature measuring system consists of multiple thermistor composite sensors. Two sensors are mounted in motor aspirated radiation shields at each of the 10 meter and 60 meter levels. The thermistor sensors are connected in a resistive network and powered by a D.C. voltage to produce a voltage that varies approximately linearly with temperature as a translator output. Each translator produces two channels of output; one channel of one translator provides the ambient temperature output for 10 meters and a second, comparator channel provides differential temperature output derived from one 10 meter sensor input and one 60 meter sensor input. The two separate sets of 10 meter and 60 meter sensors provide one 10 meter ambient temperature measurement and two difference temperature measurements between the 10 meter and 60 meter levels.

Accuracy required for the ambient temperature measurement is ±0.5°C. This requirement is met by the instrumentation used at the primary tower.

Accuracy required for the temperature difference measurement is ±0.1°C/50m. This requirement is met by the instrumentation used on the primary tower.

The dewpoint temperature is measured on the primary tower with bifilar wire electrodes wound on a cloth sleeve which covers a hollow tube or bobbin. The bifilar electrodes are not interconnected, but depend on conductivity of the atmospherically moistened lithium chloride treated bobbin for current flow. As the moisture content in the air increases, the lithium chloride absorbs water vapor and becomes conductive. Current then begins to flow between the electrodes energized by low AC voltage, and heats the bobbin. Some of the moisture is thereby evaporated until an equilibrium temperature is reached on the bobbin. The equilibrium temperature is related to the dewpoint temperature of the air. A thermistor sensor is mounted inside the bobbin to measure the cavity temperature which is converted in analog outputs, representing dewpoint temperature by a electronic temperature translator. The accuracy required for the dewpoint temperature measurement is ±1.5°C. This requirement is met by the instrumentation used on the primary tower.

On the 10 meter level of the primary tower a motor aspirated temperature and dewpoint shield houses two thermistor sensors, and the dewpoint sensor. At the 60 meter level two motor aspirated temperature shields each houses a thermistor sensor.

Precipitation is measured in a Tipping Bucket Rain Gauge at the primary tower site. This is a remote reading gauge which produces a signal proportional to total rainfall. Precipitation is collected in a collection opening and is funneled to the two buckets of the tipping mechanism. As one bucket fills with water, the weight causes it to lower, tip, and empty while the bucket on the opposite side is simultaneously raised to receive additional water. Each tipping phase causes a momentary switch closure. This closure actuates a digital counter directly proportional to accumulated rainfall. The required accuracy for the rainfall measurement is ±10% of the total accumulated catch for amounts in excess of 0.2 in. This requirement is met by the instrumentation used on the primary tower.

Vertical diffusion coefficients are computed from the vertical temperature differences. Wind sigma standard deviation of wind direction is measured at the 10 and 60 meter levels and used to compute horizontal diffusion coefficients. Sigma theta calculations based on wind direction measurements are used as a backup to temperature readings to monitor atmospheric stability.

The outputs of all sensors are handled by a modular translator system designed to convert the sensor outputs into a standardized voltage/current output. Each input channel is allotted one circuit designed for a particular sensor, such as wind speed, wind direction, or temperature, etc. The necessary signal processing and scaling is contained in each circuit to provide an electrical output of uniform range. There are two outputs from each circuit. One low voltage output is directed to a data logger accessible via telephone modem and a second low voltage output to a telemetry transmitting system which directs a specific frequency/parameter signal to telemetry receiving device in the control room which converts this signal to a 4 to 20 ma output which then inputs to an appropriate recorder in the main control room.

Each translator circuit has internal zero and full scale calibration facilities. Each calibrator switch has a "normal" position which allows normal recording of data. When depressed the calibrator switch provides a signal to the individual translator circuit producing a zero or full scale signal to the recorders. The indicated output in meteorological units for each position of the calibrator is given below.

Parameter	Zero	Full Scale	Type of Calibration
Wind Speed	0 mph	100 mph	Calibrated Voltage
Wind Direction	0°	540°	Calibrated Voltage
Temperature	-20°F	+100°F	Precision Resistance
Dewpoint	-40°F	+100°F	Precision Resistance
Delta temp	-5°F	+5°F	Calibrated Voltage
Precipitation	0 in	1 in.	Calibrated Voltage

2.3.3.4 Data Recording Systems

The primary data recording system used for the Susquehanna site's primary tower is a digital data acquisition system. The system is an integrated data conversion and recording station which scans up to 16 analog signal outputs, converts each 0 to 1 V DC input to a digital code which is stored and retrieved via modem interface.

The secondary recording system is the Control Room recorders.

It is estimated that approximately 10% of the data used at the Susquehanna site was obtained from strip chard records.

Spot checks were made to compare the strip chart and digital data. Although no formal records of these comparisons were prepared, it is estimated that the average differences between strip chart and digital data were as follows:

Temperature	1°F
Wind Speed	1 mph
Wind Direction	5 degrees
Dewpoint	1°F
Temperature	0.5°F
Precipitation	.05 inches

All telemetry transmitters, translators, and the data logger are housed in a weatherproof cinderblock building. This building has thermostatically controlled heating and air conditioning.

2.3.3.5 Calibration and Maintenance of the System

All calibration and maintenance is performed at least semi-annually in accordance with the frequencies and procedures prescribed in the manufacturer's operating and maintenance manual.

2.3.3.6 Data Analysis

The analog recording system provides a back-up in case of digital system failure, so that a high data recovery rate can be maintained. Table 2.3-74 gives the recovery rates for each year.

An hourly average for each parameter is computed. Data validity, range of hourly averages, and the number of valid observations contributing to the averages are tabulated to assist in the determination of data reliability. Comparisons between the analog and digital data are performed when the review of the digital data reveals questionable or invalid data.

Temperature and dewpoint hourly averages are computed using the following scalar equation:

where:

$$\overline{B}_j = \frac{1}{n}\sum_{i=1}^{n} r_j B_{ji}$$

$\overline{B}_j$ = the average hourly value for the variable (in physical units)

n = the total number of minute observations during the hour (normally 60), but if n is less than 15 for that hour, data are considered to be missing;

B_{ji} = the i^{th} minute observation on the j^{th} variable (millivolts);

r = the conversion factor to change the jth variable from millivolts into physical units.

After wind speed (WS) and wind direction (WD) are converted from millivolts they are related in the following manner:

If WS is invalid (999) then WD is marked invalid (999) and vice versa

If WS > threshold (non-calm) and WD = 0 (implying calm) then WD is set to 360° (North)

If WS < threshold (calm) and WD > 0 (implying non-calm) then WD is set to 0° (calm)

Hourly averages are computed as scalars for wind speed. Wind direction averages are determined by vector analysis for all non-calm wind distribution of the lowest non-calm wind speed class by stability class.

If the associated average WS is less than .36 mps then average WD is set to 0° (calm) and average WS is set to 0 mps (calm).

NRC Regulatory Guide 1.23 (Ref. 2.3-22) suggests that data be averaged over a period of at least 15 minutes once each hour. Hours containing less than 15 minutes of valid data are invalidated. The hourly averaged data are reviewed for validity, completeness, and reliability. Periods containing problems are then replaced by analog data.

Data analysis for diffusion characteristics for the site requires three basic atmospheric variables. These three variables, together with the primary and secondary (back-up) measurements for each, are as follow:

Horizontal wind speed	primary-10 m wind speed; secondary-60 m wind speed
Horizontal wind direction	primary-10 m wind direction; secondary-60 m wind direction
Temperature difference (ΔT)	primary-delta T's from 10 m to 60 m; secondary-ΔT from 10 m to 60 m

If the 10 m wind speed is unavailable the 60 m wind speed is reduced to the equivalent 10 m value as follows:

$$V_{10} = V_j \left(\frac{10}{H_j} \right)^S$$

where:

H_j = sensor height, meters

V_{10} = the equivalent 10 m wind speed

V_j = the 60 m wind speed

S = 0.25 for Pasquill classes A, B, C, and D
0.50 for Pasquill classes E, F, and G

The percentage of data recovery for the 10 m wind sensor indicates the extent of this substitution for the data period.

Temperature difference values are used to determine Pasquill stability classes. Atmospheric dispersion coefficients are assigned according to stability class and downwind travel distance.

The hourly values of the meteorological parameters are then processed to obtain the following:

a. joint frequency distributions of wind speed and stability for lower and upper levels (Tables 2.3-75 through 2.3-91)

b. wind direction persistence summaries by stability class

c. maximum, minimum and diurnal variation of temperature, and humidity

d. annual average values of relative concentration with direction and distance

e. frequency distribution of concentrations for the 0-2 hour, 0-8 hour, 8-24 hour, 1-4 day and 4-30 day time periods.

2.3.4 SHORT-TERM (ACCIDENT) DIFFUSION ESTIMATES

Atmospheric diffusion conditions (expressed as values of χ/Q) developed for use in evaluating accidents hypothesized in Chapter 15 are discussed in this section for various periods after an accident. This includes χ/Q estimates based on the methods described in Regulatory Guide 1.145. (Reference 2.3-34) All estimates use vertical temperature difference to determine stability classification. Tables 2.3-75 through 2.3-82 and 2.3-84 through 2.3-91 give the joint frequency distribution of temperature difference categories used to summarize 5 years of SSES data into Pasquill groups for use in computing σ_y and σ_z in the diffusion equations. Results are based on evaluation of a recent 5-year period of onsite meteorological data (1999-2003). A description of the site meteorological program is given in Section 2.3.3.

Methods used to estimate diffusion conditions for evaluating short-term accident releases are discussed in Section 2.3.4.1, and methods for assessing the consequences of longer term accident releases (up to 30 days) are discussed in Section 2.3.4.2.

2.3.4.1 Short-Term (0-2 hours) Releases

The methodology for determining the atmospheric dispersion that exists for short-term releases involves direction-dependent and direction-independent calculations as described in Regulatory Guide 1.145. Both methods include the effects of plume meander as discussed below.

2.3.4.1 1 Direction-Independent Calculations

The direction-independent approach involves computing χ/Q values for each hour of the period of SSES records used and then counting all of the hours that had χ/Q values equal to or greater than a selected value regardless of direction. The number of hours so obtained was then divided by the number of hours in the total period of record to obtain the probability that the selected χ/Q value would be equaled or exceeded. The resulting probabilities are independent of wind direction. A plot of cumulative centerline χ/Q values as a function of probability of occurrence was constructed using the SSES hourly data for all 5 years combined as shown in Figure 2.3-6. Equations 2.3-4, 2.3-5 and 2.3-6 in Section 2.3.4.4 were used to compute values of χ/Q. The distance to the site boundary (exclusion area boundary referred to as the EAB) was assumed to be a circle with a radius of 0.34 miles (549 meters).

2.3.4.1 2 Direction-Dependent Calculations

The direction-dependent calculations outlined in Regulatory Guide 1.145 require the χ/Q values to be calculated using the equations given in Section 2.3.4.4; however, the results are treated separately for each direction. A 5-year composite direction-dependent probability distribution was plotted by combining the frequency of occurrence of selected χ/Q values for each direction at the EAB as shown in Figure 2.3-7.

2.3.4.1 3 Determining Appropriate Short-Term Dispersion

In accordance with Regulatory Guide 1.145, the two cumulative probability distributions shown in Figures 2.3-6 and 2.3-7 are used to determine the appropriate χ/Q value at the EAB distance. The peak 5% value read from Figure 2.3-6 is 6.5E-4 s/m^3. For the direction-dependent case, the 0.5% χ/Q is determined from Figure 2.3-7 to be 8.3e-4 s/m^3. The highest of the two (χ/Q = 8.3E-4 s/m^3) is to be used in accident dose calculations and is shown in Table 2.3-92 for the 1-hour case at the EAB. Tables 2.3-93 through 2.3-98 show the direction dependent results for each of the separate years plus the total for all five years at the EAB. For "realistic" dose calculations the 50% direction independent value is also shown in Table 2.3-92.

Similar calculations for short-term dispersion were made at the LPZ distance of 3 miles (4827 meters). Tables 2.3-99 through 2.3-104 give the direction-dependent χ/Q probability distributions for 1 hour for each of the 5 years and the weighted 5 year average at the LPZ.

2.3.4.2 Long-Term Releases

For releases that occur over a longer period, it is appropriate to incorporate wind direction changes in the model used to estimate concentration at any given point. Using the same 5-year period of data from SSES, the probability that any particular average diffusion condition (or poorer one) would exist during a selected interval of time (greater than 1 hour) was determined.

The procedure for determining longer term-χ/Q values is also taken from Regulatory Guide 1.145. The calculation is made using the 5-year data set. The highest 0.5% direction-dependent short-term (1 hour) χ/Q value was used because it is greater than the 5% direction-independent at the LPZ. These χ/Q values are plotted on Figures 2.3-8 and 2.3-9 as a function of averaging time. Only the 1-hour values are used in the Reg. Guide 1.145 interpolation method. The long-term 0.5% χ/Q values for defined averaging times are determined by a log-

log interpolation between the maximum direction dependent annual average and the maximum 0.5% direction dependent 1 hour value used at the 2 hour averaging time. The long term 50% values are determined by interpolating (log-log) between the 50% direction independent 1 hour value at the 2 hour averaging time and the direction independent annual average (see Figure 2.3-10). The interpolated values are summarized in Table 2.3-105. For the 50% probable case the direction independent values were used because they are higher than the direction dependent values.

2.3.4.3 Analytical Methods for Dispersion Computations

During neutral (D) or stable (E, F, or G) atmospheric stability conditions when the wind speed at the 10 meter level is less than 6 meters per second, horizontal plume meander is taken into account. χ/Q values are determined through selective use of the following set of equations for ground level relative concentrations at the plume centerline:

$$\chi/Q = \frac{1}{\overline{U}_{10}\left(\pi\sigma_y\sigma_z + A/2\right)} \qquad 2.3\text{-}4$$

$$\chi/Q = \frac{1}{\overline{U}_{10}\left(3\pi\sigma_y\sigma_z\right)} \qquad 2.3\text{-}5$$

$$\chi/Q = \frac{1}{\overline{U}_{10}\pi\Sigma_y\sigma_z} \qquad 2.3\text{-}6$$

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 m, a function of atmospheric stability and distance

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

Σ_y is lateral spread with meander and building wake effects, in m, a function of atmospheric stability, wind speed $\overline{U}_{10}$, and distance [for distances of 800 meters or less, $\Sigma_y = M\sigma_y$, where M is a function of the atmospheric stability and wind speed; for distances greater than 800 meters, $\Sigma_y = (M - 1)\ \sigma_{y800m} + \sigma_y$].

A is the smallest vertical-plane cross-sectional area of the reactor building in m^2.

χ/Q values are calculated using Equations 2.3-4, 2.3-5 and 2.3-6. The values from Equations 2.3-4 and 2.3-5 are compared and the higher values selected. This value is compared with the value from Equations 2.3-6 and the lower value of these two is selected as the appropriate χ/Q value.

The χ/Q value used in accident consequence analysis is selected from the maximum sector χ/Q values which is exceeded 0.5% of the time.

2.3.5 LONG-TERM (ROUTINE) DIFFUSION ESTIMATES

The long-term diffusion characteristics for the Susquehanna SSES were estimated in accordance with the criteria set forth in NRC Regulatory Guide 1.111 (1977). The analysis was performed using the onsite meteorological data recorded at the primary tower for January 1999 through December 2003.

2.3.5.1 Atmospheric Diffusion Models

2.3.5.1.1 Straight Line Airflow Model

A ground level release model based on meteorological data and plant parameters was used to calculate the annual average atmospheric relative concentration (χ/Q) values. Depletion factors are computed directly from depletion curves as the relative deposition rates. For long-term, ground level relative concentrations, the plume is assumed to diffuse evenly over a 22.5-degree sector.

The hourly relative concentration values are calculated in the sector defined by the wind direction using the following equation:

$$\chi/\mathrm{Q} = \frac{2.032}{\sigma_z \bar{u} x} \qquad (5)$$

Where

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

σ_z = vertical standard deviation of the plume (m)

$\bar{u}$ = average wind speed (m/sec)

x = distance from the source (m)

However, with consideration of the turbulent wake effect, Equation 5 is revised as follows:

$$\chi/\mathrm{Q} = \frac{2.032}{\bar{u} x\sqrt{\sigma_z^{\,2} + cV^2/\pi}} \qquad (6)$$

Where

c = building shape factor

V = vertical height of the highest adjacent building

The wake factor (cV^2/π) is limited, close to the source, to a factor of $2\sigma_z^2$.

If $\sqrt{3} < \sigma_z < \sqrt{\sigma_z^2 + c\frac{V^2}{\pi}}$, the equation is

$$\chi/Q = \frac{2.032}{\sqrt{3}\sigma_z \bar{u} x} \qquad (7)$$

(i.e., χ/Q is calculated to be the larger of Equations 6 and 7). The total relative concentration at each sector and distance is then divided by the total number of hours in the database.

2.3.5.1.2 Terrain/Recirculation Correction Factors

The straight-line trajectory, Gaussian diffusion model assumes that a constant mean wind transports and diffuses plume effluents in the direction of airflow at the release point within the entire region of interest. In other words, the wind speed and atmospheric stability at the release point are assumed to determine the atmospheric dispersion characteristics in the direction of the mean wind at all distances. In areas of more complex terrain recirculation of the plume over longer time periods may occur. To account for this effect the results of a comparison of the PAM (Puff Advection Model) with the straight line model was made from which adjustment factors for the site region were determined. These correction factors were applied to the results of the straight line model by multiplying the χ/Q values by the correction factors found in Table 2.3-106.

2.3.5.1.3 Deposition and Depleted X/Q's

As radioactive effluent in a plume travels downwind, it is subject to several removal mechanisms, including radioactive decay, dry deposition, and wet deposition (during precipitation). Corrections for radioactive decay of 2.26 days for undepleted χ/Q and 8 days of depleted χ/Q are shown in the dispersion estimates reported in this subsection.

Dry deposition, which results in depletion of halogen and particulate isotopes from the plume, is calculated using Figures 2 through 5 in Regulatory Guide 1.111. Depletion factors in these curves are a function of release height and distance. All releases at the SSES are at ground level. Therefore, elevated curves were not used. Each χ/Q is multiplied by the depletion correction factor to estimate the depleted χ/Q value.

To determine relative deposition rate as a function of distance and stability, the curves given in Regulatory Guide 1.111 are used in a computerized table look-up routine. Values from the curves are divided by the sector cross-width (arc) at the point of calculation to give units m^{-2}.

2.3.5.1.4 Results of Long-Term Diffusion Estimates

Tables 2.3-107 through 2.3-118 present the annual and five year average χ/Q, decayed and depleted χ/Q and deposition values at the site boundary and exclusion area boundary for each of

the 16 cardinal directions. Tables 2.3-119 through 2.3-136 show similar information for the nearest residence, vegetable garden, meat and dairy animal location and selected special receptor locations around the plant. Tables 2.3-137 through 2.3-140 present the five year average χ/Q, decayed and depleted χ/Q and deposition values for the sixteen directions out to a distance of 80.5 kilometers (50 miles) from the plant.

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TABLE 2.3-1

HURRICANES WITHIN 75 AND 150 NAUTICAL MILES
OF THE SUSQUEHANNA SITE
PERIOD OF RECORD 1871 to 1969

	TRACKS WITHIN 75 NM*	TRACKS WITHIN 150 NM*	TOTAL NORTH ATLANTIC STORMS
TIME PERIOD			
Prior to 1900	8	18	
After 1900	0	2	
1871 TO 1969	8	20	489
Occurrence by Month			
June	0	1	
July	0	0	
August	2	3	
September	4	10	
October	2	6	
Totals	8	20	

*NM represents nautical miles.

TABLE 2.3-2

THUNDERSTORM DAYS FOR AVOCA, PENNSYLVANIA

WILKES-BARRE SCRANTON AIRPORT

PERIOD OF RECORD 1956 TO 1974

VALUES ARE EXPRESSED IN DAYS (Ref. 2.3-3)

MONTH	THUNDERSTORM DAYS (to the nearest whole day)
January	*
February	*
March	1
April	2
May	4
June	6
July	8
August	5
September	3
October	1
November	*
December	*
Annual Average	30

*Less than one-half

TABLE 2.3-3

TOTAL NUMBER OF DAYS IN 5 YEARS

MIXING HEIGHTS < 1500m

WIND SPEEDS < 4.0 sec^{-1} and

NO SIGNIFICANT PRECIPITATION

FOR EPISODES LASTING AT LEAST 2 DAYS

Station	Episodes	Episode-days	Season of Greatest # of Episode Days
Pittsburgh, PA	16	39	Autumn
New York, N.Y.	4	9	Autumn
Albany, N.Y.	7	23	Autumn

TABLE 2.3-5

MEAN MONTHLY VALUES: SUSQUEHANNA SITE (1973-1976)

Month	Wind Speed (m/sec)	Dry Bulb (C)	Wet Bulb (C)
January	2.3	-2.1	-3.6
February	2.0	-1.4	-3.4
March	2.7	3.6	1.1
April	2.8	8.8	5.2
May	2.0	13.8	10.6
June	1.7	18.8	16.1
July	1.5	20.3	17.7
August	1.4	20.0	17.7
September	1.6	15.0	12.9
October	1.9	9.9	7.6
November	2.1	4.9	2.6
December	2.1	-0.9	-2.6
Annual	2.0	9.3	6.9

TABLE 2.3-6

LONG-TERM AVERAGE WIND SPEED AND PREVAILING DIRECTION AT WILKES-BARRE SCRANTON AIRPORT

Period of Record: 1956-1974

Month	Average Speed (mph)	Prevailing Direction
January	8.9	SW
February	9.3	SW
March	9.3	NW
April	9.6	SW
May	8.8	WSW
June	7.9	SW
July	7.4	WSW
August	7.2	SW
September	7.4	SW
October	7.9	WSW
November	8.7	WSW
December	8.9	SW
Annual	8.4	SW

TABLE 2.3-7
AVERAGE WIND SPEED AND PREVAILING
DIRECTION AT THE SUSQUEHANNA SITE

Period of Record (1973-1976)

Month	Average Speed (mph)	Prevailing Direction
January	5.1	WSW
February	4.5	SSW
March	6.0	W
April	6.3	W
May	4.5	W and E
June	3.8	WSW
July	3.4	WSW
August	3.1	ENE
September	3.6	ENE
October	4.3	E
November	4.7	W
December	4.7	W
Annual	4.5	WSW

TABLE 2.3-8

WIND DIRECTION PERSISTENCE - PASQUILL A (1973 - 1976)

CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	33	5	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NE	34	12	5	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
ENE	17	4	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
E	19	2	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
ESE	10	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SE	11	5	0	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSE	12	6	2	4	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
S	46	12	8	2	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSW	49	15	11	4	3	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SW	80	26	10	9	1	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
WSW	123	55	20	13	4	4	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
W	102	69	29	17	10	3	3	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
WNW	33	16	8	4	3	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NW	17	6	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NNW	16	9	2	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
N	16	5	7	1	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

AVERAGE WIND SPEED (M/SEC)
CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	2.64	3.15	2.56	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NE	2.38	2.58	2.87	3.09	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ENE	2.41	2.86	2.88	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
E	2.51	1.85	3.62	0.	0.	3.44	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ESE	2.29	2.65	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SE	1.60	2.29	0.	2.09	2.98	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSE	2.28	2.02	3.28	1.80	2.23	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
S	2.73	2.94	3.44	4.88	3.75	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSW	2.66	2.71	3.84	4.02	5.01	0.	6.76	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SW	2.37	2.65	2.58	2.89	3.29	2.00	3.64	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WSW	2.94	2.97	3.20	3.12	3.44	4.22	2.31	0.	2.45	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
W	3.51	3.61	3.87	4.34	3.63	5.20	3.88	6.33	3.04	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WNW	4.33	4.83	4.80	5.43	4.81	0.	0.	4.99	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NW	3.23	4.59	4.38	5.62	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NNW	3.59	4.37	2.53	4.68	6.45	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
N	3.28	3.64	3.26	3.12	3.66	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

TOTAL NO. OF OBSERVATIONS = 35064

TABLE 2.3-9

WIND DIRECTION PERSISTENCE - PASQUILL B (1973 - 1976)

CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	12	4	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NE	14	6	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
ENE	7	4	4	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
E	12	1	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
ESE	8	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SE	6	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSE	7	2	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
S	21	5	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSW	20	1	0	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SW	21	3	2	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
WSW	33	8	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
W	36	12	6	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
WNW	11	5	1	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NW	15	5	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NNW	14	6	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
N	8	3	3	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

AVERAGE WIND SPEED (M/SEC)
CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	2.51	2.81	1.30	3.58	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NE	2.72	2.20	2.19	3.54	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ENE	1.65	2.34	3.29	0.	0.	3.28	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
E	2.69	1.16	0.	2.06	0.	6.58	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ESE	2.05	2.50	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SE	1.68	2.17	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSE	2.06	1.16	1.48	1.74	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
S	2.47	2.91	1.97	3.59	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSW	2.82	7.00	0.	6.52	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SW	3.27	3.06	4.70	6.69	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WSW	3.47	3.64	1.78	5.78	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
W	4.53	5.09	3.62	2.27	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WNW	3.90	4.70	4.43	5.96	0.	5.14	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NW	3.70	2.63	4.25	1.79	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NNW	3.23	3.14	3.23	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
N	2.89	2.55	3.35	2.40	0.	3.61	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

TOTAL NO. OF OBSERVATIONS = 35064

TABLE 2.3-10

WIND DIRECTION PERSISTENCE - PASQUILL C (1973 - 1976)

CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	6	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NE	7	0	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
ENE	4	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
E	5	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
ESE	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SE	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSE	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
S	14	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSW	10	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SW	13	5	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
WSW	15	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
W	11	3	1	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
WNW	3	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NW	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NNW	7	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
N	10	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

AVERAGE WIND SPEED (M/SEC)
CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	2.05	2.49	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NE	2.62	0.	3.81	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ENE	3.38	2.44	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
E	3.05	7.30	0.	0.	4.93	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ESE	2.23	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SE	1.54	1.03	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSE	1.39	1.48	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
S	3.75	3.83	0.	0.	5.95	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSW	3.63	0.	2.75	0.	0.	0.	0.	4.63	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SW	3.35	5.17	0.	3.67	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WSW	3.75	4.05	4.02	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
W	4.09	4.89	1.60	5.59	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WNW	2.23	5.45	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NW	3.46	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NNW	3.45	1.85	4.69	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
N	3.14	4.26	3.11	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

TOTAL NO. OF OBSERVATIONS = 35064

TABLE 2.3-11

WIND DIRECTION PERSISTENCE - PASQUILL D (1973 - 1976)

CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	79	25	4	3	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NE	90	27	17	4	2	2	0	1	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
ENE	60	17	5	4	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
E	53	10	6	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
ESE	33	10	6	0	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SE	24	4	0	2	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSE	33	16	4	1	2	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
S	62	24	7	7	4	4	2	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSW	74	30	4	2	0	3	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SW	101	32	8	3	0	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
WSW	134	60	23	9	7	2	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
W	102	41	26	21	13	2	1	1	2	0	0	1	0	1	0	0	0	0	0	0	0	0	0	0
WNW	68	26	15	8	3	4	2	0	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NW	63	29	7	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NNW	45	21	5	3	1	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0
N	47	22	6	1	2	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

AVERAGE WIND SPEED (M/SEC)
CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	1.98	2.77	2.37	3.15	4.38	0.	2.81	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NE	1.98	2.32	2.13	2.65	2.66	3.45	0.	2.73	2.32	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ENE	2.10	2.33	1.62	2.66	0.	4.00	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
E	1.79	1.82	2.28	2.52	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ESE	1.55	2.30	1.81	0.	2.89	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SE	1.81	1.48	0.	3.75	2.27	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSE	1.48	1.98	2.62	1.98	1.85	0.	0.	2.67	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
S	2.10	2.40	2.57	1.89	2.08	2.39	3.03	0.	3.79	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSW	2.05	1.75	1.22	5.27	0.	2.43	0.	4.54	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SW	1.79	2.37	1.77	1.15	0.	2.35	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WSW	2.49	2.73	3.03	2.92	2.82	2.17	4.08	3.75	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
W	3.19	3.56	3.96	4.20	3.11	4.58	5.59	5.50	4.71	0.	0.	4.25	0.	3.29	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WNW	3.71	3.60	4.01	3.87	5.39	5.41	4.64	0.	5.20	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NW	3.06	3.05	3.83	3.66	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NNW	2.77	2.92	3.93	3.40	6.47	0.	0.	0.	0.	0.	0.	0.	0.	0.	4.73	0.	0.	0.	0.	0.	0.	0.	0.	0.
N	2.33	3.22	2.87	1.98	3.84	0.	0.	6.83	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

TOTAL NO. OF OBSERVATIONS = 35064

TABLE 2.3-12

WIND DIRECTION PERSISTENCE - PASQUILL E (1973 - 1976)

CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	47	20	4	2	1	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NE	98	21	4	4	3	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0
ENE	103	31	21	7	0	0	2	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0
E	89	33	8	11	4	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
ESE	38	5	5	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SE	23	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSE	19	9	1	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
S	34	13	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSW	44	8	8	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SW	61	28	10	4	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
WSW	64	33	12	10	3	3	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
W	61	22	11	2	1	2	0	1	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0
WNW	32	10	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NW	22	4	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NNW	25	2	3	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
N	37	19	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

AVERAGE WIND SPEED (M/SEC)
CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	1.55	1.56	2.39	1.59	2.13	1.95	0.	1.70	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NE	1.47	1.39	1.62	2.26	1.57	0.	0.	0.	0.	0.	0.	0.	1.89	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ENE	1.34	1.03	1.46	1.12	0.	0.	.81	0.	1.02	0.	0.	0.	0.	.98	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
E	1.11	1.12	1.30	1.00	1.12	3.47	0.	1.04	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ESE	1.13	.83	1.52	1.45	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SE	1.77	.91	.89	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSE	1.72	2.05	2.97	1.86	2.38	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
S	1.58	1.63	2.37	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSW	1.28	1.40	1.52	1.11	2.21	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SW	1.47	1.35	1.70	1.31	0.	2.09	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WSW	1.93	2.29	2.47	2.37	2.71	2.69	3.65	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
W	2.59	2.57	3.59	3.59	3.40	2.50	0.	5.62	0.	0.	0.	0.	0.	5.11	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WNW	2.22	2.06	2.76	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NW	2.37	1.80	2.74	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NNW	1.59	2.66	2.34	3.03	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
N	1.72	1.49	1.62	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

TOTAL NO. OF OBSERVATIONS = 35064

TABLE 2.3-13

WIND DIRECTION PERSISTENCE - PASQUILL F (1973 - 1976)

CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	14	6	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NE	36	5	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
ENE	89	28	13	11	7	1	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0
E	85	31	14	4	6	5	3	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
ESE	20	4	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SE	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSE	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
S	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSW	2	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SW	6	2	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
WSW	5	5	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
W	9	1	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
WNW	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NW	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NNW	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
N	12	5	2	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

AVERAGE WIND SPEED (M/SEC)
CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	.88	.97	1.01	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NE	.91	.72	.54	.98	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ENE	.97	1.13	1.20	1.40	1.05	1.97	1.56	0.	0.	0.	0.	1.34	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
E	1.06	1.25	1.00	1.24	1.17	1.25	.60	.92	1.62	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ESE	.92	1.18	1.13	.93	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SE	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSE	.71	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
S	.86	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSW	1.38	.70	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SW	1.12	.53	0.	0.	0.	2.01	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WSW	1.98	.69	.93	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
W	.87	.64	.76	.84	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WNW	.79	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NW	.85	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NNW	.95	.89	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
N	1.14	1.53	.89	1.14	1.36	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

TOTAL NO. OF OBSERVATIONS = 35064

TABLE 2.3-14

WIND DIRECTION PERSISTENCE - PASQUILL G (1973 - 1976)

CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	5	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NE	28	9	3	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
ENE	50	18	12	1	4	1	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
E	36	20	13	7	0	2	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
ESE	9	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SE	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSE	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
S	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSW	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SW	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
WSW	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
W	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
WNW	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NW	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NNW	4	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
N	6	3	0	1	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

AVERAGE WIND SPEED (M/SEC)
CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	.98	1.07	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NE	1.04	1.05	1.15	.73	0.	0.	1.01	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ENE	1.03	1.02	1.22	1.47	1.33	1.67	1.55	1.76	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
E	1.02	1.01	1.22	1.23	0.	2.56	1.23	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ESE	.99	.81	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SE	.69	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSE	0.	.63	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
S	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSW	.69	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SW	.94	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WSW	.76	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
W	1.08	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WNW	.85	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NW	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NNW	.92	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
N	1.47	.95	0.	1.48	1.40	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

TOTAL NO. OF OBSERVATIONS = 35064

TABLE 2.3-15

WIND DIRECTION PERSISTENCE - PASQUILL ALL (1973 - 1976)

CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	211	81	26	15	4	3	3	2	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0
NE	315	107	60	22	11	7	4	1	3	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0
ENE	328	128	79	34	15	9	9	4	5	1	1	0	2	2	0	0	0	0	0	0	0	0	0	0
E	271	99	63	35	21	17	9	9	4	2	2	0	1	0	0	0	0	0	0	0	0	0	0	0
ESE	140	44	15	9	3	2	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SE	90	17	7	4	1	2	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSE	90	38	13	11	7	0	1	1	1	1	0	0	1	0	0	0	0	0	0	0	0	0	0	0
S	158	66	29	14	14	6	3	5	1	4	1	1	0	1	1	0	0	0	0	1	0	0	0	0
SSW	184	87	33	11	9	7	3	1	3	2	2	0	0	1	0	0	0	0	0	0	0	0	0	0
SW	305	114	54	20	14	12	7	1	2	1	0	0	0	0	1	0	0	0	0	0	0	0	0	0
WSW	373	166	93	56	23	13	16	3	7	1	1	1	2	0	0	0	1	0	0	0	0	0	0	0
W	309	129	93	63	24	25	20	9	5	4	4	2	0	2	0	0	1	0	0	0	1	0	0	0
WNW	163	65	34	14	11	7	8	2	3	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0
NW	128	58	20	8	4	0	0	0	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NNW	123	46	21	13	10	0	2	0	0	0	0	0	0	0	0	0	1	1	0	1	0	0	0	0
N	155	68	30	16	8	5	2	1	1	0	1	0	0	0	0	1	0	0	0	0	0	0	0	0

AVERAGE WIND SPEED (M/SEC)
CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	1.77	1.99	2.29	2.31	3.79	3.05	2.06	1.48	0.	0.	0.	4.45	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NE	1.54	1.80	2.01	2.25	1.99	2.34	1.80	2.73	2.74	0.	4.11	0.	1.89	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ENE	1.40	1.34	1.54	1.43	1.18	2.13	1.50	1.05	1.48	1.01	1.61	0.	1.44	1.08	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
E	1.37	1.21	1.26	1.17	1.32	1.47	1.94	2.06	1.27	2.72	3.15	0.	1.49	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ESE	1.43	1.56	1.37	1.57	2.61	1.26	0.	0.	2.90	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SE	1.63	1.55	1.88	3.20	1.67	1.85	0.	0.	2.67	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSE	1.63	1.68	1.84	2.00	2.06	0.	1.46	2.67	2.68	3.95	0.	0.	1.95	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
S	1.71	2.13	2.64	2.27	2.74	2.10	2.61	2.91	6.24	3.32	4.65	3.99	0.	5.26	4.06	0.	0.	0.	0.	4.89	0.	0.	0.	0.
SSW	1.93	2.11	2.51	2.45	3.18	4.02	1.91	4.54	5.16	4.05	4.09	0.	0.	4.94	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SW	1.91	1.99	1.94	2.26	2.48	3.23	1.72	3.41	2.22	8.48	0.	0.	0.	0.	5.11	0.	0.	0.	0.	0.	0.	0.	0.	0.
WSW	2.43	2.47	2.77	2.86	3.19	3.21	3.68	3.76	3.29	4.03	3.36	2.35	2.81	0.	0.	0.	3.69	0.	0.	0.	0.	0.	0.	0.
W	2.79	3.06	3.45	3.65	3.84	3.57	3.78	4.97	4.94	5.18	4.50	5.23	0.	4.20	0.	0.	4.44	0.	0.	0.	5.58	0.	0.	0.
WNW	2.90	3.51	3.67	3.97	4.34	5.51	4.59	5.10	5.20	0.	0.	0.	0.	5.78	5.45	0.	0.	0.	0.	0.	0.	0.	0.	0.
NW	2.70	3.08	3.51	4.07	3.05	0.	0.	0.	3.77	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NNW	2.41	3.07	3.22	3.41	3.93	0.	2.62	0.	0.	0.	0.	0.	0.	0.	0.	0.	1.86	4.72	0.	.24	0.	0.	0.	0.
N	1.97	2.28	2.82	2.38	2.60	2.42	1.73	1.28	4.24	0.	4.21	0.	0.	0.	0.	6.24	0.	0.	0.	0.	0.	0.	0.	0.

TOTAL NO. OF OBSERVATIONS = 35064

TABLE 2.3-16

WIND DIRECTION PERSISTENCE - PASQUILL E, F, & G (1973 - 1976)

CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	77	29	7	4	0	2	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NE	183	46	17	7	4	2	2	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0
ENE	235	94	54	21	15	6	10	2	6	0	1	0	2	2	0	0	0	0	0	0	0	0	0	0
E	177	78	46	30	17	14	7	6	4	2	0	0	1	0	0	0	0	0	0	0	0	0	0	0
ESE	74	16	5	5	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SE	35	5	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSE	26	13	4	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
S	44	12	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SSW	48	13	9	3	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
SW	80	31	11	5	0	1	1	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
WSW	81	39	17	16	3	4	3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
W	86	29	12	2	1	2	2	1	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0
WNW	49	12	5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NW	38	4	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NNW	41	6	3	2	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
N	63	30	8	1	2	3	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

AVERAGE WIND SPEED (M/SEC)
CONSECUTIVE HOURS

SECTOR	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	>24
NNE	1.26	1.45	2.03	1.60	0.	2.02	0.	1.70	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NE	1.26	1.17	1.48	1.69	1.44	1.06	.99	0.	0.	0.	0.	0.	1.89	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ENE	1.13	1.04	1.21	1.10	1.12	1.06	1.43	.99	1.40	0.	1.61	0.	1.44	1.08	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
E	1.05	1.06	1.03	1.08	1.21	1.33	1.26	1.07	1.99	1.28	0.	0.	1.49	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
ESE	1.06	.89	1.52	1.17	.92	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SE	1.57	.90	.89	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSE	1.55	2.02	1.63	1.86	2.38	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
S	1.39	1.73	1.46	2.94	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SSW	1.22	1.23	1.61	1.11	2.21	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
SW	1.46	1.35	1.63	1.46	0.	2.09	.92	2.03	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WSW	1.93	2.10	2.27	2.33	2.71	3.20	3.65	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
W	2.22	2.38	3.53	3.59	3.40	2.50	.77	5.62	0.	0.	0.	0.	0.	5.11	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
WNW	1.80	2.06	2.81	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NW	1.94	1.80	2.74	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
NNW	1.57	1.45	2.34	3.03	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
N	1.53	1.47	1.73	1.31	1.36	1.19	0.	1.28	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

TOTAL NO. OF OBSERVATIONS = 35064

TABLE 2.3-17

LONG TERM TEMPERATURE (°F) AT
WILKES-BARRE SCRANTON AIRPORT

1	2	3	4	5	6
	Averages			Extreme	
Month	Daily Max.	Daily Min.	Mean	Highest	Lowest
January	32.3	17.7	25.0	67	-21
February	34.7	19.0	26.9	71	-16
March	45.4	28.1	36.8	83	-4
April	58.1	38.1	48.1	92	+14
May	69.6	48.3	58.9	91	+27
June	77.8	56.8	67.3	97	+34
July	82.2	61.8	72.0	101	+43
August	80.1	60.2	70.1	95	+38
September	72.2	52.6	62.4	95	+30
October	61.1	41.9	51.5	83	+19
November	48.8	33.8	41.3	80	+9
December	36.8	23.5	30.2	67	-9
Annual	58.3	40.2	49.2	101	-21

Temperature averages are based on a 36-year climatological period from 1961-1996.
Temperature extremes are for the 43 year period from 1954-1995.

Sources: Pennsylvania State Climatologist (Ref. 2.3-5a)
National Oceanic and Atmospheric Administration Cooperative Institute for Research in Environmental Sciences, Climate Diagnostic Center (Ref. 2.3-3a) (NOAA/CIRES/CDC)

TABLE 2.3-18

MEAN MONTHLY TEMPERATURE, DEW POINT*
TEMPERATURE, AND RELATIVE HUMIDITY
WILKES-BARRE SCRANTON AIRPORT (Ref. 2.3-3)

Period of Records: 1956-1974

Month	Temperature °F	Dew Point °F	RH (%)
January	26	18	70
February	27	18	69
March	36	26	67
April	49	37	62
May	59	47	64
June	68	58	70
July	72	62	70
August	70	61	73
September	63	55	76
October	53	44	72
November	41	33	72
December	29	21	73
Annual	49	40	70

*Dew point temperatures computed from temperature and relative humidity measurements.

TABLE 2.3-19

TEMPERATURE AND MOISTURE DATA
FOR THE SUSQUEHANNA SITE

Period of Record: 1973-1976

Month	Dry Bulb (°C)			Average Wet Bulb (°C)	R.H. %
	Average	Max	Min		
January	-2.1	18.5	-20.9	-3.6	66
February	-1.4	18.7	-18.5	-3.4	61
March	3.6	22.2	-11.2	1.1	62
April	8.8	32.5	-6.1	5.2	60
May	13.8	30.6	-1.9	10.6	70
June	18.8	31.9	5.6	16.1	76
July	20.3	31.7	7.8	17.7	79
August	20.0	34.3	4.8	17.7	82
September	15.0	31.8	-0.8	12.9	81
October	9.9	27.8	-6.4	7.6	72
November	4.9	23.1	-14.0	2.6	67
December	-0.9	17.8	-19.0	-2.6	66
Annual	9.3	34.3	-20.9	6.9	70

TABLE 2.3-20
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: JANUARY 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
	9.61	9.61	9.61
HOUR	PCT	DEG C	DEG C
1	67.9	-4.2	-2.8
2	68.5	-4.4	-3.1
3	68.8	-4.5	-3.2
4	68.9	-4.7	-3.4
5	68.7	-4.8	-3.6
6	68.4	-5.1	-3.8
7	68.8	-5.3	-4.1
8	69.2	-5.4	-4.2
9	69.6	-5.1	-4.0
10	68.8	-4.3	-3.0
11	66.3	-3.5	-2.0
12	63.6	-2.8	-1.1
13	61.7	-2.3	-.4
14	60.6	-1.9	.1
15	60.0	-1.6	.5
16	59.6	-1.6	.6
17	60.1	-1.9	.2
18	61.7	-2.3	-.4
19	63.6	-2.7	-1.0
20	64.8	-3.1	-1.5
21	65.5	-3.4	-1.9
22	65.7	-3.8	-2.2
23	65.9	-4.0	-2.6
24	66.6	-4.3	-2.9

TABLE 2.3-20
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: JANUARY 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
ABSOLUTE MAX	98.5	16.4	18.5
AVG DAILY MAX	78.3	-.2	1.9
MEAN	65.6	-3.6	-2.1
CLIMATIC MEAN	65.1	-3.7	-2.2
AVG DAILY MIN	52.0	-7.3	-6.2
ABSOLUTE MIN	3.2	-21.6	-20.9
STANDARD DEV	17.9	6.0	6.2
VALID OBS	2975	2975	2975
INVALID OBS	1	1	1
TOTAL OBS	2976	2976	2976
DATA RECOVERY	100.0	100.0	100.0

TABLE 2.3-21
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: FEBRUARY 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
	9.61	9.61	9.61
HOUR	PCT	DEG C	DEG C
1	64.8	-4.4	-2.9
2	65.2	-4.6	-3.1
3	65.4	-4.9	-3.4
4	66.3	-5.1	-3.7
5	66.5	-5.4	-4.0
6	66.8	-5.5	-4.2
7	66.8	-5.7	-4.4
8	67.4	-5.6	-4.4
9	67.3	-5.0	-3.6
10	64.0	-4.0	-2.4
11	60.0	-3.0	-1.0
12	57.4	-2.2	.2
13	54.7	-1.7	1.0
14	53.2	-1.2	1.7
15	53.1	-.9	2.1
16	52.6	-.8	2.2
17	53.0	-.9	2.0
18	54.4	-1.4	1.4
19	56.7	-2.0	.5
20	59.2	-2.5	-.3
21	61.2	-2.9	-.9
22	62.8	-3.2	-1.4
23	64.1	-3.6	-1.9
24	65.1	-3.9	-2.4

TABLE 2.3-21
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: FEBRUARY 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
ABSOLUTE MAX	100.0	14.2	18.7
AVG DAILY MAX	76.9	.3	3.2
MEAN	61.2	-3.4	-1.4
CLIMATIC MEAN	61.5	-3.6	-1.5
AVG DAILY MIN	46.0	-7.4	-6.2
ABSOLUTE MIN	13.7	-18.9	-18.5
STANDARD DEV	17.0	5.9	6.5
VALID OBS	2712	2712	2712
INVALID OBS	0	0	0
TOTAL OBS	2712	2712	2712
DATA RECOVERY	100.0	100.0	100.0

TABLE 2.3-22
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: MARCH 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
	9.61	9.61	9.61
HOUR	PCT	DEG C	DEG C
1	66.1	-.1	1.9
2	66.7	-.3	1.6
3	67.4	-.5	1.3
4	67.9	-.7	1.1
5	68.3	-.9	.9
6	68.6	-1.0	.7
7	69.2	-1.0	.6
8	69.3	-.7	.9
9	66.9	-.1	1.8
10	63.4	.8	3.0
11	60.5	1.5	4.1
12	57.8	2.2	5.2
13	55.9	2.7	6.0
14	54.7	3.1	6.6
15	54.2	3.5	7.2
16	54.3	3.6	7.3
17	54.0	3.5	7.2
18	54.8	3.1	6.6
19	56.7	2.5	5.8
20	59.1	1.9	4.8
21	60.9	1.4	4.0
22	62.3	.9	3.3
23	63.9	.5	2.8
24	65.1	.2	2.3

TABLE 2.3-22
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: MARCH 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
ABSOLUTE MAX	100.0	20.8	22.2
AVG DAILY MAX	77.7	4.6	8.3
MEAN	62.0	1.1	3.6
CLIMATIC MEAN	62.3	1.0	3.6
AVG DAILY MIN	47.0	-2.6	-1.0
ABSOLUTE MIN	18.8	-12.2	-11.2
STANDARD DEV	18.1	5.2	5.8
VALID OBS	2925	2925	2925
INVALID OBS	51	51	51
TOTAL OBS	2976	2976	2976
DATA RECOVERY	98.3	98.3	98.3

TABLE 2.3-23
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: APRIL 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
	9.61	9.61	9.61
HOUR	PCT	DEG C	DEG C
1	66.7	3.7	6.1
2	68.1	3.3	5.6
3	69.5	3.0	5.1
4	70.8	2.7	4.7
5	71.9	2.4	4.2
6	72.8	2.2	4.0
7	72.5	2.5	4.4
8	69.6	3.6	5.8
9	63.9	4.7	7.5
10	58.7	5.4	9.0
11	55.6	6.2	10.3
12	52.9	6.7	11.3
13	51.4	7.2	12.1
14	49.9	7.6	12.7
15	49.4	7.8	13.1
16	49.0	7.9	13.3
17	49.1	7.8	13.2
18	49.5	7.5	12.7
19	50.9	7.0	11.8
20	53.9	6.4	10.7
21	56.8	5.8	9.6
22	59.8	5.2	8.5
23	62.2	4.7	7.7
24	64.7	4.2	6.9

TABLE 2.3-23
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: APRIL 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
ABSOLUTE MAX	97.9	20.0	32.5
AVG DAILY MAX	77.9	8.7	14.2
MEAN	60.0	5.2	8.8
CLIMATIC MEAN	60.7	4.9	8.6
AVG DAILY MIN	43.6	1.0	2.9
ABSOLUTE MIN	10.0	-7.8	-6.1
STANDARD DEV	18.6	5.8	7.1
VALID OBS	2878	2878	2878
INVALID OBS	2	2	2
TOTAL OBS	2880	2880	2880
DATA RECOVERY	99.9	99.9	99.9

TABLE 2.3-24
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: MAY 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
	9.61	9.61	9.61
HOUR	PCT	DEG C	DEG C
1	76.5	9.1	11.2
2	77.6	8.7	10.6
3	78.4	8.3	10.2
4	79.4	8.1	9.8
5	80.0	7.8	9.5
6	81.1	7.7	9.3
7	81.0	8.1	9.7
8	78.4	9.1	11.0
9	73.4	10.2	12.7
10	69.3	11.0	14.1
11	65.7	11.7	15.3
12	63.3	12.2	16.2
13	60.6	12.6	17.1
14	58.9	13.0	17.8
15	57.6	13.2	18.2
16	57.5	13.2	18.2
17	58.2	13.0	17.9
18	59.5	12.7	17.4
19	61.3	12.4	16.7
20	63.7	11.8	15.7
21	67.2	11.3	14.6
22	70.7	10.7	13.5
23	73.4	10.1	12.6
24	75.2	9.7	11.9

TABLE 2.3-24
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: MAY 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

ABSOLUTE MAX	100.0	30.6	30.6
AVG DAILY MAX	85.8	14.0	19.2
MEAN	69.5	10.6	13.8
CLIMATIC MEAN	69.0	10.3	13.7
AVG DAILY MIN	52.1	6.6	8.1
ABSOLUTE MIN	20.7	-3.3	-1.9
STANDARD DEV	18.1	5.1	5.9
VALID OBS	2964	2964	2964
INVALID OBS	12	12	12
TOTAL OBS	2976	2976	2976
DATA RECOVERY	99.6	99.6	99.6

TABLE 2.3-25
STATISTICAL AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: JUNE 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
	9.61	9.61	9.61
HOUR	PCT	DEG C	DEG C
1	85.1	14.4	15.8
2	85.9	14.0	15.4
3	86.5	13.8	15.1
4	86.8	13.5	14.8
5	87.1	13.3	14.6
6	88.0	13.4	14.6
7	87.6	14.0	15.2
8	85.4	15.0	16.4
9	80.8	15.9	18.0
10	75.4	16.7	19.5
11	70.9	17.4	20.9
12	67.7	17.8	21.8
13	64.9	18.0	22.6
14	64.0	18.2	22.9
15	63.6	18.4	23.2
16	63.2	18.4	23.3
17	63.4	18.3	23.2
18	64.8	18.0	22.6
19	67.5	17.6	21.7
20	71.3	17.0	20.5
21	76.6	16.4	19.0
22	80.4	15.7	17.9
23	83.0	15.3	17.0
24	84.0	14.8	16.4

TABLE 2.3-25
STATISTICAL AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: JUNE 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
ABSOLUTE MAX	100.0	27.2	31.9
AVG DAILY MAX	90.5	19.0	24.0
MEAN	76.4	16.1	18.8
CLIMATIC MEAN	74.7	15.8	18.9
AVG DAILY MIN	58.8	12.6	13.9
ABSOLUTE MIN	17.3	3.9	5.6
STANDARD DEV	15.8	4.0	4.7
VALID OBS	2876	2876	2876
INVALID OBS	4	4	4
TOTAL OBS	2880	2880	2880
DATA RECOVERY	99.9	99.9	99.9

TABLE 2.3-26
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: JULY 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

HOUR	REL HUMID	WET BULB	DRY BULB
	9.61	9.61	9.61
	PCT	DEG C	DEG C
1	88.9	15.9	17.1
2	89.5	15.6	16.6
3	89.6	15.3	16.3
4	89.7	15.0	16.0
5	90.2	14.7	15.7
6	90.8	14.7	15.7
7	90.8	15.3	16.2
8	88.6	16.3	17.5
9	84.0	17.4	19.2
10	77.6	18.4	21.0
11	72.9	19.0	22.4
12	69.4	19.5	23.4
13	66.7	19.8	24.2
14	64.3	20.0	24.8
15	63.3	20.1	25.2
16	63.2	20.2	25.3
17	64.0	20.1	25.0
18	66.1	19.9	24.5
19	69.6	19.7	23.7
20	74.9	19.0	22.1
21	81.0	18.1	20.3
22	84.4	17.5	19.2
23	86.3	16.9	18.4
24	87.6	16.4	17.7

TABLE 2.3-26
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: JULY 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
ABSOLUTE MAX	100.0	25.7	31.7
AVG DAILY MAX	94.0	20.8	26.0
MEAN	78.9	17.7	20.3
CLIMATIC MEAN	76.9	17.5	20.5
AVG DAILY MIN	59.9	14.1	15.1
ABSOLUTE MIN	30.9	6.9	7.8
STANDARD DEV	14.9	3.5	4.5
VALID OBS	2970	2970	2975
INVALID OBS	6	6	1
TOTAL OBS	2976	2976	2976
DATA RECOVERY	99.8	99.8	100.0

TABLE 2.3-27
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: AUGUST 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
	9.61	9.61	9.61
HOUR	PCT	DEG C	DEG C
1	91.4	16.2	17.1
2	91.7	15.8	16.7
3	92.0	15.5	16.3
4	92.5	15.3	16.1
5	92.5	15.1	15.9
6	92.9	15.0	15.7
7	93.2	15.2	15.9
8	92.4	16.1	16.8
9	88.2	17.2	18.5
10	81.6	18.3	20.4
11	75.6	19.2	22.1
12	70.5	19.7	23.5
13	67.3	20.0	24.4
14	65.5	20.3	24.9
15	65.0	20.3	25.0
16	65.2	20.2	24.9
17	66.3	20.0	24.6
18	68.9	19.8	23.9
19	73.6	19.4	22.7
20	80.5	18.7	20.9
21	85.3	17.9	19.6
22	87.8	17.4	18.7
23	89.5	16.8	17.9
24	90.7	16.5	17.4

TABLE 2.3-27
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: AUGUST 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
ABSOLUTE MAX	100.0	26.8	34.3
AVG DAILY MAX	95.0	20.8	25.8
MEAN	81.7	17.7	20.0
CLIMATIC MEAN	78.2	17.6	20.4
AVG DAILY MIN	61.4	14.3	15.1
ABSOLUTE MIN	28.1	3.9	4.8
STANDARD DEV	14.6	3.7	4.7
VALID OBS	2970	2970	2972
INVALID OBS	6	6	4
TOTAL OBS	2976	2976	2976
DATA RECOVERY	99.8	99.8	99.9

TABLE 2.3-28
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: SEPTEMBER 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
	9.61	9.61	9.61
HOUR	PCT	DEG C	DEG C
1	88.6	11.4	12.5
2	88.9	11.2	12.1
3	89.5	10.9	11.8
4	89.5	10.7	11.6
5	89.6	10.5	11.4
6	90.0	10.4	11.3
7	90.6	10.4	11.2
8	90.6	11.0	11.8
9	88.2	12.0	13.1
10	82.7	13.1	14.8
11	76.3	14.1	16.6
12	70.9	14.8	18.1
13	67.2	15.2	19.0
14	65.3	15.4	19.6
15	64.2	15.5	19.8
16	64.4	15.5	19.8
17	65.7	15.3	19.4
18	69.2	14.9	18.5
19	75.2	14.3	16.9
20	80.5	13.6	15.5
21	84.2	13.0	14.5
22	85.6	12.4	13.8
23	87.1	12.0	13.2
24	88.1	11.6	12.7

TABLE 2.3-28
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: SEPTEMBER 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
ABSOLUTE MAX	99.0	25.7	31.8
AVG DAILY MAX	93.3	16.3	20.5
MEAN	80.5	12.9	15.0
CLIMATIC MEAN	77.3	12.6	15.2
AVG DAILY MIN	61.3	9.0	9.8
ABSOLUTE MIN	28.4	-1.2	-.8
STANDARD DEV	14.9	4.6	5.3
VALID OBS	2875	2875	2875
INVALID OBS	5	5	5
TOTAL OBS	2880	2880	2880
DATA RECOVERY	99.8	99.8	99.8

TABLE 2.3-29
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: OCTOBER 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
	9.61	9.61	9.61
HOUR	PCT	DEG C	DEG C
1	79.6	6.2	7.7
2	80.2	5.9	7.2
3	80.5	5.6	6.9
4	80.9	5.3	6.5
5	80.9	5.1	6.3
6	81.3	4.9	6.1
7	82.0	4.8	5.8
8	82.6	5.2	6.3
9	80.5	6.3	7.7
10	75.0	7.7	9.6
11	69.1	8.7	11.4
12	64.7	9.5	12.9
13	61.9	9.9	13.8
14	59.7	10.4	14.6
15	58.5	10.5	14.9
16	58.3	10.5	15.0
17	59.5	10.2	14.4
18	62.3	9.6	13.3
19	66.5	8.9	11.9
20	70.8	8.2	10.7
21	73.9	7.7	9.8
22	75.7	7.2	9.0
23	76.9	6.8	8.6
24	78.6	6.4	8.0

TABLE 2.3-29
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: OCTOBER 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
ABSOLUTE MAX	100.0	20.7	27.8
AVG DAILY MAX	86.2	11.3	15.5
MEAN	72.4	7.6	9.9
CLIMATIC MEAN	70.8	7.5	10.1
AVG DAILY MIN	55.4	3.6	4.7
ABSOLUTE MIN	22.9	-7.9	-6.4
STANDARD DEV	17.4	5.6	6.0
VALID OBS	2706	2706	2975
INVALID OBS	270	270	1
TOTAL OBS	2976	2976	2976
DATA RECOVERY	90.9	90.9	100.0

TABLE 2.3-30
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: NOVEMBER 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
	9.61	9.61	9.61
HOUR	PCT	DEG C	DEG C
1	70.2	1.6	3.5
2	71.0	1.4	3.2
3	71.2	1.2	2.9
4	71.6	1.0	2.6
5	72.0	.8	2.4
6	72.2	.6	2.2
7	72.6	.5	2.0
8	73.4	.5	2.0
9	73.2	1.2	2.8
10	71.0	2.3	4.2
11	67.0	3.4	5.8
12	63.5	4.2	7.0
13	60.5	4.7	7.9
14	58.9	5.0	8.4
15	57.7	5.1	8.7
16	57.4	5.1	8.6
17	58.4	4.7	8.0
18	59.9	4.1	7.1
19	62.3	3.5	6.3
20	64.6	3.0	5.4
21	66.1	2.5	4.8
22	67.1	2.2	4.3
23	68.0	1.9	4.0
24	68.8	1.7	3.6

TABLE 2.3-30
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: NOVEMBER 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
ABSOLUTE MAX	100.0	19.4	23.1
AVG DAILY MAX	79.0	6.1	9.4
MEAN	66.6	2.6	4.9
CLIMATIC MEAN	65.9	2.5	4.9
AVG DAILY MIN	52.8	-1.1	.5
ABSOLUTE MIN	21.1	-15.4	-14.0
STANDARD DEV	17.7	6.0	6.3
VALID OBS	2873	2873	2880
INVALID OBS	7	7	7
TOTAL OBS	2880	2880	2880
DATA RECOVERY	99.8	99.8	100.0

TABLE 2.3-31
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: DECEMBER 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
	9.61	9.61	9.61
HOUR	PCT	DEG C	DEG C
1	68.4	-3.2	-1.8
2	68.8	-3.3	-1.9
3	68.9	-3.4	-2.1
4	68.8	-3.5	-2.2
5	68.7	-3.6	-2.3
6	68.8	-3.7	-2.4
7	68.9	-3.9	-2.6
8	68.5	-4.0	-2.7
9	68.5	-3.9	-2.5
10	68.3	-3.1	-1.6
11	66.3	-2.2	-.6
12	63.9	-1.6	.3
13	62.5	-1.1	1.0
14	60.7	-.9	1.4
15	59.6	-.6	1.8
16	59.4	-.8	1.6
17	59.7	-1.2	1.1
18	61.2	-1.6	.4
19	63.0	-2.0	-.1
20	64.9	-2.3	-.6
21	66.1	-2.6	-1.0
22	66.9	-2.8	-1.3
23	67.6	-3.0	-1.5
24	68.2	-3.2	-1.8

TABLE 2.3-31
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: DECEMBER 1973-1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
ABSOLUTE MAX	100.0	16.4	17.8
AVG DAILY MAX	79.2	.8	2.9
MEAN	65,7	-2.6	-.9
CLIMATIC MEAN	66.1	-2.7	-1.0
AVG DAILY MIN	52.9	-6.1	-4.8
ABSOLUTE MIN	7.9	-20.2	-19.0
STANDARD DEV	18.8	5.4	5.4
VALID OBS	2849	2849	2853
INVALID OBS	127	127	123
TOTAL OBS	2976	2976	2976
DATA RECOVERY	95.7	95.7	95.9

TABLE 2.3-32
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: JANUARY 1973 - DECEMBER 1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
	9.61	9.61	9.61
HOUR	PCT	DEG C	DEG C
1	76.3	5.6	7.2
2	76.9	5.4	6.8
3	77.4	5.1	6.5
4	77.8	4.9	6.2
5	78.1	4.7	6.0
6	78.5	4.5	5.8
7	78.7	4.7	5.9
8	78.0	5.2	6.5
9	75.4	6.0	7.7
10	71.3	6.9	9.2
11	67.2	7.8	10.5
12	63.8	8.4	11.6
13	61.3	8.9	12.5
14	59.7	9.2	13.1
15	58.9	9.4	13.4
16	58.7	9.4	13.4
17	59.3	9.2	13.1
18	61.1	8.8	12.4
19	64.0	8.3	11.5
20	67.4	7.7	10.4
21	70.5	7.2	9.5
22	72.5	6.7	8.7
23	74.1	6.3	8.1
24	75.3	5.9	7.6

TABLE 2.3-32
STATISTICS AND DIURNAL VARIATION OF METEOROLOGICAL PARAMETERS
DATA PERIOD: JANUARY 1973 - DECEMBER 1976
METEOROLOGICAL PARAMETERS (HEIGHTS IN METERS)

	REL HUMID	WET BULB	DRY BULB
ABSOLUTE MAX	100.0	30.6	34.3
AVG DAILY MAX	84.5	10.3	14.3
MEAN	70.1	6.9	9.3
CLIMATIC MEAN	69.1	6.7	9.4
AVG DAILY MIN	53.6	3.1	4.4
ABSOLUTE MIN	3.2	-21.6	-20.9
STANDARD DEV	18.6	9.4	9.9
VALID OBS	34573	34573	34860
INVALID OBS	491	491	204
TOTAL OBS	35064	35064	35064
DATA RECOVERY	98.6	98.6	99.4

TABLE 2.3-33

LONG TERM MONTHLY PRECIPITATION DATA (LIQUID EQUIVALENT, IN INCHES) FOR THE WILKES-BARRE SCRANTON AIRPORT AT AVOCA, PA.

MONTH	MEAN	GREATEST 24-HOUR
January	2.23	1.89
February	2.01	3.11
March	2.60	3.02
April	3.15	3.80
May	3.50	2.58
June	3.70	3.61
July	3.70	2.45
August	3.27	3.18
September	3.40	6.52
October	2.89	3.27
November	3.20	2.91
December	2.58	2.86
Annual	36.23	

Precipitation means are based on a 36 year climatological period from 1961-1996. Greatest 24-hour rainfall amounts are for a 38 year period from 1953-1990.

Sources: National Oceanic Atmospheric Administration, Cooperative Institute for Research in Environmental Science, Climate Diagnostic Center (NO44/CIRES/CDC) Ref. 2.3-3a
National Oceanic and Atmospheric Administration, Local Climatological Data, Annual Summary with Comparitive Data, Avoca, PA (Ref. 2.3-3)

TABLE 2.3-34

EXPECTED RAINFALL BY DURATION AND RECURRENCE

INTERVAL FOR VICINITY OF SUSQUEHANNA SITE (Ref. 2.3-20) (INCHES)

RECURRENCE INTERVAL

DURATION	1 YR	2 YR	5 YR	10 YR	25 YR	50 YR	100 YR
1 Hour	1.1	1.4	1.6	2.0	2.2	2.5	2.8
2 Hour	1.4	1.6	2.1	2.5	2.8	3.2	3.5
3 Hour	1.4	1.8	2.3	2.7	3.1	3.5	3.8
6 Hour	1.8	2.2	2.8	3.3	3.9	4.2	4.8
12 Hour	2.2	2.7	3.4	4.0	4.7	5.0	5.8
24 Hour	2.5	3.0	4.0	4.7	5.3	6.0	6.8

TABLE 2.3-35

PROBABLE MAXIMUM PRECIPITATION FOR VARYING RAINFALL DURATIONS AND AREAS (Ref. 2.3-21) (INCHES)

AREA	DURATION (HOURS)				
(Mi^2)	6	12	24	48	72
10	25.5	29.5	31.0	35.0	36.5
200	17.0	20.5	23.0	26.0	27.0

TABLE 2.3-36

PRECIPITATION DATA FOR THE SUSQUEHANNA SITE

(Inches of Water) (1973-1976)

MONTH	TOTAL
January	3.68
February	2.53
March	3.67
April	3.73
May	4.19
June	4.82
July	4.73
August	3.59
September	7.54
October	4.40
November	2.76
December	2.21
Annual	47.83

TABLE 2.3-37

FREQUENCY DISTRIBUTION OF PRECEIPITATION
DATA PERIOD: JANUARY 1973-1976

PRECIPITATION CLASS INTERVAL (INCHES)			FREQUENCY DISTRIBITION OF PRECIPITATION 1 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 2 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 3 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 6 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 12 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 24 HOUR DURATION	
.0	TO	.1	190	89.20	34	45.95	0	0.00	0	0.00	0	0.00	0	0.00
.1	TO	.2	15	7.04	33	44.59	28	75.68	0	0.00	0	0.00	0	0.00
.2	TO	.3	3	1.41	2	2.70	5	13.51	3	27.27	0	0.00	0	0.00
3	TO	.4	2	.94	2	2.70	2	5.41	6	54.55	0	0.00	0	0.00
.4	TO	.5	1	.47	0	0.00	1	2.70	0	0.00	0	0.00	0	0.00
.5	TO	.6	0	0.00	1	1.35	1	2.70	1	9.09	0	0.00	0	0.00
.6	TO	.7	1	.47	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.7	TO	.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.8	TO	.9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.9	TO	1.0	1	.47	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.0	TO	1.1	0	0.00	1	1.35	0	0.00	0	0.00	0	0.00	0	0.00
1.1	TO	1.2	0	0.00	0	0.00	0	0.00	0	0.00	1	50.00	0	0.00
1.2	TO	1.3	0	0.00	1	1.35	0	0.00	0	0.00	0	0.00	0	0.00
1.3	TO	1.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.4	TO	1.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.5	TO	1.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.6	TO	1.7	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.7	TO	1.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.8	TO	1.9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.9	TO	2.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.0	TO	2.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.2	TO	2.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.4	TO	2.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.6	TO	2.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.8	TO	3.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.0	TO	3.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.2	TO	3.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.4	TO	3.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.6	TO	3.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.8	TO	4.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.0	TO	4.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.5	T0	5.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.0	TO	5.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.5	TO	6.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.0	TO	6.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.5	TO	7.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.0	TO	7.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.0
7.5	TO	8.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
8.0	TO	9.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
9.0	TO	10.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
10.0	TO	11.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
11.0	TO	12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
	GT	12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
TOTAL			213	100.00	74	100.00	37	100.00	11	100.00	2	100.00	0	0.00
MAXIMUM AMT.			.91		1.30		.60		.76		1.12		0.00	

TOTAL PRECIPITATION FOR DATA PERIOD 14.71 INCHES

	NO.	PCT.
OBSERVATIONS WITH NO PRECIPITATION	2763	92.84
OBSERVATIONS WITH PRECIPITATION GE 0.01NCH	213	7.16
TOTAL VALID OBSERVATIONS	2976	100.00

	NO.	PCT.
VALID OBSERVATIONS	2976	100.00
INVALID OBSERVATIONS	0	0.00
TOTAL OBSERVATIONS	2976	100.00

TABLE 2.3-38

FREQUENCY DISTRIBUTION OF PRECIPITATION
DATA PERIOD: FEBRUARY 1973-1976

PRECIPITATION CLASS INTERVAL (INCHES)	FREQUENCY DISTRIBUTION OF PRECIPITATION 1 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 2 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 3 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 6 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 12 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 24 HOUR DURATION	
	NO.	PCT.	NO.	PCT.	NO.	PCT.	NO.	PCT.	NO.	PCT.	NO.	PCT.
.0 TO .1	155	88.07	37	60.66	0	0.00	0	0.00	0	0.00	0	0.00
.1 TO .2	19	10.80	19	31.15	24	82.76	0	0.00	0	0.00	0	0.00
.2 TO .3	2	1.14	2	3.28	1	3.45	0	0.00	0	0.00	0	0.00
.3 TO .4	0	0.00	3	4.92	3	10.34	3	60.00	0	0.00	0	0.00
.4 TO .5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.5 TO .6	0	0.00	0	0.00	1	3.45	1	20.00	0	0.00	0	0.00
.6 TO .7	0	0.00	0	0.00	0	0.00	1	20.00	0	0.00	0	0.00
.7 TO .8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.8 TO .9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.9 TO 1.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.0 TO 1.1	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.1 TO 1.2	0	0.00	0	0.00	0	0.00	0	0.00	1	100.00	0	0.00
1.2 TO 1.3	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.3 TO 1.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.4 TO 1.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.5 TO 1.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.6 TO 1.7	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.7 TO 1.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.8 TO 1.9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.9 TO 2.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.0 TO 2.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.2 TO 2.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.4 TO 2.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.6 TO 2.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.8 TO 3.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.0 TO 3.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.2 TO 3.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.4 TO 3.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.6 TO 3.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.8 TO 4.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.0 TO 4.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.5 TO 5.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.0 TO 5.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.5 TO 6.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.0 TO 6.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.5 TO 7.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.0 TO 7.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.5 TO 8.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
8.0 TO 9.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
9.0 TO 10.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
10.0 TO 11.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
11.0 TO 12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
GT 12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
TOTAL	176	100.00	61	100.00	29	100.00	5	100.00	1	100.00	0	0.00
MAXIMUM AMT.	.28		.40		.52		.64		1.16		0.00	

TOTAL PRECIPITATION FOR DATA PERIOD 10.12 INCHES

	NO.	PCT.
OBSERVATIONS WITH NO PRECIPITATION	2536	93.51
OBSERVATIONS WITH PRECIPITATION GE 0.01 INCH	176	6.49
TOTAL VALID OBSERVATIONS	2712	100.00

	NO.	PCT.
VALID OBSERVATIONS	2712	100.00
INVALID OBSERVATIONS	0	0.00
TOTAL OBSERVATIONS	2712	100.00

TABLE 2.3-39

FREQUENCY DISTRIBUTION OF PRECIPITATION
DATA PERIOD: MARCH 1973-1976

PRECIPITATION CLASS INTERVAL (INCHES)	FREQUENCY DISTRIBUTION OF PRECIPITATION 1 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 2 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 3 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 6 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 12 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 24 HOUR DURATION	
	NO.	PCT.	NO.	PCT.	NO.	PCT.	NO.	PCT.	NO.	PCT.	NO.	PCT.
.0 TO .1	187	84.23	32	46.38	0	0.00	0	0.00	0	0.00	0	0.00
.1 TO .2	33	14.86	25	36.23	11	40.74	0	0.00	0	0.00	0	0.00
.2 TO .3	1	.45	8	11.59	8	29.63	0	0.00	0	0.00	0	0.00
.3 TO .4	0	0.00	4	5.80	5	18.52	0	0.00	0	0.00	0	0.00
.4 TO .5	0	0.00	0	0.00	2	7.41	2	40.00	0	0.00	0	0.00
.5 TO .6	0	0.00	0	0.00	1	3.70	1	20.00	0	0.00	0	0.00
.6 TO .7	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.7 TO .8	0	0.00	0	0.00	0	0.00	2	40.00	0	0.00	0	0.00
.8 TO .9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.9 TO 1.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.0 TO 1.1	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.1 TO 1.2	1	.45	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.2 TO 1.3	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.3 TO 1.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.4 TO 1.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.5 TO 1.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.6 TO 1.7	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.7 TO 1.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.8 TO 1.9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.9 TO 2.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.0 TO 2.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.2 TO 2.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.4 TO 2.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.6 TO 2.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.8 TO 3.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.0 TO 3.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.2 TO 3.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.4 TO 3.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.6 TO 3.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.8 TO 4.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.0 TO 4.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.5 TO 5.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.0 TO 5.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.5 TO 6.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.0 TO 6.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.5 TO 7.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.0 TO 7.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.5 TO 8.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
8.0 TO 9.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
9.0 TO 10.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
10.0 TO 11.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
11.0 TO 12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
GT 12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
TOTAL	222	100.00	69	100.00	27	100.00	5	100.00	0	0.00	0	0.00
MAXIMUM AMT.	1.18		.32		.52		.72		0.00		0.00	

TOTAL PRECIPITATION FOR DATA PERIOD 14.66 INCHES

	NO.	PCT.
OBSERVATIONS WITH NO PRECIPITATION	2752	92.54
OBSERVATIONS WITH PRECIPITATION GE 0.01 INCH	222	7.46
TOTAL VALID OBSERVATIONS	2974	100.00

	NO.	PCT.
VALID OBSERVATIONS	2974	99.93
INVALID OBSERVATIONS	2	.07
TOTAL OBSERVATIONS	2976	100.00

TABLE 2.3-40

FREQUENCY DISTRIBUTION OF PRECIPITATION
DATA PERIOD: APRIL 1973-1976

PRECIPITATION CLASS INTERVAL (INCHES)	FREQUENCY DISTRIBUTION OF PRECIPITATION 1 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 2 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 3 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 6 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 12 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 24 HOUR DURATION NO.	PCT.
.0 TO .1	198	88.39	37	46.25	0	0.00	0	0.00	0	0.00	0	0.00
.1 TO .2	16	7.14	31	38.75	24	66.67	0	0.00	0	0.00	0	0.00
.2 TO .3	6	2.68	4	5.00	4	11.11	3	37.50	0	0.00	0	0.00
.3 TO .4	3	1.34	3	3.75	3	8.33	1	12.50	0	0.00	0	0.00
.4 TO .5	0	0.00	2	2.50	0	0.00	2	25.00	0	0.00	0	0.00
.5 TO .6	0	0.00	2	2.50	3	8.33	0	0.00	1	100.00	0	0.00
.6 TO .7	0	0.00	0	0.00	2	5.56	0	0.00	0	0.00	0	0.00
.7 TO .8	1	.45	1	1.25	0	0.00	2	25.00	0	0.00	0	0.00
.8 TO .9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.9 TO 1.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.0 TO 1.1	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.1 TO 1.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.2 TO 1.3	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.3 TO 1.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.4 TO 1.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.5 TO 1.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.6 TO 1.7	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.7 TO 1.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.8 TO 1.9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.9 TO 2.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.0 TO 2.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.2 TO 2.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.4 TO 2.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.6 TO 2.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.8 TO 3.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.0 TO 3.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.2 TO 3.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.4 TO 3.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.6 TO 3.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.8 TO 4.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.0 TO 4.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.5 TO 5.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.0 TO 5.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.5 TO 6.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.0 TO 6.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.5 TO 7.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.0 TO 7.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.5 TO 8.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
8.0 TO 9.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
9.0 TO 10.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
10.0 TO 11.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
11.0 TO 12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
GT 12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
TOTAL	224	100.00	80	100.00	36	100.00	8	100.00	1	100.00	0	0.00
MAXIMUM AMT.	.71		.79		.64		.72		.52		0.00	

TOTAL PRECIPITATION FOR DATA PERIOD 14.92 INCHES

	NO.	PCT.
OBSERVATIONS WITH NO PRECIPITATION	2607	92.09
OBSERVATIONS WITH PRECIPITATION GE 0.01 INCH	224	7.91
TOTAL VALID OBSERVATIONS	2831	100.00

	NO.	PCT.
VALID OBSERVATIONS	2831	98.30
INVALID OBSERVATIONS	49	1.70
TOTAL OBSERVATIONS	2880	100.00

TABLE 2.3-41

FREQUENCY DISTRIBUTION OF PRECIPITATION
DATE PERIOD: MAY 1973-1976

PRECIPITATION CLASS INTERVAL (INCHES)	FREQUENCY DISTRIBUTION OF PRECIPITATION 1 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 2 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 3 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 6 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 12 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 24 HOUR DURATION NO.	PCT.
.0 TO .1	182	81.98	37	50.00	0	0.00	0	0.00	0	0.00	0	0.00
.1 TO .2	29	13.06	24	32.43	19	55.88	0	0.00	0	0.00	0	0.00
.2 TO .3	3	1.35	3	4.05	10	29.41	1	20.00	0	0.00	0	0.00
.3 TO .4	4	1.80	2	2.70	0	0.00	1	20.00	0	0.00	0	0.00
.4 TO .5	2	.90	1	1.35	1	2.94	0	0.00	0	0.00	0	0.00
.5 TO .6	1	.45	5	6.76	0	0.00	1	20.00	0	0.00	0	0.00
.6 TO .7	0	0.00	0	0.00	1	2.94	0	0.00	0	0.00	0	0.00
.7 TO .8	0	0.00	1	1.35	1	2.94	0	0.00	0	0.00	0	0.00
.8 TO .9	0	0.00	0	0.00	1	2.94	1	20.00	0	0.00	0	0.00
.9 TO 1.0	0	0.00	0	0.00	0	0.00	1	20.00	0	0.00	0	0.00
1.0 TO 1.1	1	.45	1	1.35	0	0.00	0	0.00	0	0.00	0	0.00
1.1 TO 1.2	0	0.00	0	0.00	1	2.94	0	0.00	0	0.00	0	0.00
1.2 TO 1.3	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.3 TO 1.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.4 TO 1.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.5 TO 1.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.6 TO 1.7	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.7 TO 1.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.8 TO 1.9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.9 TO 2.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.0 TO 2.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.2 TO 2.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.4 TO 2.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.6 TO 2.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.8 TO 3.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.0 TO 3.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.2 TO 3.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.4 TO 3.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.6 TO 3.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.8 TO 4.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.0 TO 4.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.5 TO 5.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.0 TO 5.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.5 TO 6.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.0 TO 6.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.5 TO 7.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.0 TO 7.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.5 TO 8.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
8.0 TO 9.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
9.0 TO 10.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
10.0 TO 11.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
11.0 TO 12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
GT 12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
TOTAL	222	100.00	74	100.00	34	100.00	5	100.00	0	0.00	0	0.00
MAXIMUM AMT.	1.06		1.10		1.14		.99		0.00		0.00	

TOTAL PRECIPITATION FOR DATA PERIOD 16.75 INCHES

	NO.	PCT.
OBSERVATIONS WITH NO PRECIPITATION	2754	92.54
OBSERVATIONS WITH PRECIPITATION GE 0.01 INCH	222	7.46
TOTAL VALID OBSERVATIONS	2976	100.00

	NO.	PCT.
VALID OBSERVATIONS	2976	100.00
INVALID OBSERVATIONS	0	0.00
TOTAL OBSERVATIONS	2976	100.00

TABLE 2.3-42

FREQUENCY DISTRIBUTION OF PRECEIPITATION
DATA PERIOD: JUNE 1973-1976

PRECIPITATION CLASS INTERVAL (INCHES)			FREQUENCY DISTRIBITION OF PRECIPITATION 1 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 2 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 3 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 6 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 12 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 24 HOUR DURATION	
.0	TO	.1	180	78.60	44	58.67	0	0.00	0	0.00	0	0.00	0	0.00
.1	TO	.2	33	14.41	16	21.33	22	68.75	0	0.00	0	0.00	0	0.00
.2	TO	.3	5	2.18	5	6.67	5	15.63	4	57.14	0	0.00	0	0.00
3	TO	.4	4	1.75	4	5.33	1	3.13	1	14.29	0	0.00	0	0.00
.4	TO	.5	2	.87	2	2.67	1	3.13	0	0.00	0	0.00	0	0.00
.5	TO	.6	3	1.31	2	2.67	0	0.00	2	28.57	0	0.00	0	0.00
.6	TO	.7	2	.87	1	1.33	2	6.25	0	0.00	0	0.00	0	0.00
.7	TO	.8	0	0.00	1	1.33	1	2.94	0	0.00	0	0.00	0	0.00
.8	TO	.9	0	0.00	0	0.00	1	2.94	1	20.00	0	0.00	0	0.00
.9	TO	1.0	0	0.00	0	0.00	0	0.00	1	20.00	0	0.00	0	0.00
1.0	TO	1.1	1	.45	1	1.35	0	0.00	0	0.00	0	0.00	0	0.00
1.1	TO	1.2	0	0.00	0	0.00	1	2.94	0	0.00	0	0.00	0	0.00
1.2	TO	1.3	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.3	TO	1.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.4	TO	1.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.5	TO	1.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.6	TO	1.7	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.7	TO	1.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.8	TO	1.9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.9	TO	2.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.0	TO	2.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.2	TO	2.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.4	TO	2.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.6	TO	2.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.8	TO	3.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.0	TO	3.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.2	TO	3.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.4	TO	3.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.6	TO	3.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.8	TO	4.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.0	TO	4.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.5	T0	5.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.0	TO	5.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.5	TO	6.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.0	TO	6.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.5	TO	7.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.0	TO	7.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.0
7.5	TO	8.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
8.0	TO	9.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
9.0	TO	10.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
10.0	TO	11.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
11.0	TO	12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
	GT	12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
TOTAL			229	100.00	75	100.00	32	100.00	7	100.00	1	0.00	0	0.00
MAXIMUM AMT.			.67		.71		.99		.60		.48		0.00	

TOTAL PRECIPITATION FOR DATA PERIOD 19.29 INCHES

	NO.	PCT.		NO.	PCT.
OBSERVATIONS WITH NO PRECIPITATION	2651	92.05	VALID OBSERVATIONS	2880	100.00
OBSERVATIONS WITH PRECIPITATION GE 0.01NCH	229	7.95	INVALID OBSERVATIONS	0	0.00
TOTAL VALID OBSERVATIONS	2880	100.00	TOTAL OBSERVATIONS	2880	100.00

TABLE 2.3-43

FREQUENCY DISTRIBUTION OF PRECEIPITATION
DATA PERIOD: JULY 1973-1976

PRECIPITATION CLASS INTERVAL (INCHES)			FREQUENCY DISTRIBUTION OF PRECIPITATION 1 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 2 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 3 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 6 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 12 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 24 HOUR DURATION	
.0	TO	.1	128	72.73	14	33.33	0	0.00	0	0.00	0	0.00	0	0.00
.1	TO	.2	34	19.32	15	35.71	8	40.00	0	0.00	0	0.00	0	0.00
.2	TO	.3	3	1.70	3	7.14	4	20.00	1	33.33	0	0.00	0	0.00
3	TO	.4	3	1.70	6	14.29	4	20.00	1	14.29	0	0.00	0	0.00
.4	TO	.5	1	.57	0	0.00	1	5.00	0	0.00	0	0.00	0	0.00
.5	TO	.6	2	1.14	0	0.00	0	0.00	1	33.33	0	0.00	0	0.00
.6	TO	.7	1	.57	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.7	TO	.8	1	.57	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.8	TO	.9	1	.57	1	2.38	0	0.00	0	0.00	0	0.00	0	0.00
.9	TO	1.0	0	0.00	1	2.38	0	0.00	0	0.00	0	0.00	0	0.00
1.0	TO	1.1	1	.57	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.1	TO	1.2	0	0.00	1	2.38	2	10.00	0	0.00	0	0.00	0	0.00
1.2	TO	1.3	0	0.00	1	2.38	0	0.00	0	0.00	0	0.00	0	0.00
1.3	TO	1.4	0	0.00	0	0.00	1	5.00	0	0.00	0	0.00	0	0.00
1.4	TO	1.5	0	0.00	0	0.00	0	0.00	1	33.33	0	0.00	0	0.00
1.5	TO	1.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.6	TO	1.7	1	.57	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.7	TO	1.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.8	TO	1.9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.9	TO	2.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.0	TO	2.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.2	TO	2.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.4	TO	2.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.6	TO	2.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.8	TO	3.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.0	TO	3.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.2	TO	3.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.4	TO	3.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.6	TO	3.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.8	TO	4.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.0	TO	4.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.5	T0	5.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.0	TO	5.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.5	TO	6.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.0	TO	6.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.5	TO	7.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.0	TO	7.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.0
7.5	TO	8.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
8.0	TO	9.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
9.0	TO	10.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
10.0	TO	11.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
11.0	TO	12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
	GT	12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
TOTAL			176	100.00	42	100.00	20	100.00	3	100.00	0	0.00	0	0 00
MAXIMUM AMT.			1.61		1.22		1.38		1.46		0 00		0.00	
TOTAL PRECIPITATION FOR DATA PERIOD							18.90 INCHES							

	NO.	PCT.		NO	PCT.
OBSERVATIONS WITH NO PRECIPITATION	2799	94.08	VALID OBSERVATIONS	2975	99.97
OBSERVATIONS WITH PRECIPITATION GE 0.01NCH	176	5.92	INVALID OBSERVATIONS	1	.03
TOTAL VALID OBSERVATIONS	2975	100.00	TOTAL OBSERVATIONS	2976	100.00

TABLE 2.3-44

FREQUENCY DISTRIBUTION OF PRECIPITATION
DATA PERIOD: AUGUST 1973-1976

PRECIPITATION CLASS INTERVAL (INCHES)	FREQUENCY DISTRIBUTION OF PRECIPITATION 1 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 2 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 3 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 6 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 12 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 24 HOUR DURATION	
	NO.	PCT.	NO.	PCT.	NO.	PCT.	NO.	PCT.	NO.	PCT.	NO.	PCT.
.0 TO .1	152	83.06	29	58.00	0	0.00	0	0.00	0	0.00	0	0.00
.1 TO .2	19	10.38	7	14.00	12	50.00	0	0.00	0	0.00	0	0.00
.2 TO .3	6	3.28	4	8.00	4	16.67	2	50.00	0	0.00	0	0.00
.3 TO .4	3	1.64	5	10.00	4	16.67	0	0.00	0	0.00	0	0.00
.4 TO .5	1	.55	1	2.00	0	0.00	0	0.00	0	0.00	0	0.00
.5 TO .6	0	0.00	2	4.00	0	0.00	1	25.00	0	0.00	0	0.00
.6 TO .7	1	.55	0	0.00	0	0.00	1	25.00	0	0.00	0	0.00
.7 TO .8	0	0.00	0	0.00	2	8.33	0	0.00	0	0.00	0	0.00
.8 TO .9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.9 TO 1.0	0	0.00	1	2.00	1	4.17	0	0.00	0	0.00	0	0.00
1.0 TO 1.1	1	.55	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.1 TO 1.2	0	0.00	1	2.00	0	0.00	0	0.00	0	0.00	0	0.00
1.2 TO 1.3	0	0.00	0	0.00	1	4.17	0	0.00	0	0.00	0	0.00
1.3 TO 1.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.4 TO 1.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.5 TO 1.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.6 TO 1.7	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.7 TO 1.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.8 TO 1.9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.9 TO 2.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.0 TO 2.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.2 TO 2.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.4 TO 2.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.6 TO 2.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.8 TO 3.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.0 TO 3.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.2 TO 3.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.4 TO 3.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.6 TO 3.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.8 TO 4.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.0 TO 4.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.5 TO 5.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.0 TO 5.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.5 TO 6.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.0 TO 6.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.5 TO 7.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.0 TO 7.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.5 TO 8.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
8.0 TO 9.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
9.0 TO 10.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
10.0 TO 11.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
11.0 TO 12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
GT 12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
TOTAL	183	100.00	50	100.00	24	100.00	4	100.00	0	0.00	0	0.00
MAXIMUM AMT.	1.06		1.14		1.30		.68		0.00		0.00	

TOTAL PRECIPITATION FOR DATA PERIOD 14.37 INCHES

	NO.	PCT.
OBSERVATIONS WITH NO PRECIPITATION	2793	93.85
OBSERVATIONS WITH PRECIPITATION GE 0.01 INCH	183	6.15
TOTAL VALID OBSERVATIONS	2976	100.00

	NO.	PCT.
VALID OBSERVATIONS	2976	100.00
INVALID OBSERVATIONS	0	0.00
TOTAL OBSERVATIONS	2976	100.00

TABLE 2.3-45

FREQUENCY DISTRIBUTION OF PRECEIPITATION
DATA PERIOD: SEPTEMBER 1973-1976

PRECIPITATION CLASS INTERVAL (INCHES)			FREQUENCY DISTRIBITION OF PRECIPITATION 1 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 2 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 3 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 6 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 12 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 24 HOUR DURATION	
.0	TO	.1	225	76.01	47	41.59	0	0.00	0	0.00	0	0.00	0	0.00
.1	TO	.2	37	12.50	31	27.43	27	49.09	0	0.00	0	0.00	0	0.00
.2	TO	.3	14	4.73	7	6.19	4	12.73	1	6.25	0	0.00	0	0.00
.3	TO	.4	8	2.70	10	8.85	4	7.27	3	18.75	0	0.00	0	0.00
.4	TO	.5	6	2.03	5	4.42	0	1.82	1	6.25	0	0.00	0	0.00
.5	TO	.6	4	1.35	5	4.42	0	10.91	1	6.25	1	33.33	0	0.00
.6	TO	.7	0	0.00	2	1.77	0	3.64	2	12.50	0	0.00	0	0.00
.7	TO	.8	1	.34	3	2.65	2	7.27	1	6.25	0	0.00	0	0.00
.8	TO	.9	0	0.00	1	.88	0	5.45	1	6.25	0	0.00	0	0.00
.9	TO	1.0	0	0.00	0	0.00	1	0.00	1	25.00	0	0.00	0	0.00
1.0	TO	1.1	0	.34	1	.88	0	0.00	4	6.25	0	0.00	0	0.00
1.1	TO	1.2	0	0.00	0	0.00	0	0.00	1	0.00	0	0.00	0	0.00
1.2	TO	1.3	0	0.00	0	0.00	1	1.82	0	0.00	0	0.00	0	0.00
1.3	TO	1.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.4	TO	1.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.5	TO	1.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.6	TO	1.7	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.7	TO	1.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.8	TO	1.9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.9	TO	2.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.0	TO	2.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.2	TO	2.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.4	TO	2.6	1	.34	1	.88	0	0.00	0	0.00	0	0.00	0	0.00
2.6	TO	2.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.8	TO	3.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.0	TO	3.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.2	TO	3.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.4	TO	3.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.6	TO	3.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.8	TO	4.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.0	TO	4.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.5	T0	5.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.0	TO	5.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.5	TO	6.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.0	TO	6.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.5	TO	7.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.0	TO	7.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.0
7.5	TO	8.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
8.0	TO	9.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
9.0	TO	10.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
10.0	TO	11.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
11.0	TO	12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
	GT	12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
TOTAL			296	100.00	113	100.00	55	100.00	16	100.00	3	0.00	0	0.00
MAXIMUM AMT.			2.56		2.60		1.27		1.11		2.19		0.00	

TOTAL PRECIPITATION FOR DATA PERIOD 30.17 INCHES

	NO.	PCT.
OBSERVATIONS WITH NO PRECIPITATION	2578	89.70
OBSERVATIONS WITH PRECIPITATION GE 0.01NCH	296	10.30
TOTAL VALID OBSERVATIONS	2874	100.00

	NO.	PCT.
VALID OBSERVATIONS	2874	99.79
INVALID OBSERVATIONS	6	.21
TOTAL OBSERVATIONS	2880	100.00

TABLE 2.3-46

FREQUENCY DISTRIBUTION OF PRECIPITATION
DATA PERIOD: OCTOBER 1973-1976

PRECIPITATION CLASS INTERVAL (INCHES)	FREQUENCY DISTRIBUTION OF PRECIPITATION 1 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 2 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 3 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 6 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 12 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 24 HOUR DURATION NO.	PCT.
.0 TO .1	187	82.02	37	47.44	0	0.00	0	0.00	0	0.00	0	0.00
.1 TO .2	29	12.72	27	34.62	21	56.76	0	0.00	0	0.00	0	0.00
.2 TO .3	3	1.32	3	3.85	4	10.81	1	11.11	0	0.00	0	0.00
.3 TO .4	4	1.75	4	5.13	6	16.22	2	22.22	0	0.00	0	0.00
.4 TO .5	2	.88	3	3.85	2	5.41	1	11.11	0	0.00	0	0.00
.5 TO .6	2	.88	3	3.85	1	2.70	0	0.00	0	0.00	0	0.00
.6 TO .7	0	0.00	0	0.00	1	2.70	1	11.11	0	0.00	0	0.00
.7 TO .8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.8 TO .9	1	.44	0	0.00	1	2.70	2	22.22	0	0.00	0	0.00
.9 TO 1.0	0	0.00	0	0.00	0	0.00	1	11.11	0	0.00	0	0.00
1.0 TO 1.1	0	0.00	0	0.00	0	0.00	0	0.00	1	50.00	0	0.00
1.1 TO 1.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.2 TO 1.3	0	0.00	1	1.28	0	0.00	0	0.00	0	0.00	0	0.00
1.3 TO 1.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.4 TO 1.5	0	0.00	0	0.00	1	2.70	0	0.00	0	0.00	0	0.00
1.5 TO 1.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.6 TO 1.7	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.7 TO 1.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.8 TO 1.9	0	0.00	0	0.00	0	0.00	1	11.11	0	0.00	0	0.00
1.9 TO 2.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.0 TO 2.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.2 TO 2.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.4 TO 2.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.6 TO 2.8	0	0.00	0	0.00	0	0.00	0	0.00	1	50.00	0	0.00
2.8 TO 3.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.0 TO 3.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.2 TO 3.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.4 TO 3.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.6 TO 3.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.8 TO 4.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.0 TO 4.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.5 TO 5.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.0 TO 5.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.5 TO 6.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.0 TO 6.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.5 TO 7.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.0 TO 7.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.5 TO 8.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
8.0 TO 9.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
9.0 TO 10.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
10.0 TO 11.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
11.0 TO 12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
GT 12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
TOTAL	228	100.00	78	100.00	37	100.00	9	100.00	2	100.00	0	0.00
MAXIMUM AMT.	.83		1.26		1.50		1.90		2.74		0.00	

TOTAL PRECIPITATION FOR DATA PERIOD 17.59 INCHES

	NO.	PCT.
OBSERVATIONS WITH NO PRECIPITATION	2748	92.34
OBSERVATIONS WITH PRECIPITATION GE 0.01 INCH	228	7.66
TOTAL VALID OBSERVATIONS	2976	100.00

	NO.	PCT.
VALID OBSERVATIONS	2976	100.00
INVALID OBSERVATIONS	0	0.00
TOTAL OBSERVATIONS	2976	100.00

TABLE 2.3-47

FREQUENCY DISTRIBUTION OF PRECEIPITATION
DATA PERIOD: NOVEMBER 1973-1976

PRECIPITATION CLASS INTERVAL (INCHES)			FREQUENCY DISTRIBITION OF PRECIPITATION 1 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 2 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 3 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 6 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 12 HOUR DURATION		FREQUENCY DISTRIBITION OF PRECIPITATION 24 HOUR DURATION	
.0	TO	.1	156	86.67	34	58.62	0	0.00	0	0.00	0	0.00	0	0.00
.1	TO	.2	19	10.56	15	25.86	18	75.00	0	0.00	0	0.00	0	0.00
.2	TO	.3	4	2.22	6	10.34	0	0.00	3	42.86	0	0.00	0	0.00
3	TO	.4	1	.56	3	5.17	5	20.83	1	14.29	0	0.00	0	0.00
.4	TO	.5	0	0.00	0	0.00	1	4.17	0	0.00	0	0.00	0	0.00
.5	TO	.6	0	0.00	0	0.00	0	0.00	1	14.29	0	50.0	0	0.00
.6	TO	.7	0	0.00	0	0.00	0	0.00	1	14.29	0	0.00	0	0.00
.7	TO	.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.8	TO	.9	0	0.00	0	0.00	0	0.00	1	14.29	0	0.00	0	0.00
.9	TO	1.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.0	TO	1.1	0	0.00	0	0.00	0	0.00	0	0.00	1	50.00	0	0.00
1.1	TO	1.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.2	TO	1.3	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.3	TO	1.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.4	TO	1.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.5	TO	1.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.6	TO	1.7	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.7	TO	1.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.8	TO	1.9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.9	TO	2.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.0	TO	2.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.2	TO	2.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.4	TO	2.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.6	TO	2.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.8	TO	3.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.0	TO	3.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.2	TO	3.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.4	TO	3.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.6	TO	3.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.8	TO	4.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.0	TO	4.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.5	T0	5.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.0	TO	5.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.5	TO	6.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.0	TO	6.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.5	TO	7.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.0	TO	7.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.5	TO	8.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
8.0	TO	9.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
9.0	TO	10.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
10.0	TO	11.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
11.0	TO	12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0 00	0	0.00
	GT	12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
TOTAL			180	100.00	58	100.00	24	100.00	7	100.00	2	0.00	0	0.00
MAXIMUM AMT.			.35		.40		.48		.84		0.00		0.00	

TOTAL PRECIPITATION FOR DATA PERIOD 11.03 INCHES

	NO.	PCT.		NO.	PCT.
OBSERVATIONS WITH NO PRECIPITATION	2700	93.75	VALID OBSERVATIONS	2880	100.00
OBSERVATIONS WITH PRECIPITATION GE 0.01NCH	180	6.25	INVALID OBSERVATIONS	0	0.00
TOTAL VALID OBSERVATIONS	2880	100.00	TOTAL OBSERVATIONS	2880	100.00

TABLE 2.3-48

FREQUENCY DISTRIBUTION OF PRECIPITATION
DATA PERIOD: DECEMBER 1973-1976

PRECIPITATION CLASS INTERVAL (INCHES)	FREQUENCY DISTRIBUTION OF PRECIPITATION 1 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 2 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 3 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 6 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 12 HOUR DURATION NO.	PCT.	FREQUENCY DISTRIBUTION OF PRECIPITATION 24 HOUR DURATION NO.	PCT.
.0 TO .1	53	68.83	3	12.50	0	0.00	0	0.00	0	0.00	0	0.00
.1 TO .2	15	19.48	12	50.00	5	50.00	0	0.00	0	0.00	0	0.00
.2 TO .3	3	3.90	6	25.00	1	10.00	0	0.00	0	0.00	0	0.00
.3 TO .4	1	1.30	1	4.17	3	30.00	0	0.00	0	0.00	0	0.00
.4 TO .5	2	2.60	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.5 TO .6	0	0.00	0	0.00	0	0.00	3	100.00	0	0.00	0	0.00
.6 TO .7	1	1.30	1	4.17	0	0.00	0	0.00	0	0.00	0	0.00
.7 TO .8	2	2.60	1	4.17	0	0.00	0	0.00	0	0.00	0	0.00
.8 TO .9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
.9 TO 1.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.0 TO 1.1	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.1 TO 1.2	0	0.00	0	0.00	1	10.00	0	0.00	1	100.00	0	0.00
1.2 TO 1.3	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.3 TO 1.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.4 TO 1.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.5 TO 1.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.6 TO 1.7	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.7 TO 1.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.8 TO 1.9	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.9 TO 2.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.0 TO 2.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.2 TO 2.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.4 TO 2.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.6 TO 2.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.8 TO 3.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.0 TO 3.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.2 TO 3.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.4 TO 3.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.6 TO 3.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.8 TO 4.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.0 TO 4.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.5 TO 5.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.0 TO 5.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.5 TO 6.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.0 TO 6.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.5 TO 7.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.0 TO 7.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.5 TO 8.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
8.0 TO 9.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
9.0 TO 10.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
10.0 TO 11.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
11.0 TO 12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
GT 12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
TOTAL	77	100.00	24	100.00	10	100.00	3	100.00	1	100.00	0	0.00
MAXIMUM AMT.	.75		.79		1.18		.60		1.12		0.00	

TOTAL PRECIPITATION FOR DATA PERIOD 8.82 INCHES

	NO.	PCT.
OBSERVATIONS WITH NO PRECIPITATION	1645	95.53
OBSERVATIONS WITH PRECIPITATION GE 0.01 INCH	77	4.47
TOTAL VALID OBSERVATIONS	1722	100.00

	NO.	PCT.
VALID OBSERVATIONS	1722	57.86
INVALID OBSERVATIONS	1254	42.14
TOTAL OBSERVATIONS	2976	100.00

TABLE 2.3-49

FREQUENCY DISTRIBUTION OF PRECEIPITATION
DATA PERIOD: JANUARY 1973 - DECEMBER 1976

PRECIPITATION CLASS INTERVAL (INCHES)			FREQUENCY DISTRIBUTION OF PRECIPITATION 1 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 2 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 3 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 6 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 12 HOUR DURATION		FREQUENCY DISTRIBUTION OF PRECIPITATION 24 HOUR DURATION	
.0	TO	.1	1993	82.15	385	48.25	0	0.00	0	0.00	0	0.00	0	0.00
.1	TO	.2	298	12.28	255	31.95	220	60.11	0	0.00	0	0.00	0	0.00
.2	TO	.3	53	2.18	53	6.64	54	14.75	19	22.89	0	0.00	0	0.00
3	TO	.4	33	1.36	47	5.89	38	10.38	18	21.69	0	0.00	0	0.00
.4	TO	.5	17	.70	14	1.75	11	3.01	6	7.23	1	7.69	0	0.00
.5	TO	.6	12	.49	20	2.51	13	3.55	13	15.66	3	23.08	0	0.00
.6	TO	.7	6	.25	4	.50	8	2.19	7	8.43	0	0.00	0	0.00
.7	TO	.8	5	.21	7	.88	7	1.91	5	6.02	1	7.69	0	0.00
.8	TO	.9	2	.08	2	.25	5	1.37	5	6.02	0	0.00	0	0.00
.9	TO	1.0	1	.04	2	.25	2	.55	3	3.61	0	0.00	0	0.00
1.0	TO	1.1	3	.12	3	.38	0	0.00	4	4.82	0	0.00	0	0.00
1.1	TO	1.2	1	.04	2	.25	4	1.09	1	1.20	4	30.77	0	0.00
1.2	TO	1.3	0	0.00	3.	.38	2	.55	0	0.00	0	0.00	0	0.00
1.3	TO	1.4	0	0.00	0	0.00	1	.27	0	0.00	0	0.00	0	0.00
1.4	TO	1.5	0	0.00	0	0.00	1	.27	1	1.20	0	0.00	0	0.00
1.5	TO	1.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.6	TO	1.7	1	.04	0	0.00	0	0.00	0	0.00	1	7.69	0	0.00
1.7	TO	1.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
1.8	TO	1.9	0	0.00	0	0.00	0	0.00	1	1.20	0	0.00	0	0.00
1.9	TO	2.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.0	TO	2.2	0	0.00	0	0.00	0	0.00	0	0.00	1	7.69	0	0.00
2.2	TO	2.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
2.4	TO	2.6	1	.04	1	.13	0	0.00	0	0.00	0	0.00	0	0.00
2.6	TO	2.8	0	0.00	0	0.00	0	0.00	0	0.00	1	7.69	0	0.00
2.8	TO	3.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.0	TO	3.2	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.2	TO	3.4	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.4	TO	3.6	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.6	TO	3.8	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
3.8	TO	4.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.0	TO	4.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
4.5	TO	5.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.0	TO	5.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
5.5	TO	6.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.0	TO	6.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
6.5	TO	7.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.0	TO	7.5	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
7.5	TO	8.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
8.0	TO	9.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
9.0	TO	10.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
10.0	TO	11.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
11.0	TO	12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
	GT	12.0	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00	0	0.00
TOTAL			2426	100.00	798	100.00	366	100.00	83	100.00	13	100.00	0	0.00
MAXIMUM AMT.			2.56		2.60		1.50		1.90		2.74		0.00	

TOTAL PRECIPITATION FOR DATA PERIOD 191.33 INCHES

	NO.	PCT.
OBSERVATIONS WITH NO PRECIPITATION	31326	92.81
OBSERVATIONS WITH PRECIPITATION GE 0.01NCH	2426	7.19
TOTAL VALID OBSERVATIONS	33752	100.00

	NO.	PCT.
VALID OBSERVATIONS	33752	96.26
INVALLID OBSERVATIONS	1312	3.74
TOTAL OBSERVATIONS	35064	100.00

TABLE 2.3-50

PRECIPITATION WIND ROSE
JANUARY 1973 - 1976

WIND SECTOR	WIND SPEED CATEGORIES(METERS PER SECOND) 0.0-1.5	1.5-3.0	3.0-5.0	5.0-7.5	7.5-10.0	>10.0	TOTAL	MEAN SPEED
NNE	8 3.88	8 3.88	0 0.00	0 0.00	0 0.00	0 0.00	16 7.77	1.53
NE	7 3.40	6 2.91	0 0.00	0 0.00	0 0.00	0 0.00	13 6.31	1.56
ENE	6 2.91	4 1.94	0 0.00	0 0.00	0 0.00	0 0.00	10 4.85	1.44
E	3 1.46	6 2.91	0 0.00	0 0.00	0 0.00	0 0.00	9 4.37	2.09
ESE	10 4.85	8 3.88	2 .97	0 0.00	0 0.00	0 0.00	20 9.71	1.70
SE	4 1.94	4 1.94	0 0.00	2 .97	0 0.00	0 0.00	10 4.85	2.38
SSE	7 3.40	5 2.43	1 .49	1 .49	0 0.00	0 0.00	14 6.80	1.89
S	7 3.40	5 2.43	3 1.46	0 0.00	0 0.00	0 0.00	15 7.28	2.03
SSW	4 1.94	2 .97	5 2.43	4 1.94	0 0.00	0 0.00	15 7.28	3.69
SW	4 1.94	4 1.94	3 1.46	0 0.00	0 0.00	0 0.00	11 5.34	2.05
	3 1.46	4 1.94	4 1.94	2 .97	0 0.00	0 0.00	13 6.31	3.29
W	9 4.37	0 0.00	4 1.94	3 1.46	0 0.00	0 0.00	16 7.77	2.70
WNW	6 2.91	2 .97	0 0.00	1 .49	0 0.00	0 0.00	9 4.37	1.83
NW	10 4.85	1 .49	0 0.00	0 0.00	0 0.00	0 0.00	11 5.34	1.25
NNW	7 3.40	0 0.00	1 .49	0 0.00	0 0.00	0 0.00	8 3.88	1.32
N	7 3.40	7 3.40	0 0.00	0 0.00	0 0.00	0 0.00	14 6.80	1.51
CALM	2 .97						2 .97	CALM
TOTAL	104 50.49	66 32.04	23 11.17	13 6.31	0 0.00	0 0.00	206 100.00	2.03

NUMBER OF VALID OBSERVATIONS WITH PRECIPITATION	206	6.92 PCT.
NUMBER OF VALID OBSERVATIONS WITHOUT PRECIPITATION	2112	70.97 PCT.
NUMBER OF INVALID OBSERVATIONS	658	22.11 PCT.
TOTAL NUMBER OF OBSERVATIONS	2976	100.00 PCT.
TOTAL AMOUNT OF PRECIPITATION FOR DATA PERIOD		14.08 INCHES

KEY XXX NUMBER OF OCCURRENCES
XXX PERCENT OCCURRENCES

TABLE 2.3-51

PRECIPITATION WIND ROSE
FEBRUARY 1973 - 1976

WIND SECTOR	WIND SPEED CATEGORIES(METERS PER SECOND) 0.0-1.5	1.5-3.0	3.0-5.0	5.0-7.5	7.5-10.0	>10.0	TOTAL	MEAN SPEED
NNE	12 7.50	5 3.12	0 0.00	0 0.00	0 0.00	0 0.00	17 10.62	1.32
NE	2 1.25	4 2.50	0 0.00	0 0.00	0 0.00	0 0.00	6 3.75	1.87
ENE	6 3.75	5 3.12	2 1.25	0 0.00	0 0.00	0 0.00	13 8.12	1.96
E	4 2.50	2 1.25	3 1.87	0 0.00	0 0.00	0 0.00	9 5.62	2.10
ESE	5 3.12	2 1.25	0 0.00	0 0.00	0 0.00	0 0.00	7 4.37	1.26
SE	5 3.12	6 3.75	1 .62	1 .62	0 0.00	0 0.00	13 8.12	2.05
SSE	7 4.37	2 1.25	4 2.50	0 0.00	0 0.00	0 0.00	13 8.12	2.05
S	2 1.25	4 2.50	1 .62	6 3.75	2 1.25	0 0.00	15 9.37	4.55
SSW	3 1.87	1 .62	4 2.50	6 3.75	0 0.00	0 0.00	14 8.75	4.37
SW	3 1.87	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	3 1.87	.83
WSW	4 2.50	2 1.25	2 1.25	0 0.00	0 0.00	0 0.00	8 5.00	2.06
W	1 .62	0 0.00	2 1.25	1 .62	0 0.00	0 0.00	4 2.50	3.87
WNW	3 1.87	6 3.75	1 .62	0 0.00	0 0.00	0 0.00	10 6.25	1.95
NW	2 1.25	9 5.62	1 .62	0 0.00	0 0.00	0 0.00	12 7.50	2.13
NNW	1 .62	2 1.25	1 .62	0 0.00	0 0.00	0 0.00	4 2.50	2.17
N	9 5.62	3 1.87	0 0.00	0 0.00	0 0.00	0 0.00	12 7.50	1.21
CALM	0 0.00						0 0.00	CALM
TOTAL	69 43.12	53 33.12	22 13.75	14 8.75	2 1.25	0 0.00	160 100.00	2.33

NUMBER OF VALID OBSERVATIONS WITH PRECIPITATION	160	5.90 PCT.
NUMBER OF VALID OBSERVATIONS WITHOUT PRECIPITATION	1951	71.94 PCT.
NUMBER OF INVALID OBSERVATIONS	601	22.16 PCT.
TOTAL NUMBER OF OBSERVATIONS	2712	100.00 PCT.
TOTAL AMOUNT OF PRECIPITATION FOR DATA PERIOD		8.92 INCHES

KEY XXX NUMBER OF OCCURRENCES
XXX PERCENT OCCURRENCES

TABLE 2.3-52

PRECIPITATION WIND ROSE
FEBRUARY 1973 - 1976

WIND SECTOR	WIND SPEED CATEGORIES (METERS PER SECOND) 0.0-1.5	1.5-3.0	3.0-5.0	5.0-7.5	7.5-10.0	>10.0	TOTAL	MEAN SPEED
NNE	3	6	0	0	0	0	9	1.92
	1.36	2.71	0.00	0.00	0.00	0.00	4.07	
NE	2	12	2	3	0	0	22	2.71
	2.26	5.43	.90	1.36	0.00	0.00	9.95	
ENE	3	11	6	1	0	0	21	2.70
	1.36	4.98	2.71	.45	0.00	0.00	9.50	
E	7	8	9	2	0	0	26	2.92
	3.17	3.62	4.07	.90	0.00	0.00	11.76	
ESE	3	4	4	0	0	0	11	2.41
	1.36	1.81	1.81	0.00	0.00	0.00	4.98	
SE	3	6	3	0	0	0	12	2.14
	1.36	2.71	1.36	0.00	0.00	0.00	5.43	
SSE	3	2	0	1	0	0	6	1.97
	1.36	.90	0.00	.45	0.00	0.00	2.71	
S	5	4	2	2	0	0	13	2.53
	2.26	1.81	.90	.90	0.00	0.00	5.88	
SSW	7	2	3	1	0	0	13	2.23
	3.17	.90	1.36	.45	0.00	0.00	5.88	
SW	5	4	0	1	0	0	10	1.92
	2.26	1.81	0.00	.45	0.00	0.00	4.52	
WSW	3	5	2	5	0	0	15	3.21
	1.36	2.26	.90	2.26	0.00	0.00	6.79	
W	8	2	7	7	0	0	24	3.43
	3.62	.90	3.17	3.17	0.00	0.00	10.86	
WNW	4	0	1	4	1	0	10	4.20
	1.81	0.00	.45	1.81	.45	0.00	4.52	
NW	5	0	2	0	0	0	7	1.60
	2.26	0.00	.90	0.00	0.00	0.00	3.17	
NNW	2	6	1	1	0	0	10	2.62
	.90	2.71	.45	.45	0.00	0.00	4.52	
N	2	3	1	4	0	0	10	3.73
	.90	1.36	.45	1.81	0.00	0.00	4.52	
CALM	2						2	CALM
	.90						.90	
TOTAL	70	75	43	32	1	0	221	2.72
	31.67	33.94	19.46	14.48	.45	0.00	100.00	

NUMBER OF VALID OBSERVATIONS WITH PRECIPITATION	221	7.43 PCT.
NUMBER OF VALID OBSERVATIONS WITHOUT PRECIPITATION	2730	91.73 PCT
NUMBER OF INVALID OBSERVATIONS	25	.84 PCT.
TOTAL NUMBER OF OBSERVATIONS	2979	100.00 PCT.
TOTAL AMOUNT OF PRECIPITATION FOR DATA PERIOD		14.62 INCHES

KEY XXX NUMBER OF OCCURRENCES
XXX PERCENT OCCURRENCES

TABLE 2.3-53

PRECIPITATION WIND ROSE
APRIL 1973 - 1976

WIND SECTOR	WIND SPEED CATEGORIES (METERS PER SECOND) 0.0-1.5	1.5-3.0	3.0-5.0	5.0-7.5	7.5-10.0	>10.0	TOTAL	MEAN SPEED
NNE	4	14	8	0	0	0	26	2.53
	1.79	6.25	3.57	0.00	0.00	0.00	11.161	
NE	4	10	32	0	0	0	46	3.25
	1.79	4.46	14.29	0.00	0.00	0.00	20.54	
ENE	3	5	4	3	0	0	15	3.23
	1.34	2.23	1.79	1.34	0.00	0.00	6.70	
E	4	4	4	0	0	0	12	2.26
	1.79	1.79	1.79	0.00	0.00	0.00	5.36	
ESE	4	1	1	0	0	0	6	1.83
	1.79	.45	.45	0.00	0.00	0.00	2.68	
SE	2	0	0	0	0	0	2	.80
	.89	0.00	0.00	0.00	0.00	0.00	.89	
SSE	6	2	0	1	0	0	9	1.81
	2.68	.89	0.00	.45	0.00	0.00	4.02	
S	7	1	3	1	0	0	12	2.29
	3.12	.,45	1.34	.45	0.00	0.00	5.36	
SSW	5	1	2	0	0	0	8	1.85
	2.23	.45	.89	0.00	0.00	0.00	3.57	
SW	7	4	1	0	0	0	12	1.51
	3.12	1.79	.45	0.00	0.00	0.00	5.36	
WSW	4	16	5	2	0	0	27	2.70
	1.79	7.14	2.23	.89	0.00	0.00	12.05	
W	4	4	8	1	0	0	17	3.02
	1.79	1.79	3.57	.45	0.00	0.00	7.59	
WNW	0	2	7	3	0	0	12	4.34
	0.00	.89	3.12	1.34	0.00	0.00	5.36	
NW	1	0	4	1	0	0	6	3.42
	.45	0.00	1.79	.45	0.00	0.00	2.68	
NNW	1	2	3	2	0	0	8	3.69
	.45	.89	1.34	.89	0.00	0.00	3.57	
N	2	2	1	0	0	0	5	1.98
	.89	.89	.45	0.00	0.00	0.00	2.23	
CALM	1						1	CALM
	.45						.45	
TOTAL	59	68	83	14	0	0	224	2.75
	26.34	30.36	37.05	6.25	0.00	0.00	100.00	

NUMBER OF VALID OBSERVATIONS WITH PRECIPITATION	224	7.78 PCT.
NUMBER OF VALID OBSERVATIONS WITHOUT PRECIPITATION	2604	90.42 PCT.
NUMBER OF INVALID OBSERVATIONS	52	1.81 PCT.
TOTAL NUMBER OF OBSERVATIONS	2880	100.00 PCT.
TOTAL AMOUNT OF PRECIPITATION FOR DATA PERIOD		14.92 INCHES

KEY XXX NUMBER OF OCCURRENCES
XXX PERCENT OCCURRENCES

TABLE 2.3-54

PRECIPITATION WIND ROSE
MAY 1973 - 1976

WIND SECTOR	WIND SPEED CATEGORIES(METERS PER SECOND) 0.0-1.5	1.5-3.0	3.0-5.0	5.0-7.5	7.5-10.0	>10.0	TOTAL	MEAN SPEED
NNE	4 1.80	5 2.25	0 0.00	0 0.00	0 0.00	0 0.00	9 4.05	1.58
NE	5 2.25	10 4.50	1 .45	0 0.00	0 0.00	0 0.00	16 7.21	1.90
ENE	10 4.50	7 3.15	0 0.00	0 0.00	0 0.00	0 0.00	17 7.66	1.52
E	7 3.15	5 2.25	0 0.00	0 0.00	0 0.00	0 0.00	12 5.41	1.44
ESE	5 2.25	3 1.35	0 0.00	0 0.00	0 0.00	0 0.00	8 3.60	1.36
SE	5 2.25	1 .45	0 0.00	0 0.00	0 0.00	0 0.00	6 2.70	1.17
SSE	9 4.05	10 4.50	4 1.80	0 0.00	0 0.00	0 0.00	23 10.36	2.00
S	8 3.60	4 1.80	4 1.80	0 0.00	0 0.00	0 0.00	16 7.21	2.08
SSW	8 3.60	2 .90	2 .90	0 0.00	0 0.00	0 0.00	12 5.41	1.66
SW	8 3.60	10 4.50	0 0.00	0 0.00	0 0.00	0 0.00	18 8.11	1.67
WSW	4 1.80	8 3.60	3 1.35	0 0.00	0 0.00	0 0.00	15 6.76	2.46
W	8 3.60	13 5.86	6 2.70	5 2.25	0 0.00	0 0.00	32 14.41	2.82
WNW	6 2.70	3 1.35	2 .90	0 0.00	0 0.00	0 0.00	11 4.95	1.94
NW	3 1.35	4 1.80	0 0.00	0 0.00	0 0.00	0 0.00	7 3.15	2.06
NNW	5 2.25	3 1.35	1 .45	1 .45	0 0.00	0 0.00	10 4.50	2.06
N	5 2.25	4 1.80	0 0.00	0 0.00	0 0.00	0 0.00	9 4.05	1.54
CALM	1 .45						1 .45	CALM
TOTAL	101 45.50	92 41.44	23 10.36	6 2.70	0 0.00	0 0.00	222 100.00	1.95

NUMBER OF VALID OBSERVATIONS WITH PRECIPITATION	222	7.46 PCT.
NUMBER OF VALID OBSERVATIONS WITHOUT PRECIPITATION	2705	90.89 PCT.
NUMBER OF INVALID OBSERVATIONS	49	1.65 PCT.
TOTAL NUMBER OF OBSERVATIONS	2976	100.00 PCT.
TOTAL AMOUNT OF PRECIPITATION FOR DATA PERIOD		16.75 INCHES

KEY XXX NUMBER OF OCCURRENCES
XXX PERCENT OCCURRENCES

TABLE 2.3-55

PRECIPITATION WIND ROSE
JUNE 1973 - 1976

WIND SECTOR	WIND SPEED CATEGORIES (METERS PER SECOND) 0.0-1.5	1.5-3.0	3.0-5.0	5.0-7.5	7.5-10.0	>10.0	TOTAL	MEAN SPEED
NNE	7 3.18	4 1.82	0 0.00	0 0.00	0 0.00	0 0.00	11 5.00	1.43
NE	19 8.64	9 4.09	3 1.36	0 0.00	0 0.00	0 0.00	31 14.09	1.62
ENE	11 5.00	12 5.45	1 .45	0 0.00	0 0.00	0 0.00	24 10.91	1.77
E	17 7.73	10 4.55	1 .45	0 0.00	0 0.00	0 0.00	28 12.73	1.47
ESE	4 1.82	4 1.82	1 .45	0 0.00	0 0.00	0 0.00	9 4.09	1.89
SE	1 .45	8 3.64	0 0.00	0 0.00	0 0.00	0 0.00	9 4.09	1.81
SSE	7 3.18	4 1.82	0 0.00	0 0.00	0 0.00	0 0.00	11 5.00	1.30
S	8 3.64	2 .91	2 .91	0 0.00	0 0.00	0 0.00	12 5.45	1.55
SSW	4 1.82	3 1.36	0 0.00	0 0.00	0 0.00	0 0.00	7 3.18	1.59
SW	11 5.00	5 2.27	0 0.00	0 0.00	0 0.00	0 0.00	16 7.27	1.48
WSW	8 3.64	8 3.64	2 .91	0 0.00	0 0.00	0 0.00	18 8.18	1.68
W	9 4.09	9 4.09	3 1.36	0 0.00	0 0.00	0 0.00	21 9.55	1.95
WNW	4 1.82	2 .91	1 .45	0 0.00	0 0.00	0 0.00	7 3.18	1.86
NW	3 1.36	1 .45	0 0.00	0 0.00	0 0.00	0 0.00	4 1.82	1.15
NNW	5 2.27	2 .91	0 0.00	0 0.00	0 0.00	0 0.00	7 3.18	1.29
N	2 .91	1 .45	1 .45	0 0.00	0 0.00	0 0.00	4 1.82	1.97
CALM	1 .45						1 .45	CALM
TOTAL	121 55.00	84 38.18	15 6.82	0 0.00	0 0.00	0 0.00	220 100.00	1.62

NUMBER OF VALID OBSERVATIONS WITH PRECIPITATION	220	7.64 PCT.
NUMBER OF VALID OBSERVATIONS WITHOUT PRECIPITATION	2452	85.14 PCT.
NUMBER OF INVALID OBSERVATIONS	208	7.22 PCT.
TOTAL NUMBER OF OBSERVATIONS	2880	100.00 PCT.
TOTAL AMOUNT OF PRECIPITATION FOR DATA PERIOD		18.49 INCHES

KEY XXX NUMBER OF OCCURRENCES
XXX PERCENT OCCURRENCES

TABLE 2.3-56

PRECIPITATION WIND ROSE
JULY 1973 - 1976

WIND SECTOR	0.0-1.5	1.5-3.0	3.0-5.0	5.0-7.5	7.5-10.0	>10.0	TOTAL	MEAN SPEED
	WIND SPEED CATEGORIES(METERS PER SECOND)							
NNE	6 3.41	1 .57	1 .57	0 0.00	0 0.00	0 0.00	8 4.55	1.35
NE	11 6.25	6 3.41	1 .57	0 0.00	0 0.00	0 0.00	18 10.23	1.48
ENE	8 4.55	4 2.27	0 0.00	0 0.00	0 0.00	0 0.00	12 6.82	1.19
E	11 6.25	2 1.14	0 0.00	0 0.00	0 0.00	0 0.00	13 7.39	1.10
ESE	2 1.14	3 1.70	0 0.00	0 0.00	0 0.00	0 0.00	5 2.84	1.54
SE	2 1.14	1 .57	0 0.00	0 0.00	0 0.00	0 0.00	3 1.70	1.23
SSE	1 .57	1 .57	1 .57	0 0.00	0 0.00	0 0.00	3 1.70	2.07
S	3 1.70	4 2.27	0 0.00	0 0.00	0 0.00	0 0.00	7 3.98	1.83
SSW	9 5.11	5 2.84	0 0.00	0 0.00	0 0.00	0 0.00	14 7.95	1.36
SW	9 5.11	3 1.70	0 0.00	0 0.00	0 0.00	0 0.00	12 6.82	1.37
WSW	7 3.98	4 2.27	4 2.27	0 0.00	0 0.00	0 0.00	15 8.52	2.13
W	6 3.41	5 2.84	9 5.11	1 .57	0 0.00	0 0.00	21 11.93	2.86
WNW	11 6.25	0 0.00	4 2.27	0 0.00	0 0.00	0 0.00	15 8.52	1.63
NW	3 1.70	1 .57	4 2.27	0 0.00	0 0.00	0 0.00	8 4.55	2.42
NNW	5 2.84	3 1.70	3 1.70	1 .57	0 0.00	0 0.00	12 6.82	2.43
N	3 1.70	6 3.41	0 0.00	0 0.00	0 0.00	0 0.00	9 5.11	1.91
CALM	1 .57						1 .57	CALM
TOTAL	98 55.68	49 27.84	27 15.34	2 1.14	0 0.00	0 0.00	176 100.00	1.79

NUMBER OF VALID OBSERVATIONS WITH PRECIPITATION	176	5.91	PCT.
NUMBER OF VALID OBSERVATIONS WITHOUT PRECIPITATION	2798	94.02	PCT.
NUMBER OF INVALID OBSERVATIONS	2	.07	PCT.
TOTAL NUMBER OF OBSERVATIONS	2976	100.00	PCT.
TOTAL AMOUNT OF PRECIPITATION FOR DATA PERIOD		18.90	INCHES

KEY XXX NUMBER OF OCCURRENCES
XXX PERCENT OCCURRENCES

TABLE 2.3-57

PRECIPITATION WIND ROSE
AUGUST 1973 - 1976

WIND SECTOR	WIND SPEED CATEGORIES(METERS PER SECOND) 0.0-1.5	1.5-3.0	3.0-5.0	5.0-7.5	7.5-10.0	>10.0	TOTAL	MEAN SPEED
NNE	7 3.83	10 5.46	1 .55	0 0.00	0 0.00	0 0.00	18 9.84	1.79
NE	8 4.37	12 6.56	4 2.19	0 0.00	0 0.00	0 0.00	24 13.11	1.98
ENE	9 4.92	2 1.09	2 1.09	0 0.00	0 0.00	0 0.00	13 7.10	1.51
E	8 4.37	3 1.64	0 0.00	0 0.00	0 0.00	0 0.00	11 6.01	1.17
ESE	15 8.20	3 1.64	0 0.00	0 0.00	0 0.00	0 0.00	18 9.84	1.11
SE	4 2.19	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	4 2.19	.92
SSE	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0.00
S	3 1.64	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	3 1.64	.70
SSW	7 3.83	1 .55	0 0.00	0 0.00	0 0.00	0 0.00	8 4.37	1.10
SW	12 6.56	11 6.01	0 0.00	0 0.00	0 0.00	0 0.00	23 12.57	1.55
WSW	2 1.09	11 6.01	2 1.09	0 0.00	0 0.00	0 0.00	15 8.20	2.17
W	7 3.83	10 5.46	3 1.64	0 0.00	0 0.00	0 0.00	20 10.93	2.12
WNW	7 3.83	1 .55	1 .55	0 0.00	0 0.00	0 0.00	9 4.92	1.20
NW	1 .55	2 1.09	0 0.00	0 0.00	0 0.00	0 0.00	3 1.64	1.63
NNW	2 1.09	2 1.09	0 0.00	0 0.00	0 0.00	0 0.00	4 2.19	1.37
N	7 3.83	1 .55	1 .55	0 0.00	0 0.00	0 0.00	9 4.92	1.49
CALM	1 .55						1 .55	CALM
TOTAL	100 54.64	69 37.70	14 7.65	0 0.00	0 0.00	0 0.00	183 100.00	1.60

NUMBER OF VALID OBSERVATIONS WITH PRECIPITATION	183	6.15 PCT.
NUMBER OF VALID OBSERVATIONS WITHOUT PRECIPITATION	2791	93.78 PCT.
NUMBER OF INVALID OBSERVATIONS	2	.07 PCT.
TOTAL NUMBER OF OBSERVATIONS	2976	100.00 PCT.
TOTAL AMOUNT OF PRECIPITATION FOR DATA PERIOD		14.37 INCHES

KEY XXX NUMBER OF OCCURRENCES
XXX PERCENT OCCURRENCES

TABLE 2.3-58

PRECIPITATION WIND ROSE
SEPTEMBER 1973 - 1976

WIND SECTOR	WIND SPEED CATEGORIES (METERS PER SECOND) 0.0-1.5	1.5-3.0	3.0-5.0	5.0-7.5	7.5-10.0	>10.0	TOTAL	MEAN SPEED
NNE	18	18	2	1	0	0	39	1.77
	6.08	6.08	.68	.34	0.00	0.00	13.18	
NE	22	30	8	0	0	0	60	1.98
	7.43	10.14	2.70	0.00	0.00	0.00	20.27	
ENE	16	15	11	0	0	0	42	2.13
	5.41	5.07	3.72	0.00	0.00	0.00	14.19	
E	13	9	5	1	0	0	28	1.93
	4.39	3.04	1.69	.34	0.00	0.00	9.46	
ESE	6	3	0	0	0	0	9	1.38
	2.03	1.01	0.00	0.00	0.00	0.00	3.04	
SE	4	1	0	0	0	0	5	1.00
	1.35	.34	0.00	0.00	0.00	0.00	1.69	
SSE	8	0	0	0	0	0	8	.96
	2.70	0.00	0.00	0.00	0.00	0.00	2.70	
S	5	1	0	0	0	0	6	.88
	1.69	.34	0.00	0.00	0.00	0.00	2.03	
SSW	11	3	1	0	0	0	15	1.13
	3.72	1.01	.34	0.00	0.00	0.00	5.07	
SW	8	10	2	0	0	0	20	1.82
	2.70	3.38	.68	0.00	0.00	0.00	6.76	
WSW	3	10	2	0	0	0	15	2.05
	1.10	3.38	.68	0.00	0.00	0.00	5.07	
W	4	8	3	0	0	0	15	2.15
	1.35	2.70	1.01	0.00	0.00	0.00	5.07	
WNW	3	3	0	0	0	0	6	1.68
	1.01	1.01	0.00	0.00	0.00	0.00	2.03	
NW	4	2	0	0.	0	0	6	1.17
	1.35	.68	0.00	0.00	0.00	0.00	2.03	
NNW	5	0	0	0	0	0	5	1.12
	1.69	0.00	0.00	0.00	0.00	0.00	1.69	
N	5	5	0	0	0	0	10	1.48
	1.69	1.69	0.00	0.00	0.00	0.00	3.38	
CALM	7						7	CALM
	2.36						2.36	
TOTAL	142	118	34	2	0	0	296	1.74
	47.97	39.86	11.49	.68	0.00	0.00	100.00	

NUMBER OF VALID OBSERVATIONS WITH PRECIPITATION	296	10.28 PCT.
NUMBER OF VALID OBSERVATIONS WITHOUT PRECIPITATION	2574	89.28 PCT.
NUMBER OF INVALID OBSERVATIONS	10	.35 PCT.
TOTAL NUMBER OF OBSERVATIONS	2880	100.00 PCT.
TOTAL AMOUNT OF PRECIPITATION FOR DATA PERIOD		30.17 INCHES

KEY XXX NUMBER OF OCCURRENCES
XXX PERCENT OCCURRENCES

TABLE 2.3-59

PRECIPITATION WIND ROSE
OCTOBER 1973 - 1976

WIND SECTOR	WIND SPEED CATEGORIES (METERS PER SECOND) 0.0-1.5	1.5-3.0	3.0-5.0	5.0-7.5	7.5-10.0	>10.0	TOTAL	MEAN SPEED
NNE	8 / 3.52	19 / 8.37	4 / 1.76	0 / 0.00	0 / 0.00	0 / 0.00	31 / 13.66	2.18
NE	12 / 5.29	26 / 11.45	14 / 6.17	0 / 0.00	0 / 0.00	0 / 0.00	52 / 22.91	2.31
ENE	10 / 4.41	15 / 6.61	3 / 1.32	1 / .44	0 / 0.00	0 / 0.00	29 / 12.78	2.08
E	13 / 5.73	4 / 1.76	0 / 0.00	6 / 2.64	0 / 0.00	0 / 0.00	23 / 10.13	2.64
ESE	4 / 1.76	0 / 0.00	0 / 0.00	0 / 0.00	0 / 0.00	0 / 0.00	4 / 1.76	.87
SE	2 / .88	0 / 0.00	0 / 0.00	0 / 0.00	0 / 0.00	0 / 0.00	2 / .88	1.05
SSE	2 / .88	0 / 0.00	0 / 0.00	0 / 0.00	0 / 0.00	0 / 0.00	2 / .88	.80
S	5 / 2.20	1 / .44	0 / 0.00	0 / 0.00	0 / 0.00	0 / 0.00	6 / 2.64	1.03
SSW	7 / 3.08	2 / .88	0 / 0.00	0 / 0.00	0 / 0.00	0 / 0.00	9 / 3.96	1.36
SW	3 / 1.32	3 / 1.32	0 / 0.00	0 / 0.00	0 / 0.00	0 / 0.00	6 / 2.64	1.63
WSW	7 / 3.08	4 / 1.76	7 / 3.08	3 / 1.32	0 / 0.00	0 / 0.00	21 / 9.25	2.97
W	2 / .88	1 / .44	2 / .88	2 / .88	0 / 0.00	0 / 0.00	7 / 3.08	3.16
WNW	5 / 2.20	3 / 1.32	0 / 0.00	2 / .88	0 / 0.00	0 / 0.00	10 / 4.41	2.20
NW	0 / 0.00	3 / 1.32	1 / .44	0 / 0.00	0 / 0.00	0 / 0.00	4 / 1.76	2.62
NNW	4 / 1.76	1 / .44	1 / .44	0 / 0.00	0 / 0.00	0 / 0.00	6 / 2.64	1.57
N	7 / 3.08	3 / 1.32	4 / 1.76	0 / 0.00	0 / 0.00	0 / 0.00	14 / 6.17	2.11
CALM	1 / .44						1 / .44	CALM
TOTAL	92 / 40.53	85 / 37.44	36 / 15.86	14 / 6.17	0 / 0.00	0 / 0.00	227 / 100.00	2.20

NUMBER OF VALID OBSERVATIONS WITH PRECIPITATION	227	7.63 PCT.
NUMBER OF VALID OBSERVATIONS WITHOUT PRECIPITATION	2732	91.80 PCT.
NUMBER OF INVALID OBSERVATIONS	17	.57 PCT.
TOTAL NUMBER OF OBSERVATIONS	2976	100.00 PCT.
TOTAL AMOUNT OF PRECIPITATION FOR DATA PERIOD		17.55 INCHES

KEY XXX NUMBER OF OCCURRENCES
XXX PERCENT OCCURRENCES

TABLE 2.3-60

PRECIPITATION WIND ROSE
NOVEMBER 1973 - 1976

WIND SECTOR	WIND SPEED CATEGORIES(METERS PER SECOND) 0.0-1.5	1.5-3.0	3.0-5.0	5.0-7.5	7.5-10.0	>10.0	TOTAL	MEAN SPEED
NNE	5 2.91	4 2.33	0 0.00	0 0.00	0 0.00	0 0.00	9 5.23	1.64
NE	4 2.33	4 2.33	1 .58	0 0.00	0 0.00	0 0.00	9 5.23	1.74
ENE	8 4.65	4 2.33	0 0.00	0 0.00	0 0.00	0 0.00	12 6.98	1.27
E	4 2.33	1 .58	0 0.00	0 0.00	0 0.00	0 0.00	5 2.91	.96
ESE	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0.00
SE	3 1.74	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	3 1.74	.73
SSE	6 3.49	2 1.16	0 0.00	0 0.00	0 0.00	0 0.00	8 4.65	1.22
S	7 4.07	12 6.98	2 1.16	2 1.16	0 0.00	0 0.00	23 13.37	2.33
SSW	5 2.91	5 2.91	2 1.16	0 0.00	0 0.00	0 0.00	12 6.98	1.91
SW	9 5.23	4 2.33	0 0.00	0 0.00	0 0.00	0 0.00	13 7.56	1.44
WSW	8 4.65	8 4.65	14 8.14	7 4.07	0 0.00	0 0.00	37 21.51	3.33
W	7 4.07	3 1.74	10 5.81	0 0.00	0 0.00	0 0.00	20 11.63	2.46
WNW	1 .58	0 0.00	2 1.16	0 0.00	0 0.00	0 0.00	3 1.74	3.53
NW	0 0.00	0 0.00	1 .58	0 0.00	0 0.00	0 0.00	1 .58	3.80
NNW	1 .58	2 1.16	0 0.00	0 0.00	0 0.00	0 0.00	3 1.74	1.93
N	9 5.23	2 1.16	0 0.00	0 0.00	0 0.00	0 0.00	11 6.40	1.20
CALM	3 1.74						3 1.74	CALM
TOTAL	80 46.51	51 29.65	32 18.60	9 5.23	0 0.00	0 0.00	172 100.00	2.11

NUMBER OF VALID OBSERVATIONS WITH PRECIPITATION	172	5.97 PCT.
NUMBER OF VALID OBSERVATIONS WITHOUT PRECIPITATION	2552	88.61 PCT.
NUMBER OF INVALID OBSERVATIONS	156	5.42 PCT.
TOTAL NUMBER OF OBSERVATIONS	2880	100.00 PCT.
TOTAL AMOUNT OF PRECIPITATION FOR DATA PERIOD		10.63 INCHES

KEY XXX NUMBER OF OCCURRENCES
XXX PERCENT OCCURRENCES

TABLE 2.3-61

PRECIPITATION WIND ROSE
DECEMBER 1973 - 1976

WIND SECTOR	WIND SPEED CATEGORIES(METERS PER SECOND) 0.0-1.5	1.5-3.0	3.0-5.0	5.0-7.5	7.5-10.0	>10.0	TOTAL	MEAN SPEED
NNE	1 1.30	3 3.90	1 1.30	0 0.00	0 0.00	0 0.00	5 6.49	1.96
NE	0 0.00	14 18.18	2 2.60	0 0.00	0 0.00	0 0.00	16 20.78	2.37
ENE	0 0.00	2 2.60	1 1.30	0 0.00	0 0.00	0 0.00	3 3.90	2.43
E	1 1.30	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	1 1.30	1.10
ESE	2 2.60	0 0.00	0 0.00	1 1.30	0 0.00	0 0.00	3 3.90	2.40
SE	1 1.30	0 0.00	1 1.30	0 0.00	0 0.00	0 0.00	2 2.60	2.95
SSE	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0.00
S	4 5.19	6 7.79	0 0.00	0 0.00	0 0.00	0 0.00	10 12.99	1.88
SSW	4 5.19	5 6.49	1 1.30	0 0.00	0 0.00	0 0.00	10 12.99	1.85
SW	6 7.79	3 3.90	1 1.30	0 0.00	0 0.00	0 0.00	10 12.99	1.51
WSW	1 1.30	2 2.60	1 1.30	0 0.00	0 0.00	0 0.00	4 5.19	1.85
W	0 0.00	1 1.30	0 0.00	3 3.90	0 0.00	0 0.00	4 5.19	5.60
WNW	2 2.60	1 1.30	0 0.00	2 2.60	0 0.00	0 0.00	5 6.49	2.72
NW	0 0.00	1 1.30	0 0.00	0 0.00	0 0.00	0 0.00	1 1.30	1.80
NNW	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0 0.00	0.00
N	0 0.00	1 1.30	0 0.00	2 2.60	0 0.00	0 0.00	3 3.90	4.83
CALM	0 0.00						0 0.00	CALM
TOTAL	22 28.57	39 50.65	8 10.39	8 10.39	0 0.00	0 0.00	77 100.00	2.35

NUMBER OF VALID OBSERVATIONS WITH PRECIPITATION	77	2.59 PCT.
NUMBER OF VALID OBSERVATIONS WITHOUT PRECIPITATION	1522	51.14 PCT.
NUMBER OF INVALID OBSERVATIONS	1377	46.27 PCT.
TOTAL NUMBER OF OBSERVATIONS	2976	100.00 PCT.
TOTAL AMOUNT OF PRECIPITATION FOR DATA PERIOD		8.62 INCHES

KEY XXX NUMBER OF OCCURRENCES
XXX PERCENT OCCURRENCES

TABLE 2.3-62

PRECIPITATION WIND ROSE
JANUARY 1973 - DECEMBER 1976

WIND SECTOR	WIND SPEED CATEGORIES (METERS PER SECOND) 0.0-1.5	1.5-3.0	3.0-5.0	5.0-7.5	7.5-10.0	>10.0	TOTAL	MEAN SPEED
NNE	83 3.48	97 4.07	17 .71	1 .04	0 0.00	0 0.00	198 8.31	1.84
NE	99 4.15	143 6.00	68 2.85	3 .13	0 0.00	0 0.00	313 13.13	2.20
ENE	90 3.78	86 3.61	30 1.26	5 .21	0 0.00	0 0.00	211 8.85	1.99
E	92 3.86	54 2.27	22 .92	9 .38	0 0.00	0 0.00	177 7.42	1.96
ESE	60 2.52	31 1.30	8 .34	1 .04	0 0.00	0 0.00	100 4.19	1.59
SE	36 1.51	27 1.13	5 .21	3 .13	0 0.00	0 0.00	71 2.98	1.74
SSE	56 2.35	28 1.17	10 .42	3 .13	0 0.00	0 0.00	97 4.07	1.72
S	64 2.68	44 1.85	17 .71	11 .46	2 .08	0 0.00	138 5.79	2.24
SSW	74 3.10	32 1.34	20 .84	11 .46	0 0.00	0 0.00	137 5.75	2.11
SW	85 3.57	61 2.56	7 .29	1 .04	0 0.00	0 0.00	154 6.46	1.61
WSW	54 2.27	82 3.44	48 2.01	19 .80	0 0.00	0 0.00	203 8.52	2.64
W	65 2.73	56 2.35	57 2.39	23 .96	0 0.00	0 0.00	201 8.43	2.75
WNW	52 2.18	23 .96	19 .80	12 .50	1 .04	0 0.00	107 4.49	2.39
NW	32 1.34	24 1.01	13 .55	1 .04	0 0.00	0 0.00	70 2.94	1.96
NNW	38 1.59	23 .96	11 .46	5 .21	0 0.00	0 0.00	77 3.23	2.08
N	58 2.43	38 1.59	8 .34	6 .25	0 0.00	0 0.00	110 4.61	1.88
CALM	20 .84						20 .84	CALM
TOTAL	1058 44.38	849 35.61	360 15.10	114 4.78	3 .13	0 0.00	2384 100.00	2.08

NUMBER OF VALID OBSERVATIONS WITH PRECIPITATION	2384	6.80 PCT.
NUMBER OF VALID OBSERVATIONS WITHOUT PRECIPITATION	29523	84.20 PCT.
NUMBER OF INVALID OBSERVATIONS	3157	9.00 PCT.
TOTAL NUMBER OF OBSERVATIONS	35064	100.00 PCT.
TOTAL AMOUNT OF PRECIPITATION FOR DATA PERIOD		188.22 INCHES

KEY XXX NUMBER OF OCCURRENCES
XXX PERCENT OCCURRENCES

TABLE 2.3-63

HEAVY FOG (VISIBILITY 1/4 MILE OR LESS) AT AVOCA, PA. (Rev. 2.3-11)

Month	YEAR			
	1972	1973	1974	1975
January	3	3	0	3
February	0	0	1	3
March	1	3	1	2
April	2	2	0	1
May	3	3	0	0
June	2	0	1	2
July	5	2	0	0
August	0	1	2	4
September	1	2	7	3
October	3	4	2	0
November	0	1	2	3
December	8	2	0	3
Annual	28	23	16	24

TABLE 2.3-64

JOINT FREQUENTY (%) OF WIND
DIRECTION, WIND SPEED AND STABILITY (Ref. 2.3-4)

Stability Class A

Wind Speed (kts)

Sector	0-3	4-6	7-10	11-16	17-21	>21	Total
N	.0139	.0205	0	0	0	0	.0345
NNE	.0046	.0068	0	0	0	0	.0115
NE	.0230	.0000	0	0	0	0	.0230
ENE	.0046	.0068	0	0	0	0	.0115
E	.0000	0	0	0	0	0	0
ESE	.0046	.0068	0	0	0	0	.0115
SE	.0000	0	0	0	0	0	0
SSE	.0046	.0068	0	0	0	0	.0115
S	.0254	.0205	0	0	0	0	.0460
SSW	.0093	.0137	0	0	0	0	.0230
SW	.0046	.0068	0	0	0	0	.0115
WSW	.0139	.0205	0	0	0	0	.0345
W	.0186	.0274	0	0	0	0	.0460
SNW	.0093	.0137	0	0	0	0	.0230
NW	.0046	.0068	0	0	0	0	.0115
NNW	.0093	.0137	0	0	0	0	.0230
Total	.1507	.1712	0	0	0	0	

Relative frequency of occurrences of A Stability = .3219
Relative frequency of calms distributed with A Stability = .1301

Wilkes-Barre/Scranton Airport
1971-1975
Three Hourly Observations

TABLE 2.3-65

JOINT FREQUENCY (%) OF WIND
DIRECTION, WIND SPEED AND STABILITY (Ref. 2.3-4)

Stability Class B

Wind Speed (kts)

Sector	0-3	4-6	7-10	11-16	17-21	>21	.2871
N	898	.1507	.1164	0	0	0	.3569
NNE	975	.0548	.0548	0	0	0	.2071
NE	654	.0548	.0616	0	0	0	.1819
ENE	1112	.0411	.0137	0	0	0	.1660
E	768	.0274	.0342	0	0	0	.1385
ESE	516	.0205	0	0	0	0	.0721
SE	528	.0274	.0068	0	0	0	.0870
SSE	424	.0137	.0137	0	0	0	.0698
S	1675	.0890	.0205	0	0	0	.2771
SSW	898	.1507	.0959	0	0	0	.3364
SW	991	.2055	.1507	0	0	0	.4553
WSW	1773	.2877	.2123	0	0	0	.6773
W	2118	.2493	.1301	0	0	0	.6913
WNW	1449	.1918	.1164	0	0	0	.4531
NW	1449	.1986	.0890	0	0	0	.3856
NNW	748	.1096	.1027	0	0	0	.2871
Total	6507	1.9726	1.2192				

Relative frequency of occurrences of B Stability = 4.8425
Relative frequency of calms distributed with B Stability = .5274

Wilkes-Barre/Scranton Airport
1971-1975
Three Hourly Observations

TABLE 2.3-66

JOINT FREQUENCY (%) OF WIND
DIRECTION, WIND SPEED AND STABILITY (Ref. 2.3-4)

Stability Class C

Wind Speed (kts)

Sector	0-3	4-6	7-10	11-16	17-21	>21	Total
N	.0707	.2808	.3014	.0274	0	0	.6803
NNE	.0631	.1164	.0822	.0137	0	0	.2754
NE	.0488	.2055	.0959	.0137	0	0	.3639
ENE	.1069	.2671	.0685	0	0	0	.4425
E	.0712	.1233	.1233	.0068	0	0	.3246
ESE	.0132	.0616	.0411	0	0	0	.1159
SE	.0175	.0274	.0479	0	0	0	.0928
SSE	.0480	.0342	.0411	.0068	0	0	.1302
S	.0556	.1164	.1781	.0753	.0068	0	.1302
SSW	.0326	.1918	.5000	.1096	.00137	0	.8340
SW	.0413	.2055	.7877	.1507	0	0	1.1989
WSW	.0415	.3699	.6164	.1027	0	0	1.1306
W	.0714	.2877	.3630	.0342	0	0	.7563
WNW	.0681	.1712	.3493	.0959	0	0	.6846
NW	.0613	.1781	.3836	.0890	.0068	0	.7188
NNW	.0313	.1781	.4178	.1096	.0068	0	.7347
Total	.8425	2.8151	4.3973	.8356	.0342	0	

Relative frequency of occurrence of C Stability = 8.9247
Relative frequency of calms distributed with C Stability = .3082

Wilkes-Barre/Scranton Airport
1971-1975
Three Hourly Observations

TABLE 2.3-67

JOINT FREQUENCY (%) OF WIND
DIRECTION, WIND SPEED AND STABILITY (Ref. 2.3-4)

Stability Class D

Wind Speed (kts)

Sector	0-3	4-6	7-10	11-16	17-21	>21	Total
N	.4311	1.2808	1.6027	.7671	.0137	0	4.0955
NNE	.2648	.9041	1.0342	.1918	.068	0	2.4018
NE	.2677	1.0068	.8425	.3562	.0205	0	2.4937
ENE	.4100	1.4452	.9863	.4726	.0274	.0137	3.3552
E	.2648	.9041	1.0959	.3973	.0479	0	2.7100
ESE	.1397	.5616	.5616	.1233	.0205	0	1.4068
SE	.1684	.3973	.3493	.1644	.0274	.0068	1.1136
SSE	.1610	.5479	.5205	.4932	.0342	0	1.7568
S	.4506	1.5479	1.8288	1.2945	.1027	.0068	5.2314
SSW	.2837	1.2397	2.7123	1.4658	.0685	.0068	5.7768
SW	.2740	1.2192	3.3082	2.8699	.1507	.0205	7.8425
WSW	.3542	1.1164	1.2945	1.2192	.1164	.0205	4.1213
W	.2210	.9178	.7808	.8562	.1514	.0274	2.9607
WNW	.1836	.5479	1.0753	1.7740	.2949	.0411	3.9165
NW	.1995	.7055	1.8767	2.5822	.2123	.0205	5.5968
NNW	.2344	.6027	1.4589	1.5068	.0616	0	3.8645
Total	4.3082	14.9452	21.3288	16.5342	1.3630	.1644	

Relative frequency occurrence of D Stability = 58.6438
Relative frequency of calms distributed with D Stability = 1.7671

Wilkes-Barre/Scranton Airport
1971-1975
Three Hourly Observations

TABLE 2.3-68

JOINT FREQUENCY (%) OF WIND
DIRECTION, WIND SPEED AND STABILITY (2.3-4)

Stability Class E

Wind Speed (kts)

Sector	0-3	4-6	7-10	11-16	17-21	>21	Total
N	0	.3904	.4521	0	0	0	.8425
NNE	0	.2329	.0781	0	0	0	.3110
NE	0	.2534	.0479	0	0	0	.3014
ENE	0	.6574	.2945	0	0	0	.9521
E	0	.7055	.6849	0	0	0	1.3904
ESE	0	.3836	.6918	0	0	0	1.0753
SE	0	.2603	.1096	0	0	0	.3699
SSE	0	.4726	.1096	0	0	0	.5822
S	0	1.0753	.5685	0	0	0	1.6438
SSW	0	.6906	.6507	0	0	0	1.2603
SW	0	.3973	.8288	0	0	0	1.2260
WSW	0	.3836	.3014	0	0	0	.6849
W	0	.2123	.2534	0	0	0	.4658
WNW	0	.1575	.3151	0	0	0	.4726
NW	0	.2740	.5137	0	0	0	.7877
NNW	0	.2055	.4178	0	0	0	.6233
Total	0	6.6712	6.4178	0	0	0	

Relative frequency of occurrence of E Stability = 13.0890
Relative frequency of calms distributed with E Stability = 0

Wilkes-Barre/Scranton Airport
1971-1975
Three Hourly Observations

TABLE 2.3-69

JOINT FREQUENCY (%) OF WIND
DIRECTION, WIND SPEED AND STABILITY (2.3-4)

Stability Class F

Wind Speed (kts)

Sector	0-3	4-6	7-10	11-16	17-21	>21	Total
N	.0986	.4247	0	0	0	0	.5232
NNE	.0335	.2466	0	0	0	0	.2800
NE	.0813	.2945	0	0	0	0	.3758
ENE	.2463	1.1096	0	0	0	0	1.3559
E	.2782	1.3356	0	0	0	0	1.6139
ESE	.1387	.8562	0	0	0	0	.9948
SE	.1323	.3836	0	0	0	0	.5158
SSE	.1647	.6164	0	0	0	0	.7811
S	.2420	1.2466	0	0	0	0	1.4886
SSW	.1241	.6644	0	0	0	0	.7885
SW	.0875	.4726	0	0	0	0	.5601
WSW	.1028	.3836	0	0	0	0	.4864
W	.0809	.1918	0	0	0	0	.2727
WNW	.0440	.1918	0	0	0	0	.2358
NW	.0581	.3767	0	0	0	0	.4348
NNW	.0939	.3630	0	0	0	0	.4569
Total	2.0068	9.1575	0	0	0	0	

Relative frequency of occurrence of F Stability = 11.1644
Relative frequency of calms distributed with F Stability = .7877

Wilkes-Barre/Scranton Airport
1971-1975
Three Hourly Observation

TABLE 2.3-70

JOINT FREQUENCY (%) OF WIND
DIRECTION, WIND SPEED AND STABILITY (2.3-4)

Stability Class G

Wind Speed (kts)

Sector	0-3	4-6	7-10	11-16	17-21	>21	Total
N	.0976	0	0	0	0	0	.0976
NNE	.0366	0	0	0	0	0	.0366
NE	.1464	0	0	0	0	0	.1464
ENE	.3416	0	0	0	0	0	.3416
E	.6467	0	0	0	0	0	.6467
ESE	.2684	0	0	0	0	0	.2884
SE	.1342	0	0	0	0	0	.1342
SSE	.2562	0	0	0	0	0	.2562
S	.3782	0	0	0	0	0	.3782
SSW	.1586	0	0	0	0	0	.1586
SW	.0732	0	0	0	0	0	.0732
WSW	.1342	0	0	0	0	0	.1342
W	.0732	0	0	0	0	0	.0732
WNW	.0732	0	0	0	0	0	.0732
NW	.0732	0	0	0	0	0	.0732
NWN	.1220	0	0	0	0	0	.1220
Total	3.0137	0	0	0	0	0	

Relative frequency of occurrence of G Stabiity = 3.0137
Relative frequency of calms distributed with G Stability = 1.3219

Wilkes-Barre/Scranton Airport
1971-1975
Three Hourly Observations

TABLE 2.3-71

JOINT FREQUENCY (%) OF WIND
DIRECTION, WIND SPEED AND STABILITY (2.3-4)

All Stability Classes

Wind Speed (kts)

Sector	0-3	4-6	7-10	11-16	17-21	>21	Total
N	.8291	2.5479	2.4726	.7945	.0137	0	6.6579
NNE	.5271	1.5616	1.3493	.2055	.0068	0	3.6504
NE	.6243	1.8151	1.0479	.3699	.0205	0	3.8778
ENE	1.2294	3.5274	1.3630	.4726	.0274	.0137	6.335
E	1.2417	3.0959	1.9384	.4041	.0479	0	6.7280
ESE	.5947	1.8904	1.2945	.1233	.0205	0	3.9235
SE	.5050	1.0959	.5137	.1644	.0274	.0068	2.3132
SSE	.6638	1.6918	.6840	.5000	.0342	0	3.5747
S	1.3470	4.0959	2.5959	1.3699	.1092	.0068	9.5251
SSW	.7359	2.8699	3.9589	1.5753	.0685	.0068	9.2153
SW	.6186	2.5068	5.0753	3.0205	.1644	.0205	11.4063
WSW	.8230	2.5616	2.4247	1.3219	.1164	.0205	7.2682
W	.6589	1.9863	1.5274	.8904	.1574	.0274	5.2480
WNW	.5098	1.2740	1.8562	1.8699	.2945	.0411	5.8455
NW	.5091	1.7397	2.8630	2.6712	.2192	.0205	8.0228
NNW	.5551	1.4726	2.3973	1.6164	.0685	0	6.1099
Total	11.9726	35.7328	33.3630	17.3699	1.3973	.1644	

Relative frequency of occurrence of observations = 100
Relative frequency of calms distributed above = 4.8425

Wilkes-Barre/Scranton Airport
1971-1975
Three Hourly Observations

TABLE 2.3-72

MIXING HEIGHTS (meters)

Time	Spring	Summer	Autumn	Winter
4 AM-9 PM	706	510	562	774
4 PM-9 PM	1750	1816	1306	979
Other Times	1228	1163	934	877

TABLE 2.3-73

Heights of Meteorological Sensors

200 Foot Primary Tower				
Parameter	60M (849.8' MSL)	10M (685.8' MSL)	SFC (653.21' MSL)	10 Meter Nescopeck[1] Tower 10 Meters
Wind Speed	1	1		1
Wind Direction	1	1		1
Ambient Temperature		1		1[2]
Dew Point Temperature		1		
Temperature Difference (using the 10 m temperature as reference)	2	2		
Precipitation			1	
10 Meter Backup Tower				
Parameter	10 Meters (622.8' MSL)			
Wind Speed	1			
Wind Direction	1			

1. Nescopeck supplemental tower added per PLA-2467.
2. Measurements made at 8' height of tower.

TABLE 2.3-74
METEOROLOGICAL DATA RECOVERY RATES
1998 THROUGH 2003

	1999	2000	2001	2002	2003	5 YR AVG
Wind Speed 10m Primary	99.7	99.8	100.0	99.5	99.6	99.7
Wind Speed 60m Primary	99.7	99.4	99.4	99.5	99.2	99.4
Wind Direction 10m Primary	99.7	100.0	100.0	99.4	99.7	99.8
Wind Direction 60m Primary	99.6	100.0	99.3	99.5	99.7	99.6
Delta Temperature 60-10m A Primary	99.6	99.8	99.3	99.0	99.1	99.4
Temperature 10m Primary	99.6	99.7	99.9	99.6	99.0	99.6
Dew Point 10m Primary	99.3	87.2	98.8	98.6	98.8	96.5
Precipitation	99.7	100.0	100.0	100.0	100.0	99.9
<u>Composite</u>						
Wind Speed 10m, Wind Direction 10m, Delta Temperature 60-10m	99.5	99.7	99.3	99.0	99.0	99.3
Wind Speed 60m, Wind Direction 60m Delta Temperature 60-10m	99.5	99.3	99.2	99.0	98.6	99.1

TABLE 2.3-75

Joint Frequency Distribution

Hours at Each Wind Speed and Direction

Period of Record = 01/01/99 1:00 - 12/31/03 23:00 Total Period

Elevation: Speed: 10M SPD Direction: 10M WD Lapse: DT60-10

Stability Class A Delta Temperature Extremely Unstable

Wind Speed (m/s)

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	1	0	0	2	3	11	28	4	0	0	0	0	49
NNE	0	0	0	3	1	28	58	4	0	0	0	0	94
NE	0	0	0	7	15	45	23	0	0	0	0	0	90
ENE	0	0	1	8	17	11	4	0	0	0	0	0	41
E	0	0	4	26	8	4	1	0	0	0	0	0	43
ESE	0	1	7	12	7	8	6	0	0	0	0	0	41
SE	0	0	2	6	14	33	20	0	0	0	0	0	75
SSE	0	0	2	6	19	36	14	1	0	0	0	0	78
S	0	0	2	10	28	62	63	1	0	0	0	0	166
SSW	0	0	0	12	38	105	88	4	0	0	0	0	247
SW	0	0	1	10	38	177	261	28	1	0	0	0	516
WSW	0	0	0	4	7	29	125	35	2	0	0	0	202
W	0	0	0	1	2	4	48	5	0	0	0	0	60
WNW	0	0	0	1	2	6	12	0	0	0	0	0	21
NW	0	0	0	0	1	0	6	3	0	0	0	0	10
NNW	0	0	0	1	0	1	12	4	0	0	0	0	18
Totals	1	1	19	109	200	560	769	89	3	0	0	0	1751

Number of Calm Hours for this Table	18
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	297
Number of Valid Hours for this Table	1751
Total Hours for the Period	43823

TABLE 2.3-76

Joint Frequency Distribution

Hours at Each Wind Speed and Direction

Period of Record = 01/01/99 1:00 - 12/31/03 23:00 Total Period
Elevation: Speed: 10M SPD Direction: 10M WD Lapse: DT60-10

Stability Class B Delta Temperature Moderately Unstable

Wind Speed (m/s)

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	0	1	0	1	5	9	42	8	0	0	0	0	66
NNE	0	0	0	3	20	47	34	3	0	0	0	0	107
NE	0	0	0	12	16	46	24	0	0	0	0	0	98
ENE	0	0	1	24	13	10	2	0	0	0	0	0	50
E	0	0	9	15	7	10	3	0	0	0	0	0	44
ESE	0	0	4	11	14	5	5	0	0	0	0	0	39
SE	0	0	2	10	11	16	11	0	0	0	0	0	50
SSE	0	0	0	4	11	11	6	0	0	0	0	0	32
S	0	0	2	11	14	38	21	0	0	0	0	0	86
SSW	0	0	1	11	39	55	28	3	0	0	0	0	137
SW	0	0	0	4	36	105	175	31	4	0	0	0	355
WSW	0	0	1	1	7	23	100	43	2	0	0	0	177
W	0	0	0	0	1	8	34	4	0	0	0	0	47
WNW	0	0	0	0	1	1	18	1	0	0	0	0	21
NW	0	0	0	0	1	5	10	2	0	0	0	0	18
NNW	0	0	0	0	4	8	19	3	2	0	0	0	36
Totals	0	1	20	107	200	397	532	98	8	0	0	0	1363

Number of Calm Hours for this Table	18
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	297
Number of Valid Hours for this Table	1363
Total Hours for the Period	43823

TABLE 2.3-77

Joint Frequency Distribution

Hours at Each Wind Speed and Direction

Period of Record = 01/01/99 1:00 - 12/31/03 23:00 Total Period

Elevation: Speed: 10M SPD Direction: 10M WD Lapse: DT60-10

Stability Class C Delta Temperature Slightly Unstable

Wind Speed (m/s)

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	0	0	0	3	7	23	73	6	0	0	0	0	112
NNE	0	0	1	8	21	52	59	6	0	0	0	0	147
NE	0	0	0	12	24	45	20	0	0	0	0	0	101
ENE	0	0	2	25	20	14	4	0	0	0	0	0	65
E	0	1	7	19	8	8	3	0	0	0	0	0	46
ESE	0	1	6	15	16	10	3	0	0	0	0	0	51
SE	1	0	7	9	17	13	11	0	0	0	0	0	58
SSE	0	1	4	12	18	21	12	0	0	0	0	0	68
S	0	0	4	26	28	50	28	0	0	0	0	0	136
SSW	0	0	1	24	35	76	17	2	0	0	0	0	155
SW	0	0	1	18	43	146	186	37	1	0	0	0	432
WSW	0	0	1	3	11	38	139	61	7	0	0	0	260
W	0	0	0	8	7	7	51	21	1	0	0	0	95
WNW	0	0	0	0	2	14	31	1	0	0	0	0	48
NW	0	0	1	0	3	9	30	4	0	0	0	0	47
NNW	0	0	0	1	1	19	31	16	2	0	0	0	70
Totals	1	3	35	183	261	545	698	154	11	0	0	0	1891

Number of Calm Hours for this Table	18
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	297
Number of Valid Hours for this Table	1891
Total Hours for the Period	43823

TABLE 2.3-78

Joint Frequency Distribution

Hours at Each Wind Speed and Direction

Period of Record = 01/01/99 1:00 - 12/31/03 23:00 Total Period
Elevation: Speed: 10M SPD Direction: 10M WD Lapse: DT60-10

Stability Class D Delta Temperature Neutral

Wind Speed (m/s)

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	2	7	23	74	119	433	716	98	0	0	0	0	1472
NNE	1	16	64	214	267	519	324	15	0	0	0	0	1420
NE	9	33	119	276	281	418	164	6	0	0	0	0	1306
ENE	8	51	130	219	153	158	54	3	0	0	0	0	776
E	10	85	158	202	124	119	32	5	0	0	0	0	735
ESE	15	67	144	144	117	146	58	7	3	1	0	0	702
SE	7	53	110	219	169	200	121	14	1	0	0	0	894
SSE	10	38	74	163	158	179	89	9	1	0	0	0	721
S	3	26	84	210	204	257	106	7	0	0	0	0	897
SSW	0	15	54	220	242	368	139	2	0	0	0	0	1040
SW	1	8	29	193	242	594	864	170	7	0	0	0	2108
WSW	0	3	15	87	132	254	627	425	90	1	0	0	1634
W	0	1	6	38	80	196	396	172	35	0	0	0	924
WNW	0	2	4	23	49	157	314	108	19	0	0	0	676
NW	1	2	11	31	45	225	632	178	6	0	0	0	1131
NNW	0	4	8	33	52	268	752	280	8	0	0	0	1405
Totals	67	411	1033	2346	2434	4491	5388	1499	170	2	0	0	17841

Number of Calm Hours for this Table	18
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	297
Number of Valid Hours for this Table	17841
Total Hours for the Period	43823

TABLE 2.3-79

Joint Frequency Distribution

Hours at Each Wind Speed and Direction

Period of Record = 01/01/99 1:00 - 12/31/03 23:00 Total Period
Elevation: Speed: 10M SPD Direction: 10M WD Lapse: DT60-10

Stability Class E Delta Temperature Slightly Stable

Wind Speed (m/s)

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	1	14	28	90	138	144	53	2	0	0	0	0	470
NNE	3	33	112	324	269	254	79	1	0	0	0	0	1075
NE	13	124	289	551	197	159	54	0	0	0	0	0	1387
ENE	20	279	467	493	87	29	7	3	0	0	0	0	1385
E	41	378	361	154	33	29	5	0	0	0	0	0	1001
ESE	45	265	207	99	30	24	8	5	0	0	0	0	683
SE	37	201	229	152	48	50	23	10	4	0	0	0	754
SSE	19	109	183	213	95	63	40	6	0	0	0	0	728
S	10	75	216	413	193	151	49	18	0	0	0	0	1125
SSW	3	42	126	397	334	309	84	4	0	0	0	0	1299
SW	2	14	39	188	209	343	210	10	1	0	0	0	1016
WSW	0	4	12	64	80	83	61	14	3	0	0	0	321
W	2	1	9	36	33	37	19	4	0	0	0	0	141
WNW	0	2	7	16	23	34	13	0	0	0	0	0	95
NW	1	2	4	19	42	87	24	2	0	0	0	0	181
NNW	0	5	6	21	41	116	35	2	0	0	0	0	226
Totals	197	1548	2295	3230	1852	1912	764	81	8	0	0	0	11887

Number of Calm Hours for this Table	18
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	297
Number of Valid Hours for this Table	11887
Total Hours for the Period	43823

TABLE 2.3-80

Joint Frequency Distribution

Hours at Each Wind Speed and Direction
Period of Record = 01/01/99 1:00 - 12/31/03 23:00 Total Period
Elevation: Speed: 10M SPD Direction: 10M WD Lapse: DT60-10

Stability Class F Delta Temperature Moderately Stable

Wind Speed (m/s)

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	1	5	6	25	11	5	2	0	0	0	0	0	55
NNE	5	21	34	66	32	9	1	0	0	0	0	0	168
NE	13	92	198	257	50	1	1	0	0	0	0	0	612
ENE	21	361	806	994	149	6	0	0	0	0	0	0	2337
E	44	371	356	149	9	0	0	0	0	0	0	0	929
ESE	23	160	122	15	0	0	0	0	0	0	0	0	320
SE	14	82	82	25	2	0	0	0	0	0	0	0	205
SSE	8	32	78	49	6	1	0	0	0	0	0	0	174
S	2	29	78	121	14	1	1	0	0	0	0	0	246
SSW	1	16	24	81	37	4	1	0	0	0	0	0	164
SW	1	4	15	38	21	19	2	0	0	0	0	0	100
WSW	0	0	5	6	4	3	1	0	0	0	0	0	19
W	2	2	3	5	0	1	0	0	0	0	0	0	13
WNW	1	0	0	2	2	1	0	0	0	0	0	0	6
NW	0	0	2	1	1	2	1	0	0	0	0	0	7
NNW	1	0	3	2	5	2	2	0	0	0	0	0	15
Totals	137	1175	1812	1836	343	55	12	0	0	0	0	0	5370

Number of Calm Hours for this Table	18
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	297
Number of Valid Hours for this Table	5370
Total Hours for the Period	43823

TABLE 2.3-81

Joint Frequency Distribution

Hours at Each Wind Speed and Direction

Period of Record = 01/01/99 1:00 - 12/31/03 23:00 Total Period

Elevation: Speed: 10M SPD Direction: 10M WD Lapse: DT60-10

Stability Class G Delta Temperature Extremely Stable

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	1	2	2	3	1	0	0	0	0	0	0	0	9
NNE	1	16	17	17	3	1	0	0	0	0	0	0	55
NE	2	71	168	162	19	1	0	0	0	0	0	0	423
ENE	8	167	690	1065	186	4	0	0	0	0	0	0	2120
E	13	120	219	102	3	0	0	0	0	0	0	0	457
ESE	4	63	55	9	0	0	0	0	0	0	0	0	131
SE	3	31	35	12	2	1	0	0	0	0	0	0	84
SSE	0	14	23	15	3	0	0	0	0	0	0	0	55
S	0	5	12	17	0	0	0	0	0	0	0	0	34
SSW	1	2	8	7	5	1	0	0	0	0	0	0	24
SW	0	2	3	1	0	1	0	0	0	0	0	0	7
WSW	0	1	0	0	0	0	0	0	0	0	0	0	1
W	0	0	0	0	0	1	0	0	0	0	0	0	1
WNW	0	0	0	0	0	0	0	0	0	0	0	0	0
NW	0	1	0	0	0	0	0	0	0	0	0	0	1
NNW	0	2	0	1	0	0	0	0	0	0	0	0	3
Totals	33	497	1232	1411	222	10	0	0	0	0	0	0	3405

Number of Calm Hours for this Table	18
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	297
Number of Valid Hours for this Table	3405
Total Hours for the Period	43823

Table 2.3-82

Joint Frequency Distribution

Hours at Each Wind Speed and Direction
Period of Record = 01/01/99 1:00 - 12/31/03 23:00 Total Period
Elevation: Speed: 10M SPD Direction: 10M WD Lapse: DT60-10

Summary of All Stability Classes Delta Temperature

Wind Speed (m/s)

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	6	29	59	198	284	625	914	118	0	0	0	0	2233
NNE	10	86	228	635	613	910	555	29	0	0	0	0	3066
NE	37	320	774	1277	602	715	286	60	0	0	0	0	4017
ENE	57	858	2097	2828	625	232	71	6	0	0	0	0	6774
E	108	955	1114	667	192	170	44	5	0	0	0	0	3255
ESE	87	557	545	305	184	193	80	12	3	1	0	0	1967
SE	62	367	467	433	263	313	186	24	5	0	0	0	2120
SSE	37	194	364	462	310	311	161	16	1	0	0	0	1856
S	15	135	398	808	481	559	268	26	0	0	0	0	2690
SSW	5	75	214	752	730	918	357	1	0	0	0	0	3066
SW	4	28	88	452	589	1385	1698	276	14	0	0	0	4534
WSW	0	8	34	165	241	430	1053	578	104	1	0	0	2614
W	4	4	18	88	123	254	548	206	36	0	0	0	1281
WNW	1	4	11	42	79	213	388	110	19	0	0	0	867
NW	2	5	18	51	93	328	703	189	6	0	0	0	1395
NNW	1	11	17	59	103	414	851	305	12	0	0	0	1773
Totals	436	3636	6446	9222	5512	7970	8163	1921	200	2	0	0	43508

Number of Calm Hours for this Table	18
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	297
Number of Valid Hours for this Table	1751
Total Hours for the Period	43823

TABLE 2.3-84

Joint Frequency Distribution

Hours at Each Wind Speed and Direction

Period of Record = 01/01/99 1:00- 12/31/03 23:00 Total Period
Elevation: Speed: 60M SPD Direction: 60M WD Lapse:DT60-10

Stability Class A Delta Temperature Extremely Unstable

Wind Speed (m/s)

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	0	0	0	0	1	2	30	12	4	0	0	0	49
NNE	0	0	0	0	1	14	45	32	7	0	0	0	99
NE	1	0	1	11	16	23	47	11	3	0	0	0	113
ENE	0	2	2	11	12	9	3	0	1	0	0	0	40
E	0	0	2	5	9	5	1	0	1	0	0	0	23
ESE	0	0	2	6	4	8	5	5	0	0	0	0	30
SE	0	0	1	3	4	7	27	18	1	0	0	0	61
SSE	0	0	0	2	3	11	33	17	1	0	0	0	67
S	0	1	3	5	6	24	46	55	10	1	0	0	151
SSW	0	0	0	6	13	49	85	62	22	0	0	0	237
SW	0	0	0	8	12	49	239	169	28	1	0	0	506
WSW	0	0	0	2	2	13	85	121	34	3	0	0	260
W	0	0	0	1	2	0	26	40	3	0	0	0	72
WNW	0	0	0	0	1	1	5	9	0	0	0	0	16
NW	0	0	1	0	0	0	1	6	0	0	0	0	8
NNW	0	0	0	0	0	1	7	9	0	0	0	0	17
Totals	1	3	12	60	86	216	685	566	115	5	0	0	1749

Number of Calm Hours for this Table	3
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	385
Number of Valid Hours for this Table	1749
Total Hours for the Period	43823

TABLE 2.3-85

Joint Frequency Distribution

Hours at Each Wind Speed and Direction

Period of Record = 01/01/99 1:00- 12/31/03 23:00 Total Period
Elevation: Speed: 60M SPD Direction: 60M WD Lapse: DT60-10

Stability Class B Delta Temperature Moderately Unstable

Wind Speed (m/s)

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	0	0	0	1	3	8	27	27	3	0	0	0	69
NNE	0	0	0	4	10	22	57	19	5	0	0	0	117
NE	0	0	4	11	22	30	33	12	1	0	0	0	113
ENE	0	0	2	9	8	15	5	1	0	0	0	0	40
E	0	0	3	7	4	9	4	2	1	0	0	0	30
ESE	0	0	2	4	8	5	11	2	0	0	0	0	32
SE	0	0	1	6	0	10	12	10	2	0	0	0	41
SSE	0	0	0	0	1	4	20	5	0	0	0	0	30
S	0	0	0	3	6	10	28	22	3	0	0	0	72
SSW	0	0	2	2	11	31	47	25	13	1	0	0	132
SW	0	0	0	2	10	51	145	101	28	4	0	0	341
WSW	0	0	0	0	2	6	67	86	55	1	0	0	217
W	0	0	0	0	0	3	21	28	6	0	0	0	58
WNW	0	0	0	0	0	2	8	7	0	0	0	0	17
NW	0	0	0	0	3	2	13	5	1	0	0	0	24
NNW	0	0	0	0	1	3	10	12	1	1	0	0	28
Totals	0	0	14	49	89	211	508	364	119	7	0	0	1361

Number of Calm Hours for this Table	3
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	385
Number of Valid Hours for this Table	1361
Total Hours for the Period	43823

TABLE 2.3-86

Joint Frequency Distribution

Hours at Each Wind Speed and Direction

Period of Record = 01/01/99 1:00- 12/31/03 23:00 Total Period

Elevation: Speed: 60M SPD Direction: 60M WD Lapse: DT60-10

Stability Class C Delta Temperature Slightly Unstable

Wind Speed (m/s)

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	0	0	0	2	6	9	45	43	1	0	0	0	106
NNE	0	0	0	9	18	30	74	37	5	0	0	0	173
NE	0	0	4	10	8	31	32	11	1	0	0	0	97
ENE	0	2	6	13	11	21	7	4	0	0	0	0	64
E	0	1	3	13	9	5	5	1	0	0	0	0	37
ESE	0	1	1	1	4	10	11	3	0	0	0	0	31
SE	0	0	3	6	7	14	16	8	2	0	0	0	56
SSE	0	1	1	3	7	10	15	7	3	0	0	0	47
S	0	0	5	9	11	23	37	23	6	0	0	0	114
SSW	0	0	0	15	18	52	57	27	9	0	0	0	178
SW	0	0	0	3	15	69	187	77	31	1	0	0	383
WSW	0	0	0	1	7	18	88	140	78	7	0	0	339
W	0	0	0	1	2	4	24	51	20	1	0	0	103
WNW	0	1	0	1	0	5	28	12	1	0	0	0	48
NW	0	0	0	0	0	3	26	12	2	0	0	0	43
NNW	0	0	0	87	124	310	683	480	165	9	0	0	68
Totals	0	6	23	87	124	310	683	480	165	9	0	0	1887

Number of Calm Hours for this Table	3
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	385
Number of Valid Hours for this Table	1887
Total Hours for the Period	43823

TABLE 2.3-87

Joint Frequency Distribution

Hours at Each Wind Speed and Direction

Period of Record = 01/01/99 1:00 - 12/31/03 23:00 Total Period
Elevation: Speed: 60M SPD Direction: 60M WD Lapse: DT60-10

Stability Class D Delta Temperature Neutral

Wind Speed (m/s)

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	0	6	5	45	52	159	578	343	50	0	0	0	1238
NNE	4	6	33	124	154	289	584	283	46	2	0	0	1525
NE	1	25	82	183	131	288	455	139	16	1	0	0	1321
ENE	3	26	60	106	83	169	143	35	3	3	0	0	631
E	4	28	50	71	60	146	156	26	14	1	0	0	556
ESE	1	26	39	52	51	107	170	54	12	1	3	0	516
SE	2	20	45	75	53	130	241	82	22	8	0	0	678
SSE	2	13	41	88	59	115	258	88	26	7	0	0	697
S	1	15	39	115	76	105	221	122	44	5	0	0	743
SSW	1	9	32	131	156	202	255	192	66	4	0	0	1048
SW	1	2	23	112	187	405	625	403	110	5	0	0	1873
WSW	0	2	9	26	70	177	583	849	678	99	9	0	2502
W	2	5	2	7	23	77	361	423	230	34	5	0	1169
WNW	0	1	3	8	13	75	329	238	107	7	0	0	781
NW	1	2	3	5	9	81	531	482	98	1	0	0	1213
NNW	0	2	3	23	18	86	550	506	133	0	0	0	1321
Totals	23	188	469	1171	1195	2611	6040	4265	1655	178	17	0	17812

Number of Calm Hours for this Table	3
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	385
Number of Valid Hours for this Table	17812
Total Hours for the Period	43823

TABLE 2.3-88

Joint Frequency Distribution

Hours at Each Wind Speed and Direction

Period of Record = 01/01/99 1:00 - 12/31/03 23:00 Total Period
Elevation: Speed: 60M SPDDirection: 60M WD Lapse: DT60-10

Stability Class E Delta Temperature Slightly Stable

Wind Speed (m/s)

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	0	16	28	57	93	197	190	25	1	0	0	0	607
NNE	2	19	66	254	364	482	331	101	12	0	0	0	1631
NE	4	51	139	373	200	277	277	82	6	0	0	0	1409
ENE	9	50	112	139	84	140	73	7	1	3	0	0	618
E	9	54	87	107	61	71	76	10	2	1	0	0	478
ESE	5	48	70	78	36	62	63	11	7	2	0	0	382
SE	6	33	61	103	63	68	85	30	16	7	0	0	472
SSE	4	37	71	130	61	132	163	38	22	5	0	0	663
S	5	24	75	141	108	154	242	81	42	14	1	0	887
SSW	5	8	47	126	136	183	434	190	59	2	1	0	1191
SW	1	14	33	108	136	324	579	213	29	1	1	0	1439
WSW	0	6	17	42	85	157	419	384	57	3	0	0	1170
W	0	3	11	22	26	57	81	29	8	1	0	0	238
WNW	0	2	2	6	11	68	77	6	0	0	0	0	172
NW	0	3	5	15	13	49	168	35	3	0	0	0	291
NNW	1	6	10	14	14	54	118	15	0	1	0	0	233
Totals	51	374	834	1715	1491	2475	3376	1257	265	40	3	0	11881

Number of Calm Hours for this Table	3
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	385
Number of Valid Hours for this Table	11881
Total Hours for the Period	43823

TABLE 2.3-89

Joint Frequency Distribution

Hours at Each Wind Speed and Direction

Period of Record = 01/01/99 1:00 - 12/31/03 23:00 Total Period
Elevation: Speed: 60M SPD Direction: 60M WD Lapse: DT60-10

Stability Class F Delta Temperature Moderately Stable

Wind Speed (m/s)

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	2	3	13	47	82	176	37	3	0	0	0	0	363
NNE	0	16	33	269	503	677	100	0	0	0	0	0	1598
NE	7	28	97	340	243	154	34	0	0	0	0	0	903
ENE	8	28	87	132	47	20	3	0	0	0	0	0	325
E	7	20	65	101	36	15	4	1	0	0	0	0	249
ESE	3	26	55	64	23	9	2	0	0	0	0	0	182
SE	1	20	45	90	24	13	5	0	0	0	0	0	198
SSE	1	8	29	87	30	22	12	1	0	0	0	0	190
S	1	8	17	82	53	57	33	1	0	0	0	0	252
SSW	1	4	16	41	72	97	80	8	1	0	0	0	320
SW	0	4	3	31	44	128	142	16	1	0	0	0	369
WSW	0	2	5	8	14	22	124	65	2	0	0	0	242
W	0	3	3	6	5	11	6	0	0	0	0	0	34
WNW	0	2	2	2	7	8	5	0	0	0	0	0	26
NW	0	1	2	5	7	20	11	2	0	0	0	0	48
NNW	0	1	4	10	11	14	10	1	0	0	0	0	51
Totals	31	174	476	1315	1201	1443	608	98	4	0	0	0	5350

Number of Calm Hours for this Table	3
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	385
Number of Valid Hours for this Table	5350
Total Hours for the Period	43823

TABLE 2.3-90

Joint Frequency Distribution

Hours at Each Wind Speed and Direction
Period of Record = 01/01/99 1:00 - 12/31/03 23:00 Total Period
Elevation:Speed: 60M SPD Direction: 60M WD Lapse: DT60-10

Stability Class G Delta Temperature Extremely Stable

Wind Speed (m/s)

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	0	3	6	29	62	163	42	0	0	0	0	0	305
NNE	1	6	21	169	418	365	39	0	0	0	0	0	1019
NE	3	11	53	226	208	108	12	0	0	0	0	0	621
ENE	0	10	53	105	45	20	3	0	0	0	0	0	236
E	1	8	50	84	10	10	1	0	0	0	0	0	164
ESE	2	14	31	68	19	5	3	0	0	0	0	0	142
SE	0	4	26	66	16	12	1	0	0	0	0	0	125
SSE	1	5	19	47	32	10	1	1	0	0	0	0	116
S	0	3	11	41	47	60	18	2	0	0	0	0	182
SSW	0	1	4	23	31	72	49	6	0	0	0	0	186
SW	0	0	5	16	24	70	48	5	0	0	0	0	168
WSW	0	0	3	4	8	9	25	13	1	0	0	0	63
W	0	0	1	3	1	1	1	0	0	0	0	0	7
WNW	0	0	1	3	0	6	3	0	0	0	0	0	13
NW	1	1	0	4	3	8	6	0	0	0	0	0	23
NNW	0	0	0	4	0	14	7	0	0	0	0	0	25
Totals	9	66	284	892	924	933	259	27	1	0	0	0	3395

Number of Calm Hours for this Table	3
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	385
Number of Valid Hours for this Table	3395
Total Hours for the Period	43823

TABLE 2.3-91

Joint Frequency Distribution

Hours at Each Wind Speed and Direction
Period of Record = 01/01/99 1:00- 12/31/03 23:00 Total Period
Elevation: Speed: 60M SPD Direction: 60M WD Lapse: DT60-10

Summary of All Stability Classes Delta Temperature

Wind Direction From	0.23 0.50	0.51- 0.75	0.76- 1.0	1.1- 1.5	1.6- 2.0	2.1- 3.0	3.1- 5.0	5.1- 7.0	7.1- 10.0	10.1- 13.0	13.1- 18.0	> 18.0	Total
N	2	28	52	181	299	714	949	453	59	0	0	0	2737
NNE	7	47	153	829	1468	1879	1230	472	75	2	0	0	6162
NE	16	115	380	1154	828	911	890	255	27	1	0	0	4577
ENE	20	118	322	515	290	394	237	47	5	6	0	0	1954
E	21	111	260	388	189	261	247	40	18	2	0	0	1537
ESE	11	115	200	273	145	206	265	75	19	3	3	0	1315
SE	9	77	182	349	167	254	387	148	43	15	0	0	1631
SSE	8	64	161	357	193	304	502	157	52	12	0	0	1810
S	7	51	150	396	307	433	625	306	105	20	1	0	2401
SSW	7	22	101	344	437	686	1007	510	170	7	1	0	3292
SW	2	20	64	280	428	1096	1965	984	227	12	1	0	5079
WSW	0	10	34	83	188	402	1391	1658	905	113	9	0	4793
W	2	11	17	40	59	153	520	571	267	36	5	0	1681
WNW	0	6	8	20	32	165	455	272	108	7	0	0	1073
NW	2	7	11	29	35	163	756	542	104	1	0	0	1650
NNW	1	9	17	51	45	178	733	567	140	2	0	0	1743
Totals	115	811	2112	5289	5110	8199	12159	7057	2324	239	20	0	43435

Number of Calm Hours for this Table	3
Number of Variable Direction Hours for this Table	0
Number of Invalid Hours	385
Number of Valid Hours for this Table	43435
Total Hours for the Period	43823

Table 2.3-92

SUMMARY OF SHORT-TERM X/Q (SEC/M3) RESULTS AT 549 METER EAB				
Period of Record	Data Source	1-hour 5% Direction Independent	1-hour 5% Direction Dependent	1-hour 5% Direction Independent
1999	SSES Tower	6.5E-4	8.4E-4	1.2E-4
2000	SSES Tower	6.5E-4	8.2E-4	1.3E-4
2001	SSES Tower	6.6E-4	8.3E-4	1.4E-4
2002	SSES Tower	6.6E-4	8.4E-4	1.2E-4
2003	SSES Tower	4.9E-4	7.9E-4	1.2E-4
5-Year Combined	SSES Tower	6.5E-4	8.3E-4	1.3E-4

Table 2.3-93

1999 Probability values for 1 hour at SSES EAB

Probability that the X/Q is Greater than the Adjacent Quantized Level

QUANTIZED LEVEL	DIRECTION															
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
0.00E+00	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
1.00E-10	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
3.50E-09	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
1.00E-08	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
2.50E-08	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
7.00E-08	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
1.00E-07	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
1.50E-07	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
2.20E-07	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
3.20E-07	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
4.80E-07	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
7.00E-07	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
1.00E-06	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
1.50E-06	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
2.00E-06	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
3.00E-06	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.0626	0.0313	0.0177	0.0306	0.0393
4.00E-06	0.0616	0.0747	0.0962	0.1615	0.0758	0.0447	0.0445	0.0366	0.0519	0.0665	0.1017	0.0611	0.0307	0.0177	0.0306	0.0393
5.00E-06	0.0616	0.0739	0.096	0.1615	0.0758	0.0446	0.0444	0.0366	0.0518	0.0663	0.0997	0.0572	0.0289	0.0169	0.0304	0.0393
7.00E-06	0.0604	0.0725	0.0947	0.1612	0.0758	0.0446	0.0436	0.036	0.0502	0.063	0.092	0.055	0.0272	0.0166	0.0304	0.0391
8.50E-06	0.0604	0.0719	0.0937	0.1611	0.0757	0.0444	0.0422	0.0358	0.049	0.0614	0.0905	0.0547	0.0272	0.0166	0.0304	0.039
1.00E-05	0.0602	0.0717	0.0935	0.1608	0.0755	0.0444	0.0416	0.0349	0.0481	0.0609	0.0889	0.054	0.0272	0.0166	0.0303	0.039
1.50E-05	0.0596	0.0708	0.0921	0.1601	0.0751	0.0438	0.0414	0.0347	0.0475	0.0602	0.0861	0.0529	0.0263	0.0164	0.03	0.0389
2.00E-05	0.0592	0.0695	0.0916	0.16	0.0743	0.0433	0.0411	0.0347	0.0469	0.0594	0.0836	0.0501	0.0258	0.0159	0.0298	0.0386
2.50E-05	0.0584	0.0688	0.0901	0.1599	0.0741	0.0429	0.0407	0.0344	0.0459	0.0587	0.082	0.049	0.0251	0.0157	0.0297	0.0382
3.00E-05	0.0573	0.0673	0.089	0.1594	0.074	0.0424	0.0404	0.0337	0.0456	0.0579	0.0807	0.0469	0.0237	0.0157	0.0292	0.0381
3.50E-05	0.0571	0.0662	0.0886	0.1592	0.0739	0.0422	0.0403	0.0337	0.0455	0.0575	0.0795	0.0456	0.0233	0.0155	0.029	0.0373
4.00E-05	0.0564	0.0655	0.0882	0.1587	0.0737	0.0421	0.0392	0.0336	0.0453	0.0565	0.078	0.0417	0.0225	0.015	0.0286	0.0347
5.00E-05	0.051	0.0633	0.0872	0.158	0.0728	0.0419	0.0385	0.0326	0.0446	0.0555	0.0739	0.0333	0.0193	0.0136	0.0257	0.0281
6.00E-05	0.0409	0.0603	0.0861	0.1577	0.0725	0.0417	0.0373	0.0325	0.0442	0.0545	0.0653	0.0255	0.015	0.0112	0.0205	0.0212
7.00E-05	0.0325	0.0555	0.0842	0.1569	0.0721	0.0412	0.0365	0.0308	0.0429	0.0534	0.0581	0.0214	0.0119	0.0088	0.0163	0.0164
8.50E-05	0.0214	0.0432	0.0763	0.1538	0.07	0.0383	0.0336	0.0286	0.0397	0.0467	0.0455	0.0167	0.0084	0.0048	0.0095	0.0099
1.00E-04	0.0153	0.0338	0.0679	0.1506	0.0683	0.0359	0.0314	0.0248	0.0351	0.0398	0.038	0.0119	0.0062	0.0036	0.0067	0.0067
1.30E-04	0.0087	0.0201	0.0534	0.1447	0.0647	0.0337	0.0257	0.0195	0.0267	0.0266	0.0185	0.0056	0.0033	0.0017	0.0021	0.0021
1.70E-04	0.0056	0.0144	0.046	0.1399	0.0612	0.031	0.0225	0.0166	0.0224	0.0183	0.0123	0.003	0.0019	0.0009	0.0011	0.0007
2.00E-04	0.0046	0.0111	0.04	0.1356	0.0571	0.0287	0.02	0.0146	0.0188	0.0125	0.0091	0.0021	0.0015	0.0005	0.0007	0.0006
2.50E-04	0.003	0.0071	0.0299	0.1235	0.0531	0.0257	0.0172	0.0112	0.0111	0.0071	0.0041	0.0009	0.0008	0.0002	0.0003	0.0003
3.00E-04	0.0023	0.0052	0.0237	0.1102	0.0479	0.0234	0.0147	0.0094	0.0081	0.0049	0.003	0.0006	0.0006	0	0.0002	0.0002
3.50E-04	0.0016	0.0034	0.0177	0.0935	0.039	0.0171	0.0103	0.0056	0.0058	0.0022	0.0017	0.0005	0.0003	0	0	0.0002
4.00E-04	0.0009	0.0028	0.0144	0.0784	0.0317	0.0142	0.0081	0.0038	0.0036	0.001	0.0011	0.0002	0.0001	0	0	0.0001
5.00E-04	0.0006	0.0022	0.0106	0.0544	0.0259	0.011	0.0056	0.0019	0.0021	0.0005	0.0006	0.0002	0	0	0	0.0001
6.00E-04	0.0006	0.0017	0.0078	0.0268	0.015	0.008	0.0036	0.0011	0.0009	0.0002	0.0002	0	0	0	0	0.0001
7.00E-04	0.0003	0.0009	0.004	0.0133	0.0078	0.0052	0.0021	0.0007	0.0008	0.0002	0.0001	0	0	0	0	0.0001
8.50E-04	0.0001	0.0005	0.0018	0.0046	0.004	0.0029	0.0013	0.0006	0.0003	0.0001	0	0	0	0	0	0.0001
1.00E-03	0.0001	0.0002	0.001	0.0023	0.0025	0.0015	0.0007	0.0001	0.0002	0.0001	0	0	0	0	0	0
1.40E-03	0	0	0.0001	0.0003	0.0003	0	0	0	0	0	0	0	0	0	0	0

Table Rev. 36

Table 2.3-94

2000 Probability values for 1 hour at SSES EAB

Probability that the X/Q is Greater than the Adjacent Quantized Level

QUANTIZED LEVEL	DIRECTION															
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
0.00E+00	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
1.00E-10	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
3.50E-09	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
1.00E-08	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
2.50E-08	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
7.00E-08	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
1.00E-07	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
1.50E-07	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
2.20E-07	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
3.20E-07	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
4.80E-07	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
7.00E-07	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
1.00E-06	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
1.50E-06	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
2.00E-06	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
3.00E-06	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0556	0.0296	0.0235	0.0344	0.0479
4.00E-06	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0438	0.0365	0.0622	0.0669	0.0974	0.0546	0.0295	0.0235	0.0344	0.0479
5.00E-06	0.054	0.0683	0.0977	0.1611	0.0783	0.0403	0.0435	0.0364	0.0618	0.0666	0.095	0.0534	0.029	0.0234	0.0344	0.0479
7.00E-06	0.0535	0.0663	0.0967	0.161	0.0783	0.0402	0.0434	0.0363	0.0597	0.065	0.089	0.053	0.0288	0.0234	0.0342	0.0479
8.50E-06	0.0534	0.0661	0.0961	0.161	0.0782	0.0401	0.0433	0.0358	0.0589	0.064	0.0866	0.0526	0.0288	0.0234	0.0342	0.0478
1.00E-05	0.0531	0.0656	0.0955	0.1607	0.0782	0.0398	0.043	0.0356	0.0581	0.0634	0.0856	0.0519	0.0283	0.0234	0.0342	0.0476
1.50E-05	0.0524	0.0651	0.0944	0.1605	0.0779	0.0394	0.0429	0.0352	0.0574	0.0624	0.0828	0.0499	0.0265	0.0228	0.0339	0.0474
2.00E-05	0.0523	0.064	0.0938	0.16	0.0774	0.0389	0.0427	0.0349	0.0568	0.062	0.0801	0.0485	0.026	0.0226	0.0338	0.0469
2.50E-05	0.0518	0.0634	0.0934	0.1598	0.0773	0.0385	0.0425	0.0347	0.0564	0.061	0.078	0.0475	0.0256	0.0225	0.0338	0.0468
3.00E-05	0.0513	0.0627	0.093	0.1597	0.0771	0.0382	0.0424	0.0345	0.0554	0.0602	0.0766	0.0453	0.0251	0.0223	0.0337	0.0467
3.50E-05	0.0509	0.0623	0.0919	0.1594	0.0771	0.038	0.0422	0.0345	0.0547	0.0589	0.0753	0.0432	0.0244	0.0221	0.0336	0.0465
4.00E-05	0.05	0.0622	0.0912	0.1591	0.0768	0.0377	0.0422	0.0342	0.0546	0.0584	0.0733	0.0393	0.0242	0.021	0.0331	0.0451
5.00E-05	0.0467	0.0615	0.0905	0.1588	0.0767	0.0366	0.0418	0.034	0.0537	0.058	0.0689	0.0315	0.0215	0.0185	0.0303	0.0381
6.00E-05	0.0411	0.0608	0.0897	0.1584	0.0763	0.0362	0.0414	0.0338	0.0527	0.0567	0.0632	0.025	0.0174	0.0158	0.0236	0.0297
7.00E-05	0.0355	0.0588	0.0892	0.1582	0.0758	0.0357	0.041	0.0336	0.0521	0.0561	0.0575	0.0205	0.014	0.0124	0.0168	0.0218
8.50E-05	0.0225	0.049	0.0828	0.1547	0.074	0.0341	0.0394	0.0321	0.0477	0.0513	0.0441	0.0138	0.0079	0.0073	0.0084	0.0112
1.00E-04	0.0175	0.0406	0.0726	0.1514	0.0696	0.0318	0.0361	0.0295	0.043	0.0451	0.0349	0.0097	0.005	0.0042	0.0062	0.0078
1.30E-04	0.0097	0.0297	0.0599	0.1467	0.0651	0.0293	0.0306	0.0248	0.0356	0.0301	0.0177	0.0053	0.0027	0.001	0.0024	0.003
1.70E-04	0.0051	0.0193	0.0495	0.1395	0.0618	0.0269	0.0265	0.021	0.028	0.0207	0.0092	0.0027	0.0016	0.0007	0.0014	0.0018
2.00E-04	0.0039	0.0139	0.0424	0.1317	0.0574	0.0245	0.0224	0.0183	0.0226	0.0144	0.0056	0.0015	0.0009	0.0005	0.0008	0.001
2.50E-04	0.0023	0.009	0.0316	0.1169	0.051	0.0201	0.0183	0.0132	0.0147	0.0086	0.0029	0.0003	0.0007	0.0002	0.0002	0.0005
3.00E-04	0.0011	0.0061	0.0257	0.1024	0.0446	0.0166	0.0152	0.0098	0.0107	0.0057	0.0017	0.0001	0.0006	0.0001	0.0002	0.0002
3.50E-04	0.0007	0.0042	0.0196	0.0813	0.0381	0.014	0.0114	0.0068	0.007	0.003	0.001	0.0001	0.0003	0.0001	0.0001	0.0001
4.00E-04	0.0005	0.003	0.0161	0.0685	0.0311	0.011	0.0091	0.0053	0.0047	0.0023	0.0006	0.0001	0.0003	0.0001	0.0001	0.0001
5.00E-04	0.0002	0.0015	0.0122	0.0438	0.0218	0.0073	0.0058	0.0032	0.0023	0.001	0.0006	0	0.0003	0.0001	0.0001	0
6.00E-04	0.0001	0.0011	0.0079	0.025	0.0148	0.0051	0.0042	0.0017	0.0011	0.0009	0.0002	0	0.0003	0.0001	0	0
7.00E-04	0	0.0008	0.0049	0.013	0.0075	0.0021	0.0025	0.0007	0.0002	0.0006	0	0	0.0001	0.0001	0	0
8.50E-04	0	0.0005	0.0016	0.0031	0.0027	0.001	0.001	0.0001	0.0001	0.0001	0	0	0.0001	0	0	0
1.00E-03	0	0.0002	0.0009	0.0016	0.0014	0.0002	0.0006	0.0001	0	0	0	0	0	0	0	0
1.40E-03	0	0	0.0001	0.0002	0.0001	0	0	0	0	0	0	0	0	0	0	0
2.00E-03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-95

2001 Probability values for 1 hour at SSES EAB

Probability that the X/Q is Greater than the Adjacent Quantized Level

QUANTIZED LEVEL	DIRECTION															
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
0.00E+00	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
1.00E-10	0.0515	0.0592	0.0751	0.1698	0.0853	0.0466	0.0507	0.0465	0.0659	0.0674	0.1049	0.0558	0.0296	0.0235	0.0344	0.0479
3.50E-09	0.0515	0.0592	0.0751	0.1698	0.0853	0.0466	0.0507	0.0465	0.0659	0.0674	0.1049	0.0558	0.0296	0.0235	0.0344	0.0479
1.00E-08	0.0515	0.0592	0.0751	0.1698	0.0853	0.0466	0.0507	0.0465	0.0659	0.0674	0.1049	0.0558	0.0296	0.0235	0.0344	0.0479
2.50E-08	0.0515	0.0592	0.0751	0.1698	0.0853	0.0466	0.0507	0.0465	0.0659	0.0674	0.1049	0.0558	0.0296	0.0235	0.0344	0.0479
7.00E-08	0.0515	0.0592	0.0751	0.1698	0.0853	0.0466	0.0507	0.0465	0.0659	0.0674	0.1048	0.0554	0.0296	0.0235	0.0344	0.0479
1.00E-07	0.0508	0.0585	0.0751	0.1698	0.0853	0.0466	0.0506	0.0465	0.0654	0.0669	0.0998	0.0526	0.0284	0.0234	0.0344	0.0478
1.50E-07	0.0485	0.0562	0.0738	0.1697	0.0851	0.0465	0.0496	0.0457	0.0629	0.0645	0.0875	0.0495	0.0264	0.0226	0.0339	0.0476
2.20E-07	0.0482	0.0542	0.0718	0.1685	0.0846	0.0457	0.0473	0.0436	0.0596	0.0591	0.08	0.0485	0.0259	0.0225	0.0338	0.0471
3.20E-07	0.048	0.053	0.0707	0.1672	0.0835	0.0448	0.0455	0.0422	0.0575	0.0569	0.0768	0.0481	0.0258	0.0225	0.0337	0.047
4.80E-07	0.0477	0.0528	0.0698	0.1658	0.0825	0.0442	0.045	0.0417	0.0568	0.0559	0.0756	0.0476	0.0256	0.0225	0.0337	0.0469
7.00E-07	0.0474	0.0524	0.0697	0.1656	0.0818	0.0436	0.045	0.0416	0.0565	0.0558	0.0741	0.0458	0.0251	0.0224	0.0336	0.0467
1.00E-06	0.0454	0.0521	0.0695	0.1656	0.0818	0.0436	0.045	0.0415	0.0561	0.0554	0.0721	0.0445	0.0249	0.0218	0.0334	0.0466
1.50E-06	0.0453	0.052	0.0692	0.1654	0.0815	0.0434	0.0447	0.0415	0.055	0.0545	0.0706	0.042	0.0243	0.0215	0.0333	0.0462
2.00E-06	0.0451	0.0515	0.0688	0.1648	0.0813	0.043	0.0446	0.0413	0.0542	0.054	0.0681	0.0345	0.0229	0.0201	0.0316	0.0418
3.00E-06	0.0322	0.0483	0.0674	0.1633	0.081	0.0429	0.043	0.0404	0.0534	0.0523	0.0545	0.0201	0.0136	0.0116	0.0156	0.0204
4.00E-06	0.02	0.0421	0.0646	0.1624	0.0795	0.0411	0.0394	0.0385	0.0497	0.0485	0.0428	0.0137	0.0072	0.0068	0.008	0.0104
5.00E-06	0.0166	0.0367	0.0622	0.1606	0.0774	0.039	0.0362	0.0348	0.0438	0.0447	0.0374	0.0103	0.005	0.0042	0.0061	0.0078
7.00E-06	0.0129	0.0302	0.0555	0.1564	0.0747	0.0367	0.0324	0.03	0.0386	0.0373	0.0259	0.0058	0.0027	0.0024	0.005	0.0062
8.50E-06	0.0097	0.0243	0.0511	0.1536	0.0721	0.0354	0.0297	0.0264	0.034	0.0307	0.0169	0.0038	0.0019	0.0015	0.0023	0.0029
1.00E-05	0.007	0.0194	0.0473	0.1502	0.07	0.0327	0.0275	0.0241	0.0304	0.0244	0.0133	0.003	0.0017	0.0007	0.0015	0.0019
1.50E-05	0.0032	0.0112	0.0368	0.1387	0.0639	0.0281	0.0217	0.0178	0.0206	0.0118	0.0063	0.0013	0.0008	0.0002	0.0007	0.0008
2.00E-05	0.0018	0.0063	0.0301	0.1264	0.057	0.0238	0.0167	0.0124	0.0129	0.0062	0.0033	0.0002	0.0007	0.0001	0.0002	0.0005
2.50E-05	0.0014	0.0041	0.0243	0.1159	0.0492	0.0198	0.0117	0.0082	0.0086	0.0044	0.0024	0.0002	0.0006	0.0001	0.0002	0.0002
3.00E-05	0.001	0.0028	0.0202	0.1037	0.043	0.0163	0.0093	0.0064	0.0074	0.003	0.0016	0.0001	0.0005	0.0001	0.0002	0.0001
3.50E-05	0.0007	0.0024	0.0162	0.0921	0.0381	0.0132	0.0074	0.0057	0.0062	0.0023	0.0014	0	0.0005	0.0001	0.0001	0
4.00E-05	0.0006	0.0016	0.0139	0.0783	0.0346	0.011	0.0063	0.0044	0.0043	0.0016	0.0011	0	0.0003	0.0001	0	0
5.00E-05	0.0003	0.0011	0.0109	0.0588	0.0284	0.0092	0.0048	0.003	0.0024	0.001	0.0007	0	0.0002	0.0001	0	0
6.00E-05	0.0003	0.0009	0.0077	0.0393	0.0201	0.0067	0.0028	0.0017	0.0014	0.0009	0.0005	0	0.0002	0.0001	0	0
7.00E-05	0.0001	0.0008	0.0051	0.0248	0.0133	0.0046	0.0021	0.0014	0.0008	0.0005	0.0003	0	0.0001	0.0001	0	0
8.50E-05	0.0001	0.0002	0.0029	0.0107	0.0072	0.0018	0.0013	0.0003	0.0001	0.0001	0.0003	0	0.0001	0	0	0
1.00E-04	0.0001	0.0002	0.0016	0.0037	0.0034	0.0017	0.0011	0.0002	0	0	0.0001	0	0	0	0	0
1.30E-04	0	0.0001	0.0001	0.0002	0.0002	0.0001	0.0003	0	0	0	0	0	0	0	0	0
1.70E-04	0	0	0	0.0001	0	0	0	0	0	0	0	0	0	0	0	0
2.00E-04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-96
2002 Probability values for 1 hour at SSES EAB
Probability that the X/Q is Greater than the Adjacent Quantized Level

QUANTIZED LEVEL	DIRECTION N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
0.00E+00	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
1.00E-10	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
3.50E-09	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
1.00E-08	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
2.50E-08	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
7.00E-08	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
1.00E-07	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
1.50E-07	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
2.20E-07	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
3.20E-07	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
4.80E-07	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
7.00E-07	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
1.00E-06	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
1.50E-06	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
2.00E-06	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
3.00E-06	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1142	0.0627	0.0314	0.0204	0.0296	0.0398
4.00E-06	0.0491	0.0718	0.0827	0.1446	0.0676	0.0451	0.0424	0.0462	0.0672	0.0821	0.1114	0.061	0.0314	0.0204	0.0295	0.0395
5.00E-06	0.0477	0.0707	0.0827	0.1446	0.0676	0.0447	0.042	0.0458	0.0661	0.0803	0.1064	0.059	0.031	0.0204	0.0295	0.0392
7.00E-06	0.0466	0.0699	0.0823	0.1446	0.0674	0.0447	0.0418	0.0456	0.0655	0.0768	0.1001	0.058	0.0309	0.0204	0.0295	0.039
8.50E-06	0.0465	0.0694	0.0816	0.1446	0.0674	0.0446	0.0418	0.0455	0.0655	0.0756	0.0981	0.0575	0.0309	0.0204	0.0295	0.0388
1.00E-05	0.0463	0.0693	0.0813	0.1443	0.0674	0.0446	0.0418	0.0454	0.0649	0.0752	0.0964	0.0568	0.0307	0.0204	0.0295	0.0386
1.50E-05	0.0451	0.0685	0.0805	0.1437	0.0672	0.0443	0.0417	0.0451	0.0639	0.0741	0.0903	0.0538	0.0303	0.0201	0.0292	0.0378
2.00E-05	0.0441	0.0677	0.0802	0.1436	0.0669	0.0443	0.0414	0.0451	0.0634	0.0727	0.0848	0.0514	0.0299	0.0201	0.029	0.0373
2.50E-05	0.0438	0.0669	0.08	0.1435	0.0667	0.0443	0.0414	0.0447	0.0625	0.0712	0.0825	0.0504	0.0297	0.0201	0.0289	0.0369
3.00E-05	0.0429	0.0663	0.0794	0.1435	0.0666	0.0443	0.041	0.0447	0.0622	0.0707	0.0808	0.0496	0.0287	0.0201	0.0288	0.0365
3.50E-05	0.0427	0.0661	0.0793	0.1433	0.0665	0.044	0.041	0.0443	0.0618	0.07	0.0793	0.0478	0.0277	0.0194	0.0285	0.0362
4.00E-05	0.0423	0.0652	0.0792	0.1432	0.0663	0.0439	0.0408	0.044	0.0612	0.0696	0.0775	0.0451	0.0266	0.0186	0.0277	0.0348
5.00E-05	0.0418	0.0644	0.0786	0.1431	0.0659	0.0433	0.0406	0.0437	0.0604	0.0688	0.0719	0.0372	0.0218	0.0146	0.0234	0.0295
6.00E-05	0.0384	0.0635	0.0782	0.1429	0.0657	0.043	0.04	0.0431	0.0583	0.0676	0.0641	0.0295	0.0173	0.0118	0.0181	0.0235
7.00E-05	0.0338	0.0614	0.0772	0.1428	0.0655	0.0423	0.0388	0.0415	0.0554	0.0651	0.0572	0.0237	0.0148	0.0092	0.0135	0.0184
8.50E-05	0.022	0.0513	0.0716	0.1416	0.064	0.0402	0.0353	0.0362	0.0514	0.0582	0.0414	0.0171	0.0099	0.0047	0.0067	0.0092
1.00E-04	0.0171	0.0432	0.0632	0.1386	0.0618	0.0371	0.0309	0.032	0.0448	0.0482	0.0311	0.0124	0.0061	0.0033	0.0047	0.0074
1.30E-04	0.0091	0.0261	0.0549	0.1349	0.0584	0.0338	0.0258	0.0259	0.0331	0.0288	0.0136	0.0055	0.0029	0.0016	0.0021	0.0033
1.70E-04	0.0054	0.0175	0.0459	0.1306	0.0545	0.0308	0.0224	0.0205	0.0252	0.0165	0.0066	0.0027	0.0018	0.001	0.0009	0.0018
2.00E-04	0.0032	0.0134	0.0399	0.1268	0.052	0.028	0.0194	0.0176	0.0196	0.0114	0.0043	0.0014	0.0014	0.0006	0.0006	0.0015
2.50E-04	0.0022	0.0089	0.032	0.1183	0.0473	0.024	0.0152	0.0127	0.0122	0.007	0.0021	0.0008	0.0005	0.0005	0.0001	0.0007
3.00E-04	0.0016	0.0067	0.0277	0.1093	0.0415	0.0219	0.012	0.0106	0.0082	0.0045	0.0015	0.0003	0	0.0003	0.0001	0.0002
3.50E-04	0.0009	0.005	0.0213	0.0929	0.035	0.0168	0.0093	0.0078	0.0055	0.0028	0.0008	0.0002	0	0.0003	0.0001	0.0002
4.00E-04	0.0007	0.0037	0.0179	0.0801	0.0308	0.0142	0.0074	0.0061	0.0035	0.0017	0.0006	0.0002	0	0.0001	0.0001	0.0002
5.00E-04	0.0005	0.0029	0.0119	0.0568	0.0233	0.0108	0.0046	0.0038	0.0015	0.0007	0.0003	0.0001	0	0	0	0.0001
6.00E-04	0.0003	0.0018	0.0071	0.0329	0.0146	0.0073	0.0031	0.0016	0.0008	0.0005	0.0003	0.0001	0	0	0	0
7.00E-04	0.0002	0.0012	0.0032	0.0145	0.0073	0.0038	0.0017	0.0014	0.0002	0.0001	0.0001	0.0001	0	0	0	0
8.50E-04	0.0001	0.0005	0.0015	0.0041	0.0022	0.0016	0.0006	0.0003	0.0001	0.0001	0.0001	0.0001	0	0	0	0
1.00E-03	0.0001	0.0005	0.0008	0.0014	0.0009	0.0007	0.0003	0.0001	0	0	0	0.0001	0	0	0	0
1.40E-03	0	0	0	0	0.0001	0.0001	0	0	0	0	0	0	0	0	0	0
2.00E-03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3.00E-03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-97
2003 Probability values for 1 hour at SSES EAB
Probability that the X/Q is Greater than the Adjacent Quantized Level

QUANTIZED LEVEL	DIRECTION															
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
0.00E+00	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
1.00E-10	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
3.50E-09	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
1.00E-08	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
2.50E-08	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
7.00E-08	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
1.00E-07	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
1.50E-07	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
2.20E-07	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
3.20E-07	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
4.80E-07	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
7.00E-07	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
1.00E-06	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
1.50E-06	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
2.00E-06	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
3.00E-06	0.0391	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1002	0.0619	0.0297	0.0214	0.0287	0.0348
4.00E-06	0.039	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0682	0.0974	0.0605	0.0293	0.0213	0.0282	0.034
5.00E-06	0.0384	0.0764	0.1089	0.1414	0.0673	0.0497	0.0612	0.0468	0.0599	0.0674	0.096	0.0592	0.0289	0.021	0.0277	0.0333
7.00E-06	0.038	0.075	0.1082	0.1413	0.0672	0.0496	0.0609	0.0464	0.0584	0.0656	0.0924	0.0575	0.0286	0.0205	0.0277	0.0331
8.50E-06	0.038	0.075	0.1081	0.1412	0.0671	0.0494	0.0609	0.046	0.058	0.0644	0.0908	0.0565	0.0285	0.0203	0.0277	0.0326
1.00E-05	0.038	0.0749	0.1078	0.1411	0.067	0.0493	0.0606	0.0455	0.0573	0.0629	0.0894	0.0559	0.0282	0.0202	0.0277	0.0325
1.50E-05	0.0376	0.0746	0.1073	0.1408	0.0665	0.0491	0.0605	0.045	0.0566	0.0614	0.0867	0.0538	0.0277	0.0195	0.0273	0.032
2.00E-05	0.0372	0.0742	0.1072	0.1406	0.0657	0.049	0.0601	0.0446	0.056	0.0605	0.0847	0.0524	0.0271	0.0194	0.0273	0.0319
2.50E-05	0.037	0.0741	0.1066	0.1406	0.0655	0.0489	0.0601	0.044	0.056	0.0605	0.0838	0.0523	0.0267	0.0191	0.027	0.0316
3.00E-05	0.0365	0.0739	0.1062	0.1405	0.0654	0.0489	0.0597	0.0439	0.0559	0.0601	0.0829	0.0503	0.0257	0.0188	0.0263	0.0314
3.50E-05	0.0364	0.0738	0.1062	0.1404	0.0652	0.0488	0.0594	0.0436	0.0559	0.0597	0.0816	0.0488	0.0254	0.0174	0.026	0.0314
4.00E-05	0.0362	0.0728	0.1059	0.1398	0.0651	0.0486	0.059	0.0435	0.0558	0.0595	0.0805	0.0453	0.024	0.0164	0.0252	0.0307
5.00E-05	0.0348	0.0719	0.1057	0.1395	0.0648	0.0481	0.0584	0.0431	0.0557	0.0591	0.0751	0.0378	0.021	0.0148	0.0216	0.026
6.00E-05	0.0319	0.071	0.105	0.1389	0.0643	0.0473	0.0568	0.0422	0.0549	0.0581	0.0664	0.0312	0.0165	0.0111	0.0164	0.0195
7.00E-05	0.0289	0.0679	0.103	0.1381	0.0634	0.0464	0.0552	0.0413	0.0529	0.0566	0.0566	0.0258	0.013	0.0099	0.0126	0.0145
8.50E-05	0.0201	0.0561	0.0936	0.1357	0.0617	0.0424	0.0485	0.038	0.0483	0.05	0.0379	0.0179	0.0093	0.0071	0.0076	0.0089
1.00E-04	0.0154	0.0493	0.0858	0.1321	0.0602	0.0393	0.042	0.0338	0.0438	0.043	0.0267	0.0129	0.0063	0.0047	0.0054	0.007
1.30E-04	0.0075	0.032	0.0723	0.1269	0.0561	0.034	0.0362	0.0265	0.0317	0.0258	0.0121	0.006	0.0027	0.002	0.0021	0.0023
1.70E-04	0.0041	0.0217	0.0619	0.1229	0.0528	0.0302	0.03	0.0209	0.0243	0.0158	0.006	0.0029	0.0016	0.0009	0.0013	0.0014
2.00E-04	0.003	0.0151	0.0535	0.1168	0.0499	0.0271	0.0252	0.0176	0.0184	0.0112	0.0041	0.0017	0.0013	0.0007	0.0008	0.0009
2.50E-04	0.0015	0.0084	0.0385	0.1037	0.0441	0.0216	0.0195	0.0123	0.01	0.0054	0.0017	0.0009	0.0006	0.0003	0.0005	0.0006
3.00E-04	0.001	0.0045	0.0318	0.0927	0.0368	0.0173	0.0156	0.0083	0.0067	0.0038	0.0006	0.0005	0.0006	0.0001	0.0002	0.0005
3.50E-04	0.0008	0.0027	0.0243	0.0732	0.0297	0.0128	0.0114	0.0052	0.0038	0.0015	0.0003	0.0002	0.0005	0	0.0001	0.0002
4.00E-04	0.0005	0.0021	0.0212	0.0622	0.0258	0.0113	0.0078	0.0038	0.0027	0.001	0.0002	0.0002	0.0005	0	0.0001	0.0001
5.00E-04	0.0002	0.0013	0.0142	0.0411	0.0183	0.0068	0.0045	0.0014	0.0013	0.0008	0	0.0001	0.0005	0	0.0001	0
6.00E-04	0.0001	0.001	0.0093	0.0225	0.0115	0.0035	0.0031	0.0009	0.0008	0.0003	0	0	0.0003	0	0	0
7.00E-04	0	0.0007	0.0051	0.0093	0.0055	0.0018	0.0017	0.0006	0.0002	0	0	0	0.0001	0	0	0
8.50E-04	0	0.0002	0.0012	0.0023	0.0024	0.0008	0.0003	0.0002	0	0	0	0	0.0001	0	0	0
1.00E-03	0	0.0001	0.0003	0.0008	0.0006	0.0003	0.0002	0.0001	0	0	0	0	0.0001	0	0	0
1.40E-03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2.00E-03	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

TABLE 2.3-98
1999 – 2003 Probability Values for 1 hour at SSES EAB
Probability that the X/Q is Greater than the Adjacent Quantized Level

QUANTIZED LEVEL	DIRECTION N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
1.00E-10	0.0513	0.0705	0.0923	0.1558	0.0749	0.0453	0.0487	0.0426	0.0618	0.0704	0.1042	0.0599	0.0303	0.0213	0.0315	0.0419
3.50E-09	0.0513	0.0705	0.0923	0.1558	0.0749	0.0453	0.0487	0.0426	0.0618	0.0704	0.1042	0.0599	0.0303	0.0213	0.0315	0.0419
1.00E-08	0.0513	0.0705	0.0923	0.1558	0.0749	0.0453	0.0487	0.0426	0.0618	0.0704	0.1042	0.0599	0.0303	0.0213	0.0315	0.0419
2.50E-08	0.0513	0.0705	0.0923	0.1558	0.0749	0.0453	0.0487	0.0426	0.0618	0.0704	0.1042	0.0599	0.0303	0.0213	0.0315	0.0419
7.00E-08	0.0513	0.0705	0.0923	0.1558	0.0749	0.0453	0.0487	0.0426	0.0618	0.0704	0.1042	0.0599	0.0303	0.0213	0.0315	0.0419
1.00E-07	0.0513	0.0705	0.0923	0.1558	0.0749	0.0453	0.0487	0.0426	0.0618	0.0704	0.1042	0.0599	0.0303	0.0213	0.0315	0.0419
1.50E-07	0.0513	0.0705	0.0923	0.1558	0.0749	0.0453	0.0487	0.0426	0.0618	0.0704	0.1042	0.0599	0.0303	0.0213	0.0315	0.0419
2.20E-07	0.0513	0.0705	0.0923	0.1558	0.0749	0.0453	0.0487	0.0426	0.0618	0.0704	0.1042	0.0599	0.0303	0.0213	0.0315	0.0418
3.20E-07	0.0513	0.0705	0.0923	0.1558	0.0749	0.0453	0.0487	0.0426	0.0618	0.0704	0.1042	0.0599	0.0303	0.0213	0.0315	0.0418
4.80E-07	0.0513	0.0705	0.0923	0.1558	0.0749	0.0453	0.0487	0.0426	0.0618	0.0704	0.1042	0.0599	0.0303	0.0213	0.0315	0.0418
7.00E-07	0.0513	0.0705	0.0923	0.1558	0.0749	0.0453	0.0487	0.0426	0.0618	0.0704	0.1042	0.0599	0.0303	0.0213	0.0315	0.0417
1.00E-06	0.0513	0.0705	0.0923	0.1558	0.0749	0.0453	0.0487	0.0426	0.0618	0.0704	0.1042	0.0599	0.0303	0.0213	0.0315	0.0417
1.50-E-06	0.0513	0.0705	0.0923	0.1558	0.0749	0.0453	0.0487	0.0426	0.0618	0.0704	0.1042	0.0599	0.0303	0.0213	0.0315	0.0416
2.00E-06	0.0513	0.0705	0.0923	0.1558	0.0749	0.0453	0.0487	0.0426	0.0618	0.0704	0.1042	0.0599	0.0303	0.0213	0.0315	0.0406
3.00E-06	0.0513	0.0705	0.0923	0.1558	0.0749	0.0453	0.0487	0.0426	0.0618	0.0704	0.1040	0.0597	0.0303	0.0213	0.0315	0.0364
4.00E-06	0.0510	0.0702	0.0923	0.1557	0.0749	0.0453	0.0487	0.0426	0.0616	0.0702	0.1022	0.0584	0.0301	0.0213	0.0314	0.0342
5.00E-06	0.0505	0.0695	0.0921	0.1557	0.0749	0.0452	0.0483	0.0424	0.0610	0.0695	0.0992	0.0564	0.0294	0.0210	0.0313	0.0335
7.00E-06	0.0499	0.0682	0.0912	0.1556	0.0748	0.0451	0.0479	0.0420	0.0594	0.0670	0.0929	0.0553	0.0289	0.0209	0.0312	0.0331
8.50E-06	0.0498	0.0678	0.0905	0.1555	0.0747	0.0450	0.0473	0.0416	0.0586	0.0655	0.0906	0.0548	0.0288	0.0208	0.0312	0.0322
1.00E-05	0.0497	0.0676	0.0901	0.1551	0.0747	0.0448	0.0470	0.0411	0.0578	0.0646	0.0891	0.0541	0.0285	0.0208	0.0312	0.0318
1.50E-05	0.0486	0.0669	0.0893	0.1546	0.0742	0.0444	0.0467	0.0407	0.0570	0.0634	0.0857	0.0521	0.0275	0.0203	0.0309	0.0314
2.00E-05	0.0482	0.0660	0.0888	0.1544	0.0736	0.0440	0.0463	0.0404	0.0564	0.0626	0.0826	0.0502	0.0270	0.0201	0.0307	0.0310
2.50E-05	0.0478	0.0654	0.0863	0.1543	0.0734	0.0438	0.0461	0.0401	0.0559	0.0618	0.0809	0.0493	0.0265	0.0200	0.0306	0.0307
3.00E-05	0.0470	0.0646	0.0876	0.1541	0.0732	0.0436	0.0458	0.0399	0.0554	0.0612	0.0794	0.0475	0.0257	0.0198	0.0303	0.0306
3.50E-05	0.0468	0.0642	0.0873	0.1539	0.0731	0.0434	0.0456	0.0396	0.0550	0.0606	0.0779	0.0457	0.0250	0.0193	0.0301	0.0303
4.00E-05	0.0462	0.0636	0.0869	0.1535	0.0730	0.0432	0.0452	0.0394	0.0547	0.0600	0.0763	0.0421	0.0243	0.0184	0.0295	0.0291
5.00E-05	0.0438	0.0626	0.0862	0.1531	0.0726	0.0427	0.0448	0.0389	0.0540	0.0593	0.0716	0.0343	0.0210	0.0160	0.0263	0.0243
6.00E-05	0.0384	0.0614	0.0855	0.1527	0.0721	0.0423	0.0440	0.0384	0.0529	0.0582	0.0642	0.0272	0.0167	0.0131	0.0204	0.0188
7.00E-05	0.0329	0.0585	0.0843	0.1521	0.0717	0.0417	0.0430	0.0375	0.0514	0.0568	0.0570	0.0224	0.0135	0.0105	0.0152	0.0142
8.50E-05	0.0213	0.0485	0.0778	0.1498	0.0699	0.0393	0.0394	0.0347	0.0474	0.0511	0.0425	0.0159	0.0087	0.0062	0.0081	0.0078
1.00E-04	0.0163	0.0406	0.0703	0.1467	0.0675	0.0366	0.0353	0.0310	0.0421	0.0441	0.0334	0.0113	0.0057	0.0040	0.0058	0.0058
1.30E-04	0.0088	0.0268	0.0589	0.1419	0.0637	0.0333	0.0299	0.0253	0.0326	0.0285	0.0160	0.0055	0.0029	0.0015	0.0022	0.0021
1.70E-04	0.0052	0.0181	0.0503	0.1369	0.0603	0.0307	0.0259	0.0205	0.0258	0.0183	0.0090	0.0028	0.0017	0.0008	0.0012	0.0011
2.00E-04	0.0038	0.0132	0.0433	0.1310	0.0571	0.0280	0.0224	0.0177	0.0207	0.0128	0.0061	0.0016	0.0012	0.0006	0.0007	0.0008
2.50E-04	0.0022	0.0080	0.0328	0.1185	0.0515	0.0237	0.0182	0.0129	0.0129	0.0072	0.0030	0.0006	0.0007	0.0003	0.0003	0.0004
3.00E-04	0.0015	0.0054	0.0270	0.1058	0.0454	0.0205	0.0147	0.0099	0.0090	0.0047	0.0017	0.0003	0.0005	0.0001	0.0002	0.0002
3.50E-04	0.0010	0.0036	0.0204	0.0873	0.0378	0.0161	0.0108	0.0068	0.0059	0.0025	0.0011	0.0002	0.0003	0.0001	0.0001	0.0002
4.00E-04	0.0007	0.0027	0.0170	0.0738	0.0318	0.0134	0.0083	0.0050	0.0040	0.0016	0.0007	0.0002	0.0002	0.0001	0.0001	0.0001
5.00E-04	0.0004	0.0019	0.0118	0.0469	0.0236	0.0093	0.0052	0.0028	0.0019	0.0008	0.0004	0.0001	0.0002	0.0000	0.0001	0.0001
6.00E-04	0.0003	0.0013	0.0077	0.0273	0.0149	0.0062	0.0034	0.0014	0.0010	0.0005	0.0002	0.0000	0.0002	0.0000	0.0000	0.0000
7.00E-04	0.0001	0.0008	0.0042	0.0128	0.0076	0.0032	0.0019	0.0009	0.0003	0.0002	0.0001	0.0000	0.0001	0.0000	0.0000	0.0000
8.50E-04	0.0001	0.0004	0.0015	0.0036	0.0030	0.0016	0.0009	0.0003	0.0001	0.0001	0.0000	0.0000	0.0001	0.0000	0.0000	0.0000
1.00E-03	0.0001	0.0002	0.0007	0.0015	0.0013	0.0007	0.0005	0.0001	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1.40E-03	0.0000	0.0000	0.0000	0.0001	0.0001	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
2.00E-03	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Table 2.3-99

1999 Probability values for 1 hour at SSES LPZ

Probability that the X/Q is Greater than the Adjacent Quantized Level

QUANTIZED LEVEL	DIRECTION															
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
0.00E+00	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
1.00E-10	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
3.50E-09	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
1.00E-08	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
2.50E-08	0.0616	0.0757	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1023	0.063	0.0313	0.0177	0.0306	0.0393
7.00E-08	0.0616	0.0756	0.0964	0.1616	0.0759	0.0447	0.0445	0.0366	0.0521	0.0667	0.1022	0.0623	0.0313	0.0177	0.0306	0.0393
1.00E-07	0.0616	0.0736	0.096	0.1615	0.0758	0.0447	0.0444	0.0366	0.0518	0.0663	0.0997	0.0567	0.029	0.0169	0.0304	0.0393
1.50E-07	0.0597	0.0709	0.0936	0.1611	0.0757	0.0446	0.0436	0.036	0.0498	0.0625	0.0889	0.0528	0.026	0.0161	0.0302	0.0389
2.20E-07	0.0588	0.0688	0.0908	0.1603	0.0751	0.044	0.0412	0.0344	0.0464	0.0594	0.0836	0.051	0.0258	0.0158	0.0297	0.0385
3.20E-07	0.0587	0.0681	0.0892	0.1593	0.0748	0.0431	0.0406	0.0337	0.0458	0.0576	0.0819	0.0507	0.0257	0.0158	0.0297	0.0383
4.80E-07	0.0587	0.0679	0.0891	0.1584	0.0733	0.0424	0.0399	0.0336	0.0455	0.0571	0.0815	0.0497	0.0257	0.0158	0.0297	0.0383
7.00E-07	0.0577	0.067	0.0889	0.1584	0.0731	0.0423	0.0396	0.0335	0.0454	0.0571	0.0807	0.0472	0.0245	0.0157	0.0294	0.038
1.00E-06	0.0568	0.0653	0.0881	0.1583	0.0729	0.0422	0.0396	0.0333	0.0453	0.0564	0.0787	0.0461	0.0236	0.0149	0.0289	0.037
1.50E-06	0.0564	0.0642	0.0873	0.1578	0.0727	0.0419	0.0392	0.0329	0.0443	0.0548	0.0768	0.0445	0.0231	0.0148	0.0288	0.0366
2.00E-06	0.0539	0.0639	0.0872	0.1573	0.0725	0.0414	0.0377	0.0319	0.0435	0.0542	0.074	0.0367	0.0205	0.0142	0.0279	0.0313
3.00E-06	0.0312	0.0546	0.0837	0.1563	0.0717	0.0405	0.0357	0.0303	0.0425	0.0521	0.0567	0.0204	0.0114	0.0086	0.0156	0.0159
4.00E-06	0.0201	0.0423	0.0742	0.1529	0.0695	0.037	0.0331	0.0281	0.0393	0.0458	0.0451	0.0165	0.0079	0.0045	0.0091	0.0088
5.00E-06	0.0155	0.0351	0.0683	0.1503	0.0681	0.0354	0.0314	0.0255	0.0351	0.0399	0.0391	0.0125	0.0062	0.0037	0.0068	0.0067
7.00E-06	0.0122	0.0281	0.0595	0.1451	0.0646	0.0333	0.0264	0.0202	0.0304	0.0336	0.0281	0.0083	0.0047	0.0026	0.005	0.0048
8.50E-06	0.0091	0.0213	0.0537	0.1429	0.0624	0.0317	0.0237	0.0189	0.0268	0.0273	0.019	0.0056	0.0032	0.0017	0.0033	0.0021
1.00E-05	0.0069	0.0177	0.0502	0.1407	0.0592	0.0295	0.0214	0.0163	0.0226	0.0222	0.015	0.0041	0.0021	0.0009	0.0016	0.0011
1.50E-05	0.0038	0.0095	0.0372	0.1321	0.0541	0.0268	0.0183	0.0132	0.0164	0.01	0.0073	0.0018	0.0014	0.0005	0.0007	0.0006
2.00E-05	0.0029	0.006	0.0272	0.1211	0.049	0.0232	0.0147	0.0095	0.0088	0.0061	0.0042	0.001	0.0007	0.0003	0.0003	0.0003
2.50E-05	0.0021	0.0048	0.0231	0.1118	0.0419	0.0169	0.0109	0.006	0.0072	0.0039	0.0031	0.0008	0.0006	0.0001	0	0.0002
3.00E-05	0.0014	0.004	0.0198	0.1021	0.0373	0.0141	0.0093	0.0047	0.0058	0.0029	0.0023	0.0006	0.0003	0	0	0.0001
3.50E-05	0.0013	0.0032	0.0169	0.0923	0.0339	0.0125	0.0073	0.0041	0.0045	0.0021	0.0016	0.0005	0.0002	0	0	0.0001
4.00E-05	0.0009	0.0028	0.0139	0.0812	0.0299	0.0118	0.0063	0.0031	0.003	0.0009	0.0009	0.0002	0.0001	0	0	0.0001
5.00E-05	0.0005	0.0023	0.0111	0.064	0.026	0.0104	0.0056	0.0021	0.0019	0.0005	0.0006	0.0002	0	0	0	0.0001
6.00E-05	0.0005	0.0019	0.0083	0.0394	0.0167	0.0078	0.0033	0.001	0.001	0.0002	0.0002	0	0	0	0	0.0001
7.00E-05	0.0003	0.0013	0.0068	0.0241	0.0111	0.006	0.0026	0.0008	0.0009	0.0002	0.0002	0	0	0	0	0.0001
8.50E-05	0.0002	0.0008	0.0033	0.0108	0.0067	0.0041	0.0016	0.0007	0.0003	0.0001	0	0	0	0	0	0.0001
1.00E-04	0.0001	0.0005	0.0018	0.0046	0.004	0.0029	0.001	0.0006	0.0003	0.0001	0	0	0	0	0	0.0001
1.30E-04	0.0001	0	0.0003	0.0008	0.001	0.0003	0.0003	0.0001	0.0002	0.0001	0	0	0	0	0	0
1.70E-04	0	0	0.0001	0.0001	0.0001	0	0	0	0	0	0	0	0	0	0	0
2.00E-04	0	0	0.0001	0.0001	0.0001	0	0	0	0	0	0	0	0	0	0	0
2.50E-04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-100

2000 Probability values for 1 hour at SSES LPZ

Probability that the X/Q is Greater than the Adjacent Quantized Level

QUANTIZED LEVEL	DIRECTION															
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
0.00E+00	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
1.00E-10	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
3.50E-09	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
1.00E-08	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
2.50E-08	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0623	0.0669	0.0986	0.0558	0.0296	0.0235	0.0344	0.0479
7.00E-08	0.0542	0.0685	0.0978	0.1613	0.0783	0.0403	0.0439	0.0365	0.0622	0.0669	0.0985	0.0554	0.0296	0.0235	0.0344	0.0479
1.00E-07	0.0538	0.0682	0.0978	0.1612	0.0783	0.0403	0.0435	0.0364	0.062	0.0663	0.0951	0.0526	0.0284	0.0234	0.0344	0.0478
1.50E-07	0.0525	0.0659	0.0967	0.1607	0.0783	0.0402	0.0432	0.036	0.0594	0.0644	0.0854	0.0495	0.0264	0.0226	0.0339	0.0476
2.20E-07	0.0522	0.0637	0.0942	0.1604	0.0779	0.0393	0.0427	0.0349	0.0571	0.0619	0.0795	0.0485	0.0259	0.0225	0.0338	0.0471
3.20E-07	0.0515	0.0628	0.092	0.1595	0.0775	0.0385	0.0426	0.0346	0.0551	0.059	0.0773	0.0481	0.0258	0.0225	0.0337	0.047
4.80E-07	0.0514	0.0626	0.0916	0.159	0.0767	0.0374	0.0424	0.0345	0.0547	0.0586	0.0771	0.0476	0.0256	0.0225	0.0337	0.0469
7.00E-07	0.0509	0.0623	0.0916	0.1589	0.0764	0.0371	0.0422	0.0344	0.0545	0.0586	0.0764	0.0458	0.0251	0.0224	0.0336	0.0467
1.00E-06	0.0506	0.062	0.0909	0.1588	0.0764	0.0371	0.0421	0.0342	0.0542	0.0583	0.074	0.0445	0.0249	0.0218	0.0334	0.0466
1.50E-06	0.0506	0.0614	0.0896	0.1584	0.0763	0.0368	0.042	0.0339	0.0533	0.0567	0.0713	0.042	0.0243	0.0215	0.0333	0.0462
2.00E-06	0.0491	0.0611	0.0894	0.1582	0.076	0.0364	0.042	0.0337	0.0527	0.0566	0.0691	0.0345	0.0229	0.0201	0.0316	0.0418
3.00E-06	0.0344	0.0583	0.0884	0.1578	0.0755	0.0354	0.0409	0.0332	0.0514	0.0559	0.0564	0.0201	0.0136	0.0116	0.0156	0.0204
4.00E-06	0.0217	0.0476	0.0803	0.1541	0.0734	0.0334	0.0387	0.0318	0.0475	0.051	0.0434	0.0137	0.0072	0.0068	0.008	0.0104
5.00E-06	0.0174	0.0408	0.0726	0.1513	0.0694	0.0317	0.036	0.0298	0.0435	0.046	0.0357	0.0103	0.005	0.0042	0.0061	0.0078
7.00E-06	0.0122	0.0322	0.0621	0.1465	0.0647	0.029	0.0309	0.0244	0.0374	0.0368	0.0247	0.0058	0.0027	0.0024	0.005	0.0062
8.50E-06	0.0091	0.0261	0.0547	0.1425	0.0622	0.0274	0.0276	0.0219	0.0338	0.0301	0.0152	0.0038	0.0019	0.0015	0.0023	0.0029
1.00E-05	0.0072	0.0224	0.0505	0.1384	0.0584	0.0253	0.0239	0.0205	0.0309	0.0241	0.0112	0.003	0.0017	0.0007	0.0015	0.0019
1.50E-05	0.0029	0.0122	0.0377	0.1264	0.0517	0.0194	0.0184	0.0153	0.0204	0.0128	0.004	0.0013	0.0008	0.0002	0.0007	0.0008
2.00E-05	0.0022	0.0075	0.0282	0.1123	0.0454	0.0164	0.015	0.0102	0.012	0.0076	0.0029	0.0002	0.0007	0.0001	0.0002	0.0005
2.50E-05	0.0014	0.0058	0.0226	0.1022	0.0394	0.0132	0.0112	0.0068	0.0087	0.0053	0.0017	0.0002	0.0006	0.0001	0.0002	0.0002
3.00E-05	0.0008	0.0042	0.0202	0.0927	0.035	0.011	0.0095	0.0057	0.0073	0.0038	0.001	0.0001	0.0005	0.0001	0.0002	0.0001
3.50E-05	0.0006	0.0034	0.0182	0.0793	0.0314	0.0088	0.0075	0.0048	0.0056	0.0023	0.0008	0	0.0005	0.0001	0.0001	0
4.00E-05	0.0003	0.0027	0.0156	0.068	0.0273	0.0081	0.0063	0.0038	0.0035	0.0019	0.0006	0	0.0003	0.0001	0	0
5.00E-05	0.0003	0.0015	0.0126	0.0519	0.0216	0.0068	0.0051	0.0029	0.0023	0.001	0.0006	0	0.0002	0.0001	0	0
6.00E-05	0.0001	0.0014	0.0089	0.0329	0.016	0.0046	0.004	0.0017	0.0011	0.0009	0.0002	0	0.0002	0.0001	0	0
7.00E-05	0	0.0011	0.0066	0.0217	0.0105	0.0032	0.0027	0.001	0.0008	0.0006	0	0	0.0001	0.0001	0	0
8.50E-05	0	0.0006	0.0035	0.009	0.0055	0.0016	0.0018	0.0002	0.0001	0.0005	0	0	0.0001	0	0	0
1.00E-04	0	0.0005	0.0015	0.0027	0.0024	0.001	0.0008	0.0001	0.0001	0.0001	0	0	0	0	0	0
1.30E-04	0	0	0.0001	0.0002	0.0006	0	0.0003	0	0	0	0	0	0	0	0	0
1.70E-04	0	0	0	0	0.0001	0	0	0	0	0	0	0	0	0	0	0
2.00E-04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-101

2001 Probability values for 1 hour at SSES LPZ

Probability that the X/Q is Greater than the Adjacent Quantized Level

QUANTIZED LEVEL	DIRECTION N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
0.00E+00	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
1.00E-10	0.0515	0.0592	0.0751	0.1698	0.0853	0.0466	0.0507	0.0465	0.0659	0.0674	0.1049	0.0558	0.0296	0.0235	0.0344	0.0479
3.50E-09	0.0515	0.0592	0.0751	0.1698	0.0853	0.0466	0.0507	0.0465	0.0659	0.0674	0.1049	0.0558	0.0296	0.0235	0.0344	0.0479
1.00E-08	0.0515	0.0592	0.0751	0.1698	0.0853	0.0466	0.0507	0.0465	0.0659	0.0674	0.1049	0.0558	0.0296	0.0235	0.0344	0.0479
2.50E-08	0.0515	0.0592	0.0751	0.1698	0.0853	0.0466	0.0507	0.0465	0.0659	0.0674	0.1049	0.0558	0.0296	0.0235	0.0344	0.0479
7.00E-08	0.0515	0.0592	0.0751	0.1698	0.0853	0.0466	0.0507	0.0465	0.0659	0.0674	0.1048	0.0554	0.0296	0.0235	0.0344	0.0479
1.00E-07	0.0508	0.0585	0.0751	0.1698	0.0853	0.0466	0.0506	0.0465	0.0654	0.0669	0.0998	0.0526	0.0284	0.0234	0.0344	0.0478
1.50E-07	0.0485	0.0562	0.0738	0.1697	0.0851	0.0465	0.0496	0.0457	0.0629	0.0645	0.0875	0.0495	0.0264	0.0226	0.0339	0.0476
2.20E-07	0.0482	0.0542	0.0718	0.1685	0.0846	0.0457	0.0473	0.0436	0.0596	0.0591	0.08	0.0485	0.0259	0.0225	0.0338	0.0471
3.20E-07	0.048	0.053	0.0707	0.1672	0.0835	0.0448	0.0455	0.0422	0.0575	0.0569	0.0768	0.0481	0.0258	0.0225	0.0337	0.047
4.80E-07	0.0477	0.0528	0.0698	0.1658	0.0825	0.0442	0.045	0.0417	0.0568	0.0559	0.0756	0.0476	0.0256	0.0225	0.0337	0.0469
7.00E-07	0.0474	0.0524	0.0697	0.1656	0.0818	0.0436	0.045	0.0416	0.0565	0.0558	0.0741	0.0458	0.0251	0.0224	0.0336	0.0467
1.00E-06	0.0454	0.0521	0.0695	0.1656	0.0818	0.0436	0.045	0.0415	0.0561	0.0554	0.0721	0.0445	0.0249	0.0218	0.0334	0.0466
1.50E-06	0.0453	0.052	0.0692	0.1654	0.0815	0.0434	0.0447	0.0415	0.055	0.0545	0.0706	0.042	0.0243	0.0215	0.0333	0.0462
2.00E-06	0.0451	0.0515	0.0688	0.1648	0.0813	0.043	0.0446	0.0413	0.0542	0.054	0.0681	0.0345	0.0229	0.0201	0.0316	0.0418
3.00E-06	0.0322	0.0483	0.0674	0.1633	0.081	0.0429	0.043	0.0404	0.0534	0.0523	0.0545	0.0201	0.0136	0.0116	0.0156	0.0204
4.00E-06	0.02	0.0421	0.0646	0.1624	0.0795	0.0411	0.0394	0.0385	0.0497	0.0485	0.0428	0.0137	0.0072	0.0068	0.008	0.0104
5.00E-06	0.0166	0.0367	0.0622	0.1606	0.0774	0.039	0.0362	0.0348	0.0438	0.0447	0.0374	0.0103	0.005	0.0042	0.0061	0.0078
7.00E-06	0.0129	0.0302	0.0555	0.1564	0.0747	0.0367	0.0324	0.03	0.0386	0.0373	0.0259	0.0058	0.0027	0.0024	0.005	0.0062
8.50E-06	0.0097	0.0243	0.0511	0.1536	0.0721	0.0354	0.0297	0.0264	0.034	0.0307	0.0169	0.0038	0.0019	0.0015	0.0023	0.0029
1.00E-05	0.007	0.0194	0.0473	0.1502	0.07	0.0327	0.0275	0.0241	0.0304	0.0244	0.0133	0.003	0.0017	0.0007	0.0015	0.0019
1.50E-05	0.0032	0.0112	0.0368	0.1387	0.0639	0.0281	0.0217	0.0178	0.0206	0.0118	0.0063	0.0013	0.0008	0.0002	0.0007	0.0008
2.00E-05	0.0018	0.0063	0.0301	0.1264	0.057	0.0238	0.0167	0.0124	0.0129	0.0062	0.0033	0.0002	0.0007	0.0001	0.0002	0.0005
2.50E-05	0.0014	0.0041	0.0243	0.1159	0.0492	0.0198	0.0117	0.0082	0.0086	0.0044	0.0024	0.0002	0.0006	0.0001	0.0002	0.0002
3.00E-05	0.001	0.0028	0.0202	0.1037	0.043	0.0163	0.0093	0.0064	0.0074	0.003	0.0016	0.0001	0.0005	0.0001	0.0002	0.0001
3.50E-05	0.0007	0.0024	0.0162	0.0921	0.0381	0.0132	0.0074	0.0057	0.0062	0.0023	0.0014	0	0.0005	0.0001	0.0001	0
4.00E-05	0.0006	0.0016	0.0139	0.0783	0.0346	0.011	0.0063	0.0044	0.0043	0.0016	0.0011	0	0.0003	0.0001	0	0
5.00E-05	0.0003	0.0011	0.0109	0.0588	0.0284	0.0092	0.0048	0.003	0.0024	0.001	0.0007	0	0.0002	0.0001	0	0
6.00E-05	0.0003	0.0009	0.0077	0.0393	0.0201	0.0067	0.0028	0.0017	0.0014	0.0009	0.0005	0	0.0002	0.0001	0	0
7.00E-05	0.0001	0.0008	0.0051	0.0248	0.0133	0.0046	0.0021	0.0014	0.0008	0.0005	0.0003	0	0.0001	0.0001	0	0
8.50E-05	0.0001	0.0002	0.0029	0.0107	0.0072	0.0018	0.0013	0.0003	0.0001	0.0001	0.0003	0	0.0001	0	0	0
1.00E-04	0.0001	0.0002	0.0016	0.0037	0.0034	0.0017	0.0011	0.0002	0	0	0.0001	0	0	0	0	0
1.30E-04	0	0.0001	0.0001	0.0002	0.0002	0.0001	0.0003	0	0	0	0	0	0	0	0	0
1.70E-04	0	0	0	0.0001	0	0	0	0	0	0	0	0	0	0	0	0
2.00E-04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-102

2002 Probability values for 1 hour at SSES LPZ

Probability that the X/Q is Greater than the Adjacent Quantized Level

QUANTIZED LEVEL	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
	DIRECTION															
0.00E+00	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
1.00E-10	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
3.50E-09	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
1.00E-08	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
2.50E-08	0.0499	0.072	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.083	0.1147	0.0628	0.0314	0.0204	0.0296	0.0398
7.00E-08	0.0495	0.0719	0.0827	0.1446	0.0676	0.0451	0.0424	0.0463	0.0678	0.0829	0.1131	0.0624	0.0314	0.0204	0.0296	0.0397
1.00E-07	0.0476	0.0707	0.0827	0.1446	0.0676	0.045	0.042	0.0458	0.0663	0.0807	0.1052	0.0582	0.031	0.0204	0.0295	0.0387
1.50E-07	0.045	0.0689	0.0822	0.1446	0.0674	0.0447	0.0418	0.0455	0.0648	0.076	0.0916	0.0539	0.0303	0.0202	0.0292	0.0377
2.20E-07	0.0439	0.0674	0.0807	0.1442	0.0673	0.0444	0.0415	0.0451	0.0632	0.0723	0.0838	0.0524	0.03	0.0202	0.0292	0.0376
3.20E-07	0.0435	0.0664	0.0793	0.1433	0.0667	0.044	0.041	0.0444	0.0626	0.0704	0.0817	0.0521	0.0297	0.0201	0.0292	0.0375
4.80E-07	0.0435	0.0663	0.0792	0.1431	0.0658	0.0436	0.0408	0.0441	0.0617	0.0702	0.0816	0.0512	0.0295	0.0201	0.029	0.0372
7.00E-07	0.043	0.0661	0.0792	0.1431	0.0656	0.0436	0.0407	0.044	0.0612	0.0699	0.0805	0.0496	0.0292	0.0201	0.0288	0.0365
1.00E-06	0.0423	0.0651	0.0791	0.1431	0.0656	0.0436	0.0405	0.044	0.0607	0.0694	0.0776	0.0483	0.029	0.0199	0.0287	0.0363
1.50E-06	0.0423	0.0643	0.0782	0.1429	0.0656	0.0435	0.0402	0.0436	0.0603	0.0687	0.0755	0.0473	0.0278	0.0196	0.0284	0.0357
2.00E-06	0.0422	0.0637	0.0778	0.1427	0.0652	0.0429	0.04	0.0435	0.0602	0.0682	0.074	0.0412	0.0243	0.0164	0.0257	0.0319
3.00E-06	0.0329	0.0609	0.0768	0.142	0.0649	0.0416	0.0386	0.0407	0.0558	0.0648	0.0558	0.0232	0.0139	0.0088	0.0126	0.0173
4.00E-06	0.0214	0.0508	0.0705	0.1407	0.0636	0.0399	0.0344	0.0355	0.0515	0.058	0.0407	0.0163	0.0097	0.0043	0.0068	0.0095
5.00E-06	0.0173	0.0435	0.0632	0.1383	0.0618	0.037	0.0307	0.032	0.0453	0.0503	0.0326	0.0131	0.0065	0.0033	0.0053	0.0078
7.00E-06	0.0136	0.0335	0.0563	0.1344	0.058	0.0334	0.0255	0.0269	0.0377	0.0384	0.0224	0.0071	0.004	0.0025	0.003	0.0046
8.50E-06	0.01	0.025	0.0521	0.1315	0.0552	0.0312	0.0231	0.0244	0.0323	0.0295	0.0133	0.0047	0.0031	0.0018	0.0016	0.0028
1.00E-05	0.0071	0.0206	0.0488	0.13	0.0533	0.0292	0.0209	0.0224	0.0289	0.0239	0.0096	0.0038	0.002	0.0013	0.0012	0.0021
1.50E-05	0.0029	0.0121	0.0368	0.124	0.0486	0.0251	0.0172	0.0156	0.0172	0.0104	0.004	0.0012	0.001	0.0005	0.0006	0.0009
2.00E-05	0.0021	0.0077	0.0301	0.1159	0.043	0.022	0.013	0.0116	0.0104	0.0068	0.0022	0.0006	0.0002	0.0003	0.0001	0.0005
2.50E-05	0.0012	0.0063	0.0262	0.1095	0.0361	0.0169	0.0093	0.0089	0.0081	0.0051	0.0014	0.0003	0.0001	0.0003	0.0001	0.0003
3.00E-05	0.0009	0.0052	0.0236	0.1016	0.0332	0.0146	0.0077	0.0083	0.0067	0.0033	0.0012	0.0002	0	0.0003	0.0001	0.0002
3.50E-05	0.0006	0.0044	0.0201	0.0916	0.0311	0.0135	0.0074	0.0074	0.005	0.0024	0.0008	0.0002	0	0.0002	0.0001	0.0002
4.00E-05	0.0005	0.0036	0.0166	0.0793	0.0281	0.0126	0.0067	0.0056	0.0029	0.0017	0.0005	0.0002	0	0	0.0001	0.0002
5.00E-05	0.0003	0.0029	0.0119	0.0621	0.0225	0.0104	0.0046	0.0037	0.0015	0.0009	0.0002	0.0001	0	0	0	0.0001
6.00E-05	0.0003	0.0018	0.0084	0.0432	0.0151	0.0071	0.0031	0.0018	0.001	0.0005	0.0002	0.0001	0	0	0	0
7.00E-05	0.0002	0.0015	0.0061	0.0273	0.01	0.0048	0.0023	0.0015	0.0003	0.0001	0.0002	0.0001	0	0	0	0
8.50E-05	0.0002	0.0008	0.0025	0.0107	0.0041	0.0024	0.0009	0.0008	0.0001	0.0001	0.0001	0.0001	0	0	0	0
1.00E-04	0.0001	0.0005	0.0015	0.0038	0.0017	0.0015	0.0005	0.0003	0	0.0001	0.0001	0.0001	0	0	0	0
1.30E-04	0	0.0001	0.0002	0.0003	0.0002	0.0002	0.0002	0	0	0	0	0	0	0	0	0
1.70E-04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-103

2003 Probability values for 1 hour at SSES LPZ

Probability that the X/Q is Greater than the Adjacent Quantized Level

QUANTIZED LEVEL	DIRECTION															
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
0.00E+00	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
1.00E-10	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
3.50E-09	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
1.00E-08	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
2.50E-08	0.0392	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.1004	0.0621	0.0297	0.0214	0.0287	0.0348
7.00E-08	0.039	0.0769	0.1096	0.1414	0.0673	0.0497	0.062	0.0474	0.061	0.0683	0.0996	0.0615	0.0297	0.0213	0.0287	0.0346
1.00E-07	0.0385	0.0764	0.109	0.1414	0.0673	0.0497	0.0613	0.0469	0.0603	0.0677	0.096	0.0587	0.0287	0.021	0.0274	0.0331
1.50E-07	0.0377	0.0746	0.108	0.1412	0.067	0.0494	0.0607	0.0461	0.0579	0.0656	0.09	0.0549	0.028	0.0196	0.0273	0.0323
2.20E-07	0.0373	0.0743	0.1068	0.141	0.0666	0.049	0.0603	0.0448	0.0564	0.0619	0.0854	0.053	0.0273	0.0191	0.0271	0.032
3.20E-07	0.0371	0.0739	0.1064	0.1405	0.0657	0.0489	0.0598	0.044	0.056	0.0601	0.0846	0.0527	0.0272	0.019	0.0271	0.0318
4.80E-07	0.0371	0.0737	0.1063	0.1397	0.065	0.0482	0.0595	0.0437	0.0557	0.0595	0.0843	0.0523	0.027	0.019	0.0271	0.0318
7.00E-07	0.0365	0.0737	0.106	0.1397	0.0649	0.0482	0.0595	0.0436	0.0554	0.0594	0.0829	0.0516	0.0266	0.0189	0.0266	0.0314
1.00E-06	0.0363	0.0728	0.1059	0.1397	0.0649	0.0482	0.0591	0.0435	0.0553	0.0591	0.0808	0.0513	0.0266	0.0188	0.0264	0.0314
1.50E-06	0.0361	0.0725	0.1056	0.1397	0.0649	0.0479	0.0587	0.0432	0.0553	0.0587	0.0799	0.0485	0.0255	0.0174	0.026	0.0314
2.00E-06	0.0353	0.0724	0.1055	0.1391	0.0648	0.0479	0.0583	0.043	0.0551	0.0581	0.0779	0.0416	0.0222	0.0152	0.0231	0.0292
3.00E-06	0.0286	0.0667	0.1025	0.138	0.0633	0.0463	0.0549	0.0413	0.0522	0.0558	0.0551	0.0258	0.0124	0.0098	0.0121	0.0141
4.00E-06	0.0198	0.0551	0.0928	0.1353	0.0618	0.0428	0.0481	0.0377	0.0475	0.0496	0.037	0.0182	0.0092	0.0066	0.0071	0.0086
5.00E-06	0.016	0.0499	0.0868	0.1322	0.0604	0.0392	0.0421	0.0346	0.0441	0.0438	0.0279	0.0139	0.0066	0.0048	0.0055	0.0074
7.00E-06	0.0108	0.0379	0.0753	0.1285	0.0566	0.0353	0.0364	0.0288	0.0358	0.034	0.0179	0.0081	0.0038	0.0027	0.0035	0.0051
8.50E-06	0.0075	0.0318	0.0688	0.1251	0.0542	0.0318	0.0324	0.024	0.0299	0.025	0.0106	0.0055	0.0025	0.0017	0.0022	0.0022
1.00E-05	0.0058	0.025	0.0633	0.1215	0.0512	0.029	0.0289	0.0219	0.0256	0.0189	0.008	0.0043	0.0018	0.001	0.0015	0.0013
1.50E-05	0.0024	0.0123	0.0485	0.1126	0.0446	0.0228	0.0226	0.0149	0.0146	0.0097	0.0037	0.0014	0.001	0.0006	0.0006	0.0008
2.00E-05	0.0014	0.0066	0.0358	0.0989	0.0382	0.0182	0.0165	0.01	0.0082	0.0053	0.0017	0.0007	0.0005	0.0002	0.0003	0.0006
2.50E-05	0.0013	0.0046	0.0303	0.0893	0.0302	0.0129	0.0118	0.0056	0.0059	0.0029	0.0009	0.0002	0.0005	0	0.0001	0.0003
3.00E-05	0.0008	0.003	0.0272	0.0823	0.0271	0.0114	0.0086	0.0046	0.0045	0.0023	0.0002	0.0002	0.0005	0	0	0.0001
3.50E-05	0.0005	0.0027	0.0236	0.0709	0.0229	0.0091	0.0066	0.0038	0.0033	0.0016	0	0.0001	0.0005	0	0	0.0001
4.00E-05	0.0002	0.0021	0.0209	0.0604	0.0212	0.0084	0.0053	0.003	0.0024	0.0009	0	0.0001	0.0005	0	0	0.0001
5.00E-05	0.0002	0.0014	0.0152	0.0448	0.0175	0.0066	0.0043	0.0015	0.0013	0.0007	0	0.0001	0.0003	0	0	0
6.00E-05	0.0002	0.0012	0.0109	0.0322	0.012	0.0035	0.003	0.0009	0.001	0.0005	0	0	0.0002	0	0	0
7.00E-05	0	0.001	0.0081	0.019	0.0071	0.0025	0.0018	0.0007	0.0006	0	0	0	0.0001	0	0	0
8.50E-05	0	0.0005	0.0036	0.006	0.0038	0.0012	0.0009	0.0005	0.0001	0	0	0	0.0001	0	0	0
1.00E-04	0	0.0002	0.0012	0.0023	0.0021	0.0007	0.0003	0.0002	0	0	0	0	0.0001	0	0	0
1.30E-04	0	0	0.0001	0	0	0	0.0001	0	0	0	0	0	0	0	0	0
1.70E-04	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 2.3-104

1999 - 2003 Average Probability Values for 1 hour at SSES LPZ

Probability that the X/Q is Greater than the Adjacent Quantized Level

QUANTIZED LEVEL	DIRECTION															
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW
0.00E+00	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
1.00E-10	5.13E-02	7.05E-02	9.23E-02	1.56E-01	7.49E-02	4.53E-02	4.87E-02	4.26E-02	6.18E-02	7.04E-02	1.04E-01	5.99E-02	3.03E-02	2.13E-02	3.15E-02	4.20E-02
3.50E-09	5.13E-02	7.05E-02	9.23E-02	1.56E-01	7.49E-02	4.53E-02	4.87E-02	4.26E-02	6.18E-02	7.04E-02	1.04E-01	5.99E-02	3.03E-02	2.13E-02	3.15E-02	4.20E-02
1.00E-08	5.13E-02	7.05E-02	9.23E-02	1.56E-01	7.49E-02	4.53E-02	4.87E-02	4.26E-02	6.18E-02	7.04E-02	1.04E-01	5.99E-02	3.03E-02	2.13E-02	3.15E-02	4.20E-02
2.50E-08	5.13E-02	7.05E-02	9.23E-02	1.56E-01	7.49E-02	4.53E-02	4.87E-02	4.26E-02	6.18E-02	7.04E-02	1.04E-01	5.99E-02	3.03E-02	2.13E-02	3.15E-02	4.20E-02
7.00E-08	5.12E-02	7.04E-02	9.23E-02	1.56E-01	7.49E-02	4.53E-02	4.87E-02	4.26E-02	6.18E-02	7.04E-02	1.04E-01	5.94E-02	3.03E-02	2.13E-02	3.15E-02	4.19E-02
1.00E-07	5.05E-02	6.95E-02	9.21E-02	1.56E-01	7.49E-02	4.53E-02	4.83E-02	4.24E-02	6.12E-02	6.96E-02	9.92E-02	5.58E-02	2.91E-02	2.10E-02	3.12E-02	4.14E-02
1.50E-07	4.87E-02	6.73E-02	9.09E-02	1.55E-01	7.47E-02	4.51E-02	4.78E-02	4.18E-02	5.90E-02	6.66E-02	8.87E-02	5.21E-02	2.74E-02	2.02E-02	3.09E-02	4.08E-02
2.20E-07	4.81E-02	6.57E-02	8.89E-02	1.55E-01	7.43E-02	4.45E-02	4.66E-02	4.05E-02	5.65E-02	6.29E-02	8.25E-02	5.07E-02	2.70E-02	2.00E-02	3.07E-02	4.05E-02
3.20E-07	4.78E-02	6.48E-02	8.75E-02	1.54E-01	7.37E-02	4.38E-02	4.59E-02	3.98E-02	5.54E-02	6.08E-02	8.05E-02	5.03E-02	2.68E-02	2.00E-02	3.07E-02	4.03E-02
4.80E-07	4.77E-02	6.47E-02	8.72E-02	1.53E-01	7.27E-02	4.31E-02	4.55E-02	3.95E-02	5.49E-02	6.03E-02	8.00E-02	4.97E-02	2.67E-02	2.00E-02	3.06E-02	4.02E-02
7.00E-07	4.71E-02	6.43E-02	8.71E-02	1.53E-01	7.24E-02	4.29E-02	4.54E-02	3.94E-02	5.46E-02	6.02E-02	7.89E-02	4.80E-02	2.61E-02	1.99E-02	3.04E-02	3.99E-02
1.00E-06	4.63E-02	6.35E-02	8.67E-02	1.53E-01	7.23E-02	4.29E-02	4.52E-02	3.93E-02	5.43E-02	5.97E-02	7.66E-02	4.69E-02	2.58E-02	1.94E-02	3.02E-02	3.96E-02
1.50E-06	4.62E-02	6.29E-02	8.60E-02	1.53E-01	7.22E-02	4.27E-02	4.49E-02	3.90E-02	5.36E-02	5.87E-02	7.48E-02	4.49E-02	2.50E-02	1.90E-02	3.00E-02	3.92E-02
2.00E-06	4.51E-02	6.25E-02	8.57E-02	1.52E-01	7.20E-02	4.23E-02	4.45E-02	3.87E-02	5.31E-02	5.82E-02	7.26E-02	3.77E-02	2.26E-02	1.72E-02	2.80E-02	3.52E-02
3.00E-06	3.19E-02	5.78E-02	8.38E-02	1.52E-01	7.13E-02	4.13E-02	4.26E-02	3.72E-02	5.11E-02	5.62E-02	5.57E-02	2.19E-02	1.30E-02	1.01E-02	1.43E-02	1.76E-02
4.00E-06	2.06E-02	4.76E-02	7.65E-02	1.49E-01	6.96E-02	3.88E-02	3.87E-02	3.43E-02	4.71E-02	5.06E-02	4.18E-02	1.57E-02	8.24E-03	5.80E-03	7.80E-03	9.54E-03
5.00E-06	1.66E-02	4.12E-02	7.06E-02	1.47E-01	6.74E-02	3.65E-02	3.53E-02	3.13E-02	4.24E-02	4.49E-02	3.45E-02	1.20E-02	5.86E-03	4.04E-03	5.96E-03	7.50E-03
7.00E-06	1.23E-02	3.24E-02	6.17E-02	1.42E-01	6.37E-02	3.35E-02	3.03E-02	2.61E-02	3.60E-02	3.60E-02	2.38E-02	7.02E-03	3.58E-03	2.52E-03	4.30E-03	5.38E-03
8.50E-06	9.08E-03	2.57E-02	5.61E-02	1.39E-01	6.12E-02	3.15E-02	2.73E-02	2.31E-02	3.14E-02	2.85E-02	1.50E-02	4.68E-03	2.52E-03	1.64E-03	2.34E-03	2.58E-03
1.00E-05	6.80E-03	2.10E-02	5.20E-02	1.36E-01	5.84E-02	2.91E-02	2.45E-02	2.10E-02	2.77E-02	2.27E-02	1.14E-02	3.64E-03	1.86E-03	9.19E-04	1.46E-03	1.66E-03
1.50E-05	3.04E-03	1.15E-02	3.94E-02	1.27E-01	5.26E-02	2.44E-02	1.96E-02	1.54E-02	1.78E-02	1.09E-02	5.06E-03	1.40E-03	1.00E-03	4.00E-04	6.60E-04	7.80E-04
2.00E-05	2.08E-03	6.82E-03	3.03E-02	1.15E-01	4.65E-02	2.07E-02	1.52E-02	1.07E-02	1.05E-02	6.40E-03	2.86E-03	5.40E-04	5.60E-04	2.00E-04	2.20E-04	4.80E-04
2.50E-05	1.48E-03	5.12E-03	2.53E-02	1.06E-01	3.94E-02	1.59E-02	1.10E-02	7.10E-03	7.70E-03	4.32E-03	1.90E-03	3.40E-04	4.80E-04	1.20E-04	1.20E-04	2.40E-04
3.00E-05	9.80E-04	3.84E-03	2.22E-02	9.65E-02	3.51E-02	1.35E-02	8.88E-03	5.94E-03	6.34E-03	3.06E-03	1.26E-03	2.40E-04	3.60E-04	9.99E-05	1.00E-04	1.20E-04
3.50E-05	7.40E-04	3.22E-03	1.90E-02	8.52E-02	3.15E-02	1.14E-02	7.24E-03	5.16E-03	4.92E-03	2.14E-03	9.21E-04	1.60E-04	3.40E-04	8.00E-05	6.00E-05	7.98E-05
4.00E-05	5.00E-04	2.56E-03	1.62E-02	7.34E-02	2.82E-02	1.04E-02	6.18E-03	3.98E-03	3.22E-03	1.40E-03	6.20E-04	9.99E-05	2.40E-04	4.01E-05	1.99E-05	7.98E-05
5.00E-05	3.20E-04	1.84E-03	1.23E-02	5.63E-02	2.32E-02	8.68E-03	4.88E-03	2.64E-03	1.88E-03	8.20E-04	4.21E-04	7.99E-05	1.40E-04	4.01E-05	0.00E+00	4.00E-05
6.00E-05	2.80E-04	1.44E-03	8.84E-03	3.74E-02	1.60E-02	5.94E-03	3.24E-03	1.42E-03	1.10E-03	6.00E-04	2.20E-04	1.99E-05	1.20E-04	4.01E-05	0.00E+00	2.00E-05
7.00E-05	1.20E-04	1.14E-03	6.54E-03	2.34E-02	1.04E-02	4.22E-03	2.30E-03	1.08E-03	6.81E-04	2.81E-04	1.40E-04	1.99E-05	6.00E-05	4.01E-05	0.00E+00	2.00E-05
8.50E-05	9.99E-05	5.80E-04	3.16E-03	9.44E-03	5.46E-03	2.22E-03	1.30E-03	5.00E-04	1.40E-04	1.61E-04	7.99E-05	1.99E-05	6.00E-05	0.00E+00	0.00E+00	2.00E-05
1.00E-04	5.99E-05	3.80E-04	1.52E-03	3.42E-03	2.72E-03	1.56E-03	7.41E-04	2.80E-04	8.02E-05	6.01E-05	3.99E-05	1.99E-05	1.99E-05	0.00E+00	0.00E+00	2.00E-05
1.30E-04	2.00E-05	3.99E-05	1.60E-04	3.00E-04	4.01E-04	1.20E-04	2.40E-04	2.00E-05	4.01E-05	2.00E-05	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
1.70E-04	0.00E+00	0.00E+00	2.00E-05	4.00E-05	4.02E-05	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
2.00E-04	0.00E+00	0.00E+00	2.00E-05	2.00E-05	2.00E-05	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
2.50E-04	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
3.00E-04	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
3.50E-04	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
4.00E-04	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
5.00E-04	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
6.00E-04	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
7.00E-04	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
8.50E-04	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
1.00E-03	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
1.40E-03	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
2.00E-03	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
3.00E-03	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
4.00E-03	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00

TABLE 2.3-105

SUMMARY OF LONG-TERM X/Q (SEC/M3) RESULTS AT 4827 METER LPZ

Period of Record	USNRC Reg. Guide1.145 Interpolation Methodology								Annual Average	
	0-8 hours		8-24 hours		24-96 hours		96-720 hours		8760 hours	
	0.5%*	50%**	0.5%*	50%**	0.5%*	50%**	0.5%*	50%**	Direction Dependent	Direction Independent
1999	5.4E-05	4.6E-06	3.8E-05	3.7E-06	1.8E-05	2.3E-06	6.3E-06	1.1E-06	1.7E-06	4.8E-07
2000	4.9E-05	4.8E-06	3.5E-05	3.8E-06	1.7E-05	2.3E-06	6.0E-06	1.1E-06	1.7E-06	4.8E-07
2001	5.0E-05	5.4E-06	3.6E-05	4.3E-06	1.7E-05	2.6E-06	6.1E-06	1.3E-06	1.7E-06	5.3E-07
2002	5.0E-05	4.5E-06	3.6E-05	3.6E-06	1.7E-05	2.2E-06	6.1E-06	1.1E-06	1.7E-06	4.5E-07
2003	4.6E-05	4.7E-06	3.3E-05	3.7E-06	1.6E-05	2.2E-06	5.9E-06	1.1E-06	1.7E-06	4.4E-07
5-year Combined	4.9E-05	4.8E-06	3.5E-05	3.8E-06	1.7E-05	2.3E-06	6.1E-06	1.1E-06	1.7E-06	4.7E-07

*direction dependent values (see Figure 2.3-10)
**direction independent values (see Figure 2.3-10)

TABLE 2.3-106
DISTANCES AND TERRAIN/RECIRCULATION CORRECTION FACTORS
FOR SSES 2003 LAND USE CENSUS LOCATIONS

RESIDENCE			GARDEN		
AFFECTED SECTOR	**MILES**	**Terrain Correction Factor**	**AFFECTED SECTOR**	**MILES**	**Terrain Correction Factor**
N	1.3	2.15	N	3.2	2.19
NNE	1	2.50	NNE	2.3	2.55
NE	0.9	2.33	NE	2.7	2.47
ENE	2.1	2.42	ENE	2.4	2.48
E	1.4	2.09	E	1.8	2.07
ESE	0.5	2.58	ESE	2.5	2.00
SE	0.5	2.43	SE	0.6	2.44
SSE	0.6	2.71	SSE	1.5	2.44
S	1	2.46	S	1.1	2.43
SSW	0.9	2.39	SSW	1.2	2.35
SW	1.5	2.14	SW	1.9	2.11
WSW	1.3	2.32	WSW	1.3	2.32
W	1.2	2.18	W	1.2	2.18
WNW	0.8	2.74	WNW		
NW	0.8	3.30	NW	1.8	3.06
NNW	0.6	2.53	NNW	4	2.40
PRODUCTION ANIMAL			**DAIRY ANIMAL**		
AFFECTED SECTOR	**MILES**	**Terrain Correction Factor**	**AFFECTED SECTOR**	**MILES**	**Terrain Correction Factor**
NNE	2.3	2.55	E	4.5	1.80
ENE	2.4	2.48	ESE	2.7	1.96
E	1.4	2.09	ESE	4.2	1.58
SSW	3	2.35	SSW	3	2.11
SSW	3.5	1.88	SSW	3.1	2.06
WSW	1.7	2.34	SSW	3.5	1.88
NW	1.8	3.06	SSW	14.01	1.03
			WSW	1.7	2.34
			W	5	1.46
			NNW	4.2	2.4

Distances to the nearest garden, residence, dairy animal and production animal in each of the affected sectors was provided by the 2003 SSES Land Use Census. The terrain/recirculation correction factors listed for the distances in the above tables were mathematically interpolated from the terrain/recirculation factors quoted for standard distances in the SSES Final Safety Analysis Report.

Table 2.3-107

1999 AVERAGE RELATIVE CONCENTRATION (sec/meter^3) AND DEPOSITION (meter^{-2}) ESTIMATES AT THE SITE BOUNDARY					
Affected Sector		Relative Concentration (sec/meter^3)			Deposition
	Distance (miles	No Decay Undepleted	2.26 Days of Decay Undepleted	8.0 Days of Decay Depleted	D/Q (meter^{-2})
N	0.59	5.57E-06	5.55E-06	5.03E-06	1.75E-08
NNE	0.78	3.88E-06	3.86E-06	3.43E-06	1.51E-08
NE	0.7	4.40E-06	4.39E-06	3.93E-06	2.70E-08
ENE	0.86	1.50E-06	1.50E-06	1.32E-06	1.29E-08
E	0.8	8.48E-07	8.46E-07	7.50E-07	6.68E-09
ESE	0.5	1.13E-06	1.13E-06	1.03E-06	9.26E-09
SE	0.43	2.39E-06	2.39E-06	2.21E-06	1.97E-08
SSE	0.41	3.14E-06	3.14E-06	2.91E-06	2.88E-08
S	0.38	6.71E-06	6.70E-06	6.25E-06	4.66E-08
SSW	0.39	1.07E-05	1.07E-05	9.96E-06	5.41E-08
SW	0.61	1.13E-05	1.13E-05	1.02E-05	2.86E-08
WSW	1.22	1.28E-05	1.27E-05	1.10E-05	1.66E-08
W	1.03	7.68E-06	7.61E-06	6.67E-06	1.00E-08
WNW	0.61	1.04E-05	1.04E-05	9.38E-06	1.72E-08
NW	0.66	7.44E-06	7.40E-06	6.67E-06	1.70E-08
NNW	0.59	5.14E-06	5.12E-06	4.64E-06	1.42E-08

The above values were calculated using the XDCALC atmospheric dispersion model with terrain/recirculation factors included.

Table 2.3-108

2000 AVERAGE RELATIVE CONCENTRATION (sec/meter^3) AND DEPOSITION (meter^{-2}) ESTIMATES AT THE SITE BOUNDARY					
Affected Sector		Relative Concentration (sec/meter^3)			Deposition
	Distance (miles	No Decay Undepleted	2.26 Days of Decay Undepleted	8.0 Days of Decay Depleted	D/Q (meter^{-2})
N	0.59	6.83E-06	6.80E-06	6.16E-06	2.10E-08
NNE	0.78	4.35E-06	4.33E-06	3.86E-06	1.51E-08
NE	0.7	4.17E-06	4.16E-06	3.73E-06	2.61E-08
ENE	0.86	1.40E-06	1.39E-06	1.23E-06	1.14E-08
E	0.8	8.89E-07	8.86E-07	7.86E-07	6.31E-09
ESE	0.5	1.57E-06	1.56E-06	1.43E-06	1.23E-08
SE	0.43	2.60E-06	2.60E-06	2.40E-06	2.20E-08
SSE	0.41	4.09E-06	4.09E-06	3.79E-06	3.52E-08
S	0.38	6.21E-06	6.20E-06	5.78E-06	4.11E-08
SSW	0.39	1.22E-05	1.22E-05	1.14E-05	4.90E-08
SW	0.61	1.24E-05	1.24E-05	1.12E-05	2.90E-08
WSW	1.22	1.24E-05	1.23E-05	1.07E-05	1.65E-08
W	1.03	7.62E-06	7.55E-06	6.61E-06	1.03E-08
WNW	0.61	8.27E-06	8.23E-06	7.45E-06	1.55E-08
NW	0.66	8.40E-06	8.36E-06	7.53E-06	1.68E-08
NNW	0.59	5.99E-06	5.96E-06	5.41E-06	1.41E-08
The above values were calculated using the XDCALC atmospheric dispersion model with terrain/recirculation factors included.					

TABLE 2.3-109

2001 AVERAGE RELATIVE CONCENTRATION (sec/meter^3) AND DEPOSITION (meter^{-2}) ESTIMATES AT THE SITE BOUNDARY					
Affected Sector		Relative Concentration (sec/meter^3)			Deposition
	Distance (miles	No Decay Undepleted	2.26 Days of Decay Undepleted	8.0 Days of Decay Depleted	D/Q (meter^{-2})
N	0.59	7.27E-06	7.24E-06	6.56E-06	2.22E-08
NNE	0.78	4.18E-06	4.16E-06	3.70E-06	1.52E-08
NE	0.7	4.34E-06	4.33E-06	3.87E-06	2.77E-08
ENE	0.86	1.35E-06	1.34E-06	1.19E-06	1.16E-08
E	0.8	7.05E-07	7.03E-07	6.24E-07	5.37E-09
ESE	0.5	1.17E-06	1.17E-06	1.07E-06	8.69E-09
SE	0.43	2.89E-06	2.88E-06	2.66E-06	2.37E-08
SSE	0.41	3.85E-06	3.84E-06	3.56E-06	3.06E-08
S	0.38	6.07E-06	6.06E-06	5.65E-06	3.90E-08
SSW	0.39	1.01E-05	1.01E-05	9.35E-06	4.23E-08
SW	0.61	1.07E-05	1.06E-05	9.60E-06	2.23E-08
WSW	1.22	1.37E-05	1.36E-05	1.18E-05	1.74E-08
W	1.03	8.94E-06	8.86E-06	7.76E-06	1.12E-08
WNW	0.61	1.09E-05	1.08E-06	9.77E-06	1.79E-08
NW	0.66	8.76E-06	8.72E-06	7.85E-06	1.94E-08
NNW	0.59	7.07E-06	7.04E-06	6.38E-06	1.80E-08
The above values were calculated using the XDCALC atmospheric dispersion model with terrain/recirculation factors included.					

TABLE 2.3-110

2002 AVERAGE RELATIVE CONCENTRATION (sec/meter^3) AND DEPOSITION (meter^{-2}) ESTIMATES AT THE SITE BOUNDARY					
Affected Sector		Relative Concentration (sec/meter^3)			Deposition
	Distance (miles	No Decay Undepleted	2.26 Days of Decay Undepleted	8.0 Days of Decay Depleted	D/Q (meter^{-2})
N	0.59	6.61E-06	6.58E-06	5.97E-06	2.28E-08
NNE	0.78	4.44E-06	4.43E-06	3.94E-06	1.87E-08
NE	0.7	4.03E-06	4.02E-06	3.60E-06	3.03E-08
ENE	0.86	1.58E-06	1.57E-06	1.39E-06	1.28E-08
E	0.8	9.13E-07	9.10E-07	8.08E-07	6.69E-09
ESE	0.5	1.30E-06	1.30E-06	1.19E-06	1.07E-08
SE	0.43	2.10E-06	2.10E-06	1.94E-06	1.90E-08
SSE	0.41	3.28E-06	3.28E-06	3.04E-06	2.91E-08
S	0.38	5.71E-06	5.70E-06	5.31E-06	3.78E-08
SSW	0.39	1.24E-05	1.23E-05	1.15E-05	5.14E-08
SW	0.61	1.13E-05	1.13E-05	1.02E-05	2.45E-08
WSW	1.22	1.26E-05	1.25E-05	1.08E-05	1.48E-08
W	1.03	6.95E-06	6.89E-06	6.04E-06	8.88E-09
WNW	0.61	1.02E-05	1.02E-05	9.19E-06	1.74E-08
NW	0.66	7.22E-06	7.18E-06	6.46E-06	1.63E-08
NNW	0.59	6.63E-06	6.61E-06	5.99E-06	1.79E-08

The above values were calculated using the XDCALC atmospheric dispersion model with terrain/recirculation factors included.

TABLE 2.3-111

2003 AVERAGE RELATIVE CONCENTRATION (sec/meter^3) AND DEPOSITION (meter^{-2}) ESTIMATES AT THE SITE BOUNDARY					
Affected Sector		Relative Concentration (sec/meter^3)			Deposition
	Distance (miles	No Decay Undepleted	2.26 Days of Decay Undepleted	8.0 Days of Decay Depleted	D/Q (meter^{-2})
N	0.59	6.10E-06	6.08E-06	5.51E-06	2.05E-08
NNE	0.78	3.87E-06	3.86E-06	3.43E-06	1.54E-08
NE	0.7	3.70E-06	3.69E-06	3.30E-06	2.65E-08
ENE	0.86	1.61E-06	1.61E-06	1.42E-06	1.27E-08
E	0.8	9.09E-07	9.06E-07	8.04E-07	6.37E-09
ESE	0.5	1.33E-06	1.33E-06	1.22E-06	1.12E-08
SE	0.43	2.09E-06	2.09E-06	1.93E-06	1.84E-08
SSE	0.41	2.80E-06	2.80E-06	2.59E-06	2.55E-08
S	0.38	4.69E-06	4.69E-06	4.37E-06	2.97E-08
SSW	0.39	1.28E-05	1.28E-05	1.19E-06	5.49E-08
SW	0.61	1.45E-05	1.45E-05	1.31E-05	3.25E-08
WSW	1.22	1.10E-05	1.09E-05	9.46E-06	1.45E-08
W	1.03	6.36E-06	6.31E-06	5.53E-06	8.85E-09
WNW	0.61	9.16E-06	9.12E-06	8.25E-06	1.91E-08
NW	0.66	9.36E-06	9.32E-06	8.39E-06	2.37E-08
NNW	0.59	6.10E-06	6.08E-06	5.51E-06	1.83E-08
The above values were calculated using the XDCALC atmospheric dispersion model with terrain/recirculation factors included.					

TABLE 2.3-112

1999 - 2003 AVERAGE RELATIVE CONCENTRATION (sec/meter^3) AND DEPOSITION (meter^{-2}) ESTIMATES AT THE SITE BOUNDARY

Affected Sector		Relative Concentration (sec/meter^3)			Deposition
	Distance (miles	No Decay Undepleted	2.26 Days of Decay Undepleted	8.0 Days of Decay Depleted	D/Q (meter^{-2})
N	0.59	6.47E-06	6.45E-06	5.85E-06	2.08E-08
NNE	0.78	4.15E-06	4.13E-06	3.67E-06	1.59E-08
NE	0.7	4.13E-06	4.12E-06	3.69E-06	2.75E-08
ENE	0.86	1.49E-06	1.48E-06	1.31E-06	1.23E-08
E	0.8	8.53E-07	8.50E-07	7.55E-07	6.28E-09
ESE	0.5	1.30E-06	1.30E-06	1.19E-06	1.05E-08
SE	0.43	2.42E-06	2.41E-06	2.23E-06	2.06E-08
SSE	0.41	3.43E-06	3.43E-06	3.18E-06	2.98E-08
S	0.38	5.88E-06	5.87E-06	5.47E-06	3.88E-08
SSW	0.39	1.16E-05	1.16E-05	1.08E-05	5.03E-08
SW	0.61	1.21E-05	1.20E-05	1.09E-05	2.74E-08
WSW	1.22	1.25E-05	1.24E-05	1.07E-05	1.60E-08
W	1.03	7.51E-06	7.45E-06	6.52E-06	9.85E-09
WNW	0.61	9.78E-06	9.73E-06	8.81E-06	1.74E-08
NW	0.66	8.24E-06	8.19E-06	7.38E-06	1.86E-08
NNW	0.59	6.19E-06	6.16E-06	5.59E-06	1.65E-08

The above values were calculated using the XDCALC atmospheric dispersion model with terrain/recirculation factors included.

TABLE 2.3-113

1999 AVERAGE RELATIVE CONCENTRATION (sec/meter^3) AND DEPOSITION (meter^{-2}) ESTIMATES AT THE EXCLUSION AREA BOUNDARY					
Affected Sector		Relative Concentration (sec/meter^3)			Deposition
	Distance (miles	No Decay Undepleted	2.26 Days of Decay Undepleted	8.0 Days of Decay Depleted	D/Q (meter^{-2})
N	0.34	1.23E-05	1.23E-05	1.15E-05	4.43E-08
NNE	0.34	1.09E-05	1.09E-05	1.02E-05	5.21E-08
NE	0.34	1.18E-05	1.18E-05	1.10E-05	8.56E-08
ENE	0.34	5.60E-06	5.60E-06	5.25E-06	5.80E-08
E	0.34	2.98E-06	2.98E-06	2.80E-06	2.81E-08
ESE	0.34	2.16E-06	2.16E-06	2.03E-06	1.92E-08
SE	0.34	3.39E-06	3.39E-06	3.18E-06	2.94E-08
SSE	0.34	4.01E-06	4.00E-06	3.76E-06	3.82E-08
S	0.34	7.75E-06	7.74E-06	7.27E-06	5.51E-08
SSW	0.34	1.32E-05	1.32E-05	1.24E-05	6.85E-08
SW	0.34	2.50E-05	2.49E-05	2.34E-05	7.10E-08
WSW	0.34	6.81E-05	6.80E-05	6.38E-05	1.06E-07
W	0.34	4.22E-05	4.22E-05	3.96E-05	6.73E-08
WNW	0.34	2.38E-05	2.37E-05	2.23E-05	4.43E-08
NW	0.34	1.96E-05	1.96E-05	1.84E-05	5.20E-08
NNW	0.34	1.03E-05	1.03E-05	9.68E-06	3.23E-08
The above values were calculated using the XDCALC atmospheric dispersion model with terrain/recirculation factors included.					

TABLE 2.3-114

2000 AVERAGE RELATIVE CONCENTRATION (sec/meter^3) AND DEPOSITION (meter^{-2}) ESTIMATES AT THE EXCLUSION AREA BOUNDARY					
Affected Sector		Relative Concentration (sec/meter^3)			Deposition
	Distance (miles	No Decay Undepleted	2.26 Days of Decay Undepleted	8.0 Days of Decay Depleted	D/Q (meter^{-2})
N	0.34	1.50E-05	1.50E-05	1.41E-05	5.30E-08
NNE	0.34	1.23E-05	1.23E-05	1.15E-05	5.22E-08
NE	0.34	1.14E-05	1.13E-05	1.06E-05	8.26E-08
ENE	0.34	5.29E-06	5.29E-06	4.96E-06	5.14E-08
E	0.34	3.16E-06	3.15E-06	2.96E-06	2.66E-08
ESE	0.34	3.00E-06	3.00E-06	2.82E-06	2.55E-08
SE	0.34	3.69E-06	3.69E-06	3.47E-06	3.30E-08
SSE	0.34	5.22E-06	5.22E-06	4.89E-06	4.66E-08
S	0.34	7.17E-06	7.17E-06	6.72E-06	4.85E-08
SSW	0.34	1.50E-05	1.50E-05	1.41E-05	6.20E-08
SW	0.34	2.74E-05	2.74E-05	2.57E-05	7.20E-08
WSW	0.34	6.45E-05	6.45E-05	6.05E-05	1.05E-07
W	0.34	4.12E-05	4.11E-05	3.86E-05	6.93E-08
WNW	0.34	1.88E-05	1.87E-05	1.76E-05	4.00E-08
NW	0.34	2.21E-05	2.21E-05	2.07E-05	5.14E-08
NNW	0.34	1.19E-05	1.19E-05	1.12E-05	3.22E-08

The above values were calculated using the XDCALC atmospheric dispersion model with terrain/recirculation factors included.

TABLE 2.3-115

2001 AVERAGE RELATIVE CONCENTRATION (sec/meter^3) AND DEPOSITION (meter^{-2}) ESTIMATES AT THE EXCLUSION AREA BOUNDARY					
Affected Sector		Relative Concentration (sec/meter^3)			Deposition
	Distance (miles	No Decay Undepleted	2.26 Days of Decay Undepleted	8.0 Days of Decay Depleted	D/Q (meter^{-2})
N	0.34	1.61E-05	1.60E-05	1.51E-05	5.61E-08
NNE	0.34	1.17E-05	1.17E-05	1.10E-05	5.26E-08
NE	0.34	1.19E-05	1.18E-05	1.11E-05	8.78E-08
ENE	0.34	5.14E-06	5.14E-06	4.82E-06	5.23E-08
E	0.34	2.51E-06	2.51E-06	2.35E-06	2.26E-08
ESE	0.34	2.23E-06	2.23E-06	2.09E-06	1.80E-08
SE	0.34	4.12E-06	4.12E-06	3.86E-06	3.54E-08
SSE	0.34	4.90E-06	4.90E-06	4.60E-06	4.05E-08
S	0.34	7.01E-06	7.00E-06	6.57E-06	4.61E-08
SSW	0.34	1.23E-05	1.23E-05	1.16E-05	5.35E-08
SW	0.34	2.34E-05	2.34E-05	2.20E-05	5.53E-08
WSW	0.34	7.16E-05	7.15E-05	6.71E-05	1.11E-07
W	0.34	4.83E-05	4.83E-05	4.53E-05	7.56E-08
WNW	0.34	2.44E-05	2.43E-05	2.28E-05	4.62E-08
NW	0.34	2.29E-05	2.29E-05	2.15E-05	5.93E-08
NNW	0.34	1.40E-05	1.40E-05	1.31E-05	4.10E-08
The above values were calculated using the XDCALC atmospheric dispersion model with terrain/recirculation factors included.					

TABLE 2.3-116

2002 AVERAGE RELATIVE CONCENTRATION (sec/meter^3) AND DEPOSITION (meter^{-2}) ESTIMATES AT THE EXCLUSION AREA BOUNDARY					
Affected Sector		Relative Concentration (sec/meter^3)			Deposition
	Distance (miles	No Decay Undepleted	2.26 Days of Decay Undepleted	8.0 Days of Decay Depleted	D/Q (meter^{-2})
N	0.34	1.45E-05	1.45E-05	1.36E-05	5.77E-08
NNE	0.34	1.25E-05	1.25E-05	1.17E-05	6.47E-08
NE	0.34	1.10E-05	1.10E-05	1.03E-05	9.60E-08
ENE	0.34	5.93E-06	5.93E-06	5.56E-06	5.79E-08
E	0.34	3.20E-06	3.20E-06	3.00E-06	2.82E-08
ESE	0.34	2.48E-06	2.48E-06	2.32E-06	2.22E-08
SE	0.34	2.99E-06	2.99E-06	2.81E-06	2.84E-08
SSE	0.34	4.19E-06	4.19E-06	3.93E-06	3.86E-08
S	0.34	6.58E-06	6.57E-06	6.17E-06	4.47E-08
SSW	0.34	1.52E-05	1.52E-05	1.42E-05	6.51E-08
SW	0.34	2.51E-05	2.51E-05	2.36E-05	6.08E-08
WSW	0.34	6.65E-05	6.64E-05	6.24E-05	9.45E-08
W	0.34	3.72E-05	3.71E-05	3.49E-05	5.98E-08
WNW	0.34	2.32E-05	2.31E-05	2.17E-05	4.47E-08
NW	0.34	1.91E-05	1.90E-05	1.79E-05	4.97E-08
NNW	0.34	1.33E-05	1.33E-05	1.25E-05	4.09E-08

The above values were calculated using the XDCALC atmospheric dispersion model with terrain/recirculation factors included.

TABLE 2.3-117

2003 AVERAGE RELATIVE CONCENTRATION (sec/meter^3) AND DEPOSITION (meter^{-2}) ESTIMATES AT THE EXCLUSION AREA BOUNDARY

Affected Sector		Relative Concentration (sec/meter^3)			Deposition
	Distance (miles	No Decay Undepleted	2.26 Days of Decay Undepleted	8.0 Days of Decay Depleted	D/Q (meter^{-2})
N	0.34	1.34E-05	1.34E-05	1.26E-05	5.19E-08
NNE	0.34	1.08E-05	1.08E-05	1.02E-05	5.33E-08
NE	0.34	1.00E-05	1.00E-05	9.38E-06	8.40E-08
ENE	0.34	5.94E-06	5.93E-06	5.57E-06	5.73E-08
E	0.34	3.18E-06	3.18E-06	2.98E-06	2.68E-08
ESE	0.34	2.53E-06	2.53E-06	2.38E-06	2.33E-08
SE	0.34	2.97E-06	2.97E-06	2.79E-06	2.75E-08
SSE	0.34	3.56E-06	3.56E-06	3.34E-06	3.38E-08
S	0.34	5.41E-06	5.40E-06	5.07E-06	3.51E-08
SSW	0.34	1.57E-05	1.57E-05	1.47E-05	6.96E-08
SW	0.34	3.21E-05	3.21E-05	3.01E-05	8.07E-08
WSW	0.34	5.65E-05	5.65E-05	5.30E-05	9.25E-08
W	0.34	3.37E-05	3.37E-05	3.16E-05	5.96E-08
WNW	0.34	2.05E-05	2.05E-05	1.92E-05	4.93E-08
NW	0.34	2.43E-05	2.43E-05	2.28E-05	7.25E-08
NNW	0.34	1.21E-05	1.21E-05	1.14E-05	4.18E-08

The above values were calculated using the XDCALC atmospheric dispersion model with terrain/recirculation factors included.

TABLE 2.3-118

1999 - 2003 AVERAGE RELATIVE CONCENTRATION (sec/meter^3) AND DEPOSITION (meter^{-2}) ESTIMATES AT THE EXCLUSION AREA BOUNDARY					
Affected Sector		Relative Concentration (sec/meter^3)			Deposition
	Distance (miles	No Decay Undepleted	2.26 Days of Decay Undepleted	8.0 Days of Decay Depleted	D/Q (meter^{-2})
N	0.34	1.43E-05	1.43E-05	1.34E-05	5.26E-08
NNE	0.34	1.16E-05	1.16E-05	1.09E-05	5.50E-08
NE	0.34	1.12E-05	1.12E-05	1.05E-05	8.72E-08
ENE	0.34	5.58E-06	5.58E-06	5.23E-06	5.54E-08
E	0.34	3.01E-06	3.00E-06	2.82E-06	2.65E-08
ESE	0.34	2.48E-06	2.48E-06	2.33E-06	2.16E-08
SE	0.34	3.43E-06	3.43E-06	3.22E-06	3.08E-08
SSE	0.34	4.38E-06	4.37E-06	4.11E-06	3.96E-08
S	0.34	6.78E-05	6.78E-06	6.36E-06	4.59E-08
SSW	0.34	1.43E-05	1.43E-05	1.34E-05	6.37E-08
SW	0.34	2.66E-05	2.66E-05	2.49E-05	6.80E-08
WSW	0.34	6.54E-05	6.54E-05	6.14E-05	1.02E-07
W	0.34	4.05E-05	4.05E-05	3.80E-05	6.64E-08
WNW	0.34	2.21E-05	2.21E-05	2.07E-05	4.49E-08
NW	0.34	2.16E-05	2.16E-05	2.02E-05	5.70E-08
NNW	0.34	1.23E-05	1.23E-05	1.16E-05	3.76E-08
The above values were calculated using the XDCALC atmospheric dispersion model with terrain/recirculation factors included.					

TABLE 2.3-119

1999 ATMOSPHERIC DISPERSION ESTIMATES
FOR NEAREST RESIDENCE AND GARDEN*

NEAREST RESIDENCE WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
1	N	1.3	1.72E-06	1.71E-06	1.47E-06	4.44E-09
2	NNE	1	2.85E-06	2.83E-06	2.48E-06	1.04E-08
3	NE	0.9	3.07E-06	3.05E-06	2.69E-06	1.77E-08
4	ENE	2.1	4.03E-07	4.01E-07	3.32E-07	3.06E-09
5	E	1.4	3.28E-07	3.27E-07	2.80E-07	2.30E-09
6	ESE	0.5	1.13E-06	1.13E-06	1.03E-06	9.24E-09
7	SE	0.5	1.90E-06	1.90E-06	1.74E-06	1.51E-08
8	SSE	0.6	1.87E-06	1.87E-06	1.69E-06	1.58E-08
9	S	1	1.69E-06	1.68E-06	1.47E-06	9.42E-09
10	SSW	0.9	3.23E-06	3.21E-06	2.83E-06	1.35E-08
11	SW	1.5	3.06E-06	3.03E-06	2.59E-06	6.42E-09
12	WSW	1.3	1.16E-05	1.15E-05	9.90E-06	1.49E-08
13	W	1.2	5.96E-06	5.90E-06	5.12E-06	7.54E-09
14	WNW	0.8	7.13E-06	7.08E-06	6.30E-06	1.12E-08
15	NW	0.8	6.11E-06	6.06E-06	5.40E-06	1.34E-08
16	NNW	0.6	5.02E-06	5.00E-06	4.52E-06	1.38E-08

NEAREST GARDEN WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
1	N	3.2	4.61E-07	4.52E-07	3.61E-07	9.99E-10
2	NNE	2.3	8.50E-07	8.40E-07	6.92E-07	2.73E-09
3	NE	2.7	6.27E-07	6.21E-07	5.02E-07	3.07E-09
4	ENE	2.1	4.03E-07	4.01E-07	3.32E-07	3.06E-09
5	E	1.8	2.21E-07	2.20E-07	1.84E-07	1.52E-09
6	ESE	2.5	7.59E-08	7.54E-08	6.14E-08	4.97E-10
7	SE	0.6	1.46E-06	1.46E-06	1.32E-06	1.11E-08
8	SSE	1.5	4.28E-07	4.26E-07	3.63E-07	2.99E-09
9	S	1.1	1.45E-06	1.44E-06	1.25E-06	7.92E-09
10	SSW	1.2	2.07E-06	2.06E-06	1.78E-06	8.11E-09
11	SW	1.9	2.15E-06	2.12E-06	1.78E-06	4.38E-09
12	WSW	1.3	1.16E-05	1.15E-05	9.90E-06	1.49E-08
13	W	1.2	5.96E-06	5.90E-06	5.12E-06	7.54E-09
14	WNW	(1)	-	-	-	-
15	NW	1.8	1.71E-06	1.69E-06	1.43E-06	3.19E-09
16	NNW	4	2.87E-07	2.80E-07	2.18E-07	5.10E-10

(1) No garden within 5 miles for this sector

***Locations use the 2003 Land Use Census Locations**

TABLE 2.3-120

2000 ATMOSPHERIC DISPERSION ESTIMATES
FOR NEAREST RESIDENCE AND GARDEN*

NEAREST RESIDENCE WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
1	N	1.3	2.11E-06	2.10E-06	1.81E-06	5.31E-09
2	NNE	1	3.19E-06	3.18E-06	2.78E-06	1.04E-08
3	NE	0.9	2.89E-06	2.88E-06	2.54E-06	1.71E-08
4	ENE	2.1	3.66E-07	3.64E-07	3.01E-07	2.71E-09
5	E	1.4	3.42E-07	3.40E-07	2.91E-07	2.17E-09
6	ESE	0.5	1.56E-06	1.56E-06	1.43E-06	1.23E-08
7	SE	0.5	2.07E-06	2.06E-06	1.89E-06	1.69E-08
8	SSE	0.6	2.44E-06	2.44E-06	2.20E-06	1.94E-08
9	S	1	1.56E-06	1.55E-06	1.35E-06	8.29E-09
10	SSW	0.9	3.69E-06	3.67E-06	3.24E-06	1.22E-08
11	SW	1.5	3.33E-06	3.30E-06	2.82E-06	6.51E-09
12	WSW	1.3	1.13E-05	1.12E-05	9.65E-06	1.49E-08
13	W	1.2	5.92E-06	5.87E-06	5.09E-06	7.77E-09
14	WNW	0.8	5.68E-06	5.64E-06	5.02E-06	1.00E-08
15	NW	0.8	6.90E-06	6.85E-06	6.10E-06	1.32E-08
16	NNW	0.6	5.84E-06	5.82E-06	5.27E-06	1.37E-08

NEAREST GARDEN WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
1	N	3.2	5.63E-07	5.53E-07	4.41E-07	1.20E-09
2	NNE	2.3	9.52E-07	9.41E-07	7.75E-07	2.74E-09
3	NE	2.7	5.76E-07	5.70E-07	4.62E-07	2.96E-09
4	ENE	2.1	3.66E-07	3.64E-07	3.01E-07	2.71E-09
5	E	1.8	2.30E-07	2.29E-07	1.92E-07	1.44E-09
6	ESE	2.5	1.04E-07	1.03E-07	8.43E-08	6.62E-10
7	SE	0.6	1.58E-06	1.58E-06	1.43E-06	1.25E-08
8	SSE	1.5	5.59E-07	5.56E-07	4.73E-07	3.65E-09
9	S	1.1	1.34E-06	1.33E-06	1.16E-06	6.97E-09
10	SSW	1.2	2.37E-06	2.36E-06	2.04E-06	7.34E-09
11	SW	1.9	2.33E-06	2.31E-06	1.93E-06	4.44E-09
12	WSW	1.3	1.13E-05	1.12E-05	9.65E-06	1.49E-08
13	W	1.2	5.92E-06	5.87E-06	5.09E-06	7.77E-09
15	NW	1.8	1.93E-06	1.91E-06	1.61E-06	3.15E-09
16	NNW	4	3.36E-07	3.27E-07	2.55E-07	5.09E-10

(1) No garden within 5 miles for this sector

*****Locations use the 2003 Land Use Census Locations

TABLE 2.3-121

2001 ATMOSPHERIC DISPERSION ESTIMATES
FOR NEAREST RESIDENCE AND GARDEN*

NEAREST RESIDENCE WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
1	N	1.3	2.24E-06	2.22E-06	1.92E-06	5.62E-09
2	NNE	1	3.07E-06	3.06E-06	2.68E-06	1.05E-08
3	NE	0.9	3.00E-06	2.99E-06	2.63E-06	1.82E-08
4	ENE	2.1	3.52E-07	3.50E-07	2.89E-07	2.75E-09
5	E	1.4	2.71E-07	2.70E-07	2.31E-07	1.85E-09
6	ESE	0.5	1.17E-06	1.16E-06	1.07E-06	8.67E-09
7	SE	0.5	2.28E-06	2.28E-06	2.09E-06	1.82E-08
8	SSE	0.6	2.30E-06	2.29E-06	2.07E-06	1.68E-08
9	S	1	1.51E-06	1.51E-06	1.32E-06	7.88E-09
10	SSW	0.9	3.06E-06	3.05E-06	2.69E-06	1.05E-08
11	SW	1.5	2.88E-06	2.86E-06	2.44E-06	5.00E-09
12	WSW	1.3	1.24E-05	1.23E-05	1.06E-05	1.57E-08
13	W	1.2	6.95E-06	6.88E-06	5.97E-06	8.47E-09
14	WNW	0.8	7.49E-06	7.43E-06	6.62E-06	1.16E-08
15	NW	0.8	7.21E-06	7.16E-06	6.37E-06	1.52E-08
16	NNW	0.6	6.90E-06	6.87E-06	6.22E-06	1.75E-08

NEAREST GARDEN WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
1	N	3.2	5.95E-07	5.84E-07	4.66E-07	1.26E-09
2	NNE	2.3	9.17E-07	9.06E-07	7.46E-07	2.75E-09
3	NE	2.7	6.01E-07	5.94E-07	4.81E-07	3.15E-09
4	ENE	2.1	3.52E-07	3.50E-07	2.89E-07	2.75E-09
5	E	1.8	1.82E-07	1.81E-07	1.51E-07	1.23E-09
6	ESE	2.5	7.86E-08	7.80E-08	6.36E-08	4.66E-10
7	SE	0.6	1.74E-06	1.74E-06	1.57E-06	1.34E-08
8	SSE	1.5	5.27E-07	5.25E-07	4.47E-07	3.17E-09
9	S	1.1	1.30E-06	1.29E-06	1.12E-06	6.63E-09
10	SSW	1.2	1.98E-06	1.96E-06	1.70E-06	6.34E-09
11	SW	1.9	2.03E-06	2.00E-06	1.68E-06	3.41E-09
12	WSW	1.3	1.24E-05	1.23E-05	1.06E-05	1.57E-08
13	W	1.2	6.95E-06	6.88E-06	5.97E-06	8.47E-09
14	NWW	(1)	-	-	-	-
15	NW	1.8	2.03E-06	2.01E-06	1.69E-06	3.63E-09
16	NNW	4	4.02E-07	3.91E-07	3.05E-07	6.48E-10

(1) No garden within 5 miles for this sector

***Locations use the 2003 Land Use Census Locations**

TABLE 2.3-122

2002 ATMOSPHERIC DISPERSION ESTIMATES
FOR NEAREST RESIDENCE AND GARDEN*

NEAREST RESIDENCE WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
1	N	1.3	2.06E-06	2.04E-06	1.76E-06	5.79E-09
2	NNE	1	3.26E-06	3.25E-06	2.84E-06	1.29E-08
3	NE	0.9	2.78E-06	2.77E-06	2.44E-06	1.99E-08
4	ENE	2.1	4.18E-07	4.15E-07	3.43E-07	3.05E-09
5	E	1.4	3.55E-07	3.53E-07	3.02E-07	2.30E-09
6	ESE	0.5	1.30E-06	1.30E-06	1.19E-06	1.07E-08
7	SE	0.5	1.67E-06	1.67E-06	1.53E-06	1.46E-08
8	SSE	0.6	1.96E-06	1.95E-06	1.76E-06	1.60E-08
9	S	1	1.45E-06	1.44E-06	1.26E-06	7.63E-09
10	SSW	0.9	3.73E-06	3.71E-06	3.27E-06	1.28E-08
11	SW	1.5	3.03E-06	3.00E-06	2.56E-06	5.50E-09
12	WSW	1.3	1.14E-05	1.13E-05	9.74E-06	1.33E-08
13	W	1.2	5.41E-06	5.36E-06	4.65E-06	6.70E-09
14	WNW	0.8	7.00E-06	6.95E-06	6.19E-06	1.12E-08
15	NW	0.8	5.92E-06	5.88E-06	5.23E-06	1.28E-08
16	NNW	0.6	6.47E-06	6.45E-06	5.84E-06	1.74E-08

NEAREST GARDEN WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
1	N	3.2	5.51E-07	5.42E-07	4.31E-07	1.30E-09
2	NNE	2.3	9.70E-07	9.61E-07	7.90E-07	3.39E-09
3	NE	2.7	5.50E-07	5.45E-07	4.41E-07	3.44E-09
4	ENE	2.1	4.18E-07	4.15E-07	3.43E-07	3.05E-09
5	E	1.8	2.38E-07	2.37E-07	1.99E-07	1.52E-09
6	ESE	2.5	8.95E-08	8.89E-08	7.24E-08	5.74E-10
7	SE	0.6	1.28E-06	1.28E-06	1.15E-06	1.07E-08
8	SSE	1.5	4.46E-07	4.44E-07	3.78E-07	3.02E-09
9	S	1.1	1.25E-06	1.24E-06	1.08E-06	6.42E-09
10	SSW	1.2	2.40E-06	2.39E-06	2.07E-06	7.71E-09
11	SW	1.9	2.13E-06	2.11E-06	1.76E-06	3.76E-09
12	WSW	1.3	1.14E-05	1.13E-05	9.74E-06	1.33E-08
13	W	1.2	5.41E-06	5.36E-06	4.65E-06	6.70E-09
14	NWW	(1)	-	-	-	-
15	NW	1.8	1.65E-06	1.63E-06	1.38E-06	3.05E-09
16	NNW	4	3.82E-07	3.73E-07	2.90E-07	6.46E-10

(1) No garden within 5 miles for this sector

*****Locations use the 2003 Land Use Census Locations

TABLE 2.3-123

2003 ATMOSPHERIC DISPERSION ESTIMATES
FOR NEAREST RESIDENCE AND GARDEN*

NEAREST RESIDENCE WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
1	N	1.3	1.89E-06	1.87E-06	1.61E-06	5.20E-09
2	NNE	1	2.85E-06	2.83E-06	2.48E-06	1.06E-08
3	NE	0.9	2.56E-06	2.55E-06	2.25E-06	1.74E-08
4	ENE	2.1	4.35E-07	4.32E-07	3.57E-07	3.02E-09
5	E	1.4	3.53E-07	3.51E-07	3.01E-07	2.19E-09
6	ESE	0.5	1.33E-06	1.33E-06	1.21E-06	1.12E-08
7	SE	0.5	1.66E-06	1.66E-06	1.52E-06	1.42E-08
8	SSE	0.6	1.68E-06	1.67E-06	1.51E-06	1.40E-08
9	S	1	1.20E-06	1.20E-06	1.05E-06	5.99E-09
10	SSW	0.9	3.92E-06	3.90E-06	3.44E-06	1.37E-08
11	SW	1.5	3.91E-06	3.88E-06	3.31E-06	7.30E-09
12	WSW	1.3	1.00E-05	9.93E-06	8.56E-06	1.30E-08
13	W	1.2	4.96E-06	4.91E-06	4.26E-06	6.68E-09
14	WNW	0.8	6.33E-06	6.29E-06	5.59E-06	1.24E-08
15	NW	0.8	7.72E-06	7.67E-06	6.82E-06	1.86E-08
16	NNW	0.6	5.96E-06	5.94E-06	5.37E-06	1.78E-08

NEAREST GARDEN WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
1	N	3.2	4.97E-07	4.89E-07	3.89E-07	1.17E-09
2	NNE	2.3	8.48E-07	8.39E-07	6.90E-07	2.79E-09
3	NE	2.7	5.08E-07	5.04E-07	4.07E-07	3.01E-09
4	ENE	2.1	4.35E-07	4.32E-07	3.57E-07	3.02E-09
5	E	1.8	2.38E-07	2.37E-07	1.99E-07	1.45E-09
6	ESE	2.5	8.98E-08	8.92E-08	7.27E-08	6.03E-10
7	SE	0.6	1.27E-06	1.27E-06	1.15E-06	1.04E-08
8	SSE	1.5	3.88E-07	3.86E-07	3.29E-07	2.65E-09
9	S	1.1	1.03E-06	1.03E-06	8.95E-07	5.04E-09
10	SSW	1.2	2.53E-06	2.52E-06	2.18E-06	8.24E-09
11	SW	1.9	2.75E-06	2.72E-06	2.28E-06	4.98E-09
12	WSW	1.3	1.00E-05	9.93E-06	8.56E-06	1.30E-08
13	W	1.2	4.96E-06	4.91E-06	4.26E-06	6.68E-09
14	NWW	-	-	-	-	-
15	NW	1.8	2.18E-06	2.16E-06	1.82E-06	4.44E-09
16	NNW	4	3.42E-07	3.34E-07	2.60E-07	6.60E-10

(1) No garden within 5 miles for this sector

*Locations use the 2003 Land Use Census Locations

TABLE 2.3-124

1999 - 2003 ATMOSPHERIC DISPERSION ESTIMATES
FOR NEAREST RESIDENCE AND GARDEN*

NEAREST RESIDENCE WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
1	N	1.3	2.00E-06	1.99E-06	1.71E-06	5.27E-09
2	NNE	1	3.05E-06	3.03E-06	2.65E-06	1.10E-08
3	NE	0.9	2.86E-06	2.85E-06	2.51E-06	1.81E-08
4	ENE	2.1	3.95E-07	3.92E-07	3.24E-07	2.92E-09
5	E	1.4	3.30E-07	3.28E-07	2.81E-07	2.16E-09
6	ESE	0.5	1.30E-06	1.30E-06	1.19E-06	1.04E-08
7	SE	0.5	1.92E-06	1.92E-06	1.75E-06	1.58E-08
8	SSE	0.6	2.05E-06	2.05E-06	1.85E-06	1.64E-08
9	S	1	1.48E-06	1.48E-06	1.29E-06	7.84E-09
10	SSW	0.9	3.53E-06	3.51E-06	3.09E-06	1.25E-08
11	SW	1.5	3.24E-06	3.21E-06	2.74E-06	6.15E-09
12	WSW	1.3	1.14E-05	1.13E-05	9.70E-06	1.43E-08
13	W	1.2	5.84E-06	5.79E-06	5.02E-06	7.43E-09
14	WNW	0.8	6.73E-06	6.68E-06	5.94E-06	1.13E-08
15	NW	0.8	6.77E-06	6.72E-06	5.98E-06	1.46E-08
16	NNW	0.6	6.04E-06	6.01E-06	5.44E-06	1.60E-08

NEAREST GARDEN WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
1	N	3.2	5.34E-07	5.24E-07	4.17E-07	1.19E-09
2	NNE	2.3	9.07E-07	8.98E-07	7.39E-07	2.88E-09
3	NE	2.7	5.72E-07	5.67E-07	4.59E-07	3.13E-09
4	ENE	2.1	3.95E-07	3.92E-07	3.24E-07	2.92E-09
5	E	1.8	2.22E-07	2.21E-07	1.85E-07	1.43E-09
6	ESE	2.5	8.76E-08	8.70E-08	7.09E-08	5.60E-10
7	SE	0.6	1.47E-06	1.47E-06	1.32E-06	1.16E-08
8	SSE	1.5	4.70E-07	4.68E-07	3.98E-07	3.10E-09
9	S	1.1	1.27E-06	1.27E-06	1.10E-06	6.60E-09
10	SSW	1.2	2.27E-06	2.26E-06	1.95E-06	7.55E-09
11	SW	1.9	2.28E-06	2.25E-06	1.89E-06	4.20E-09
12	WSW	1.3	1.14E-05	1.13E-05	9.70E-06	1.43E-08
13	W	1.2	5.84E-06	5.79E-06	5.02E-06	7.43E-09
14	NWW	(1)	-	-	-	-
15	NW	1.8	1.90E-06	1.88E-06	1.58E-06	3.49E-09
16	NNW	4	3.50E-07	3.41E-07	2.65E-07	5.94E-10

(1) No garden within 5 miles for this sector

*****Locations use the 2003 Land Use Census Locations

TABLE 2.3-125

1999 ATMOSPHERIC DISPERSION ESTIMATES
FOR NEAREST MEAT ANIMAL. DAIRY LOCATIONS
AND SPECIAL RECEPTORS*

ANIMAL RAISED FOR MEAT CONSUMPTION
WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
2	NNE	2.3	8.50E-07	8.40E-07	6.92E-07	2.73E-09
4	ENE	2.4	3.37E-07	3.35E-07	2.74E-07	2.55E-09
5	E	1.4	3.28E-07	3.27E-07	2.80E-07	2.30E-09
10	SSW	3	4.70E-07	4.63E-07	3.71E-07	1.58E-09
10	SSW	3.5	3.30E-07	3.25E-07	2.56E-07	1.05E-09
12	WSW	1.7	7.90E-06	7.82E-06	6.61E-06	9.71E-09
15	NW	1.8	1.71E-06	1.69E-06	1.43E-06	3.19E-09

ALL DAIRY LOCATIONS WITHIN A 5-MILE RADIUS OF SSES

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
5	E	4.5	4.62E-08	4.56E-08	3.46E-08	2.66E-10
6	ESE	2.7	6.54E-08	6.50E-08	5.24E-08	4.19E-10
6	ESE	4.2	2.64E-08	2.61E-08	2.00E-08	1.49E-10
10	SSW	3	4.70E-07	4.63E-07	3.71E-07	1.58E-09
10	SSW	3.1	4.37E-07	4.31E-07	3.43E-07	1.46E-09
10	SSW	3.5	3.30E-07	3.25E-07	2.56E-07	1.05E-09
12	WSW	1.7	7.90E-06	7.82E-06	6.61E-06	9.71E-09
13	W	5	5.49E-07	5.28E-07	4.03E-07	4.36E-10
16	NNW	4.2	2.69E-07	2.62E-07	2.03E-07	4.68E-10

SPECIAL RECEPTOR LOCATIONS

SECTOR NUMBER	AFFECTED SECTOR	LOCATION	MILES	X/Q(1)	X/Q DEC(2)	X/Q DEC+DEP(3)	DEPOSITION(4)
3	NE	Riverlands / EIC	0.7	4.40E-06	4.39E-06	3.93E-06	2.70E-08
12	WSW	Tower's Club	0.5	4.06E-05	4.05E-05	3.71E-05	5.97E-08
5	E	East Gate	0.5	1.70E-06	1.70E-06	1.55E-06	1.48E-08

***Locations use the 2003 Land Use Census Locations**

(1) Relative concentration (sec/m^3)
(2) Decayed and undepleted, half-life 2.26 days (sec/m^3)
(3) Decayed and depleted, half-life 8 days (sec/m^3)
(4) Relative deposition rate (l/m^2)

TABLE 2.3-126

2000 ATMOSPHERIC DISPERSION ESTIMATES
FOR NEAREST MEAT ANIMAL, DAIRY LOCATIIONS
AND SPECIAL RECEPTORS*

ANIMAL RAISED FOR MEAT CONSUMPTION
WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
2	NNE	2.3	9.52E-07	9.41E-07	7.75E-07	2.74E-09
4	ENE	2.4	3.04E-07	3.02E-07	2.47E-07	2.26E-09
5	E	1.4	3.42E-07	3.40E-07	2.91E-07	2.17E-09
10	SSW	3	5.36E-07	5.28E-07	4.23E-07	1.43E-09
10	SSW	3.5	3.77E-07	3.70E-07	2.92E-07	9.52E-10
12	WSW	1.7	7.71E-06	7.63E-06	6.45E-06	9.69E-09
15	NW	1.8	1.93E-06	1.91E-06	1.61E-06	3.15E-09

ALL DAIRY LOCATIONS WITHIN A 5-MILE RAIDUS OF SSES

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
5	E	4.5	4.78E-08	4.70E-08	3.58E-08	2.51E-10
6	ESE	2.7	8.98E-08	8.90E-08	7.20E-08	5.58E-10
6	ESE	4.2	3.60E-08	3.55E-08	2.72E-08	1.99E-10
10	SSW	3	5.36E-07	5.28E-07	4.23E-07	1.43E-09
10	SSW	3.1	4.99E-07	4.91E-07	3.92E-07	1.32E-09
10	SSW	3.5	3.77E-07	3.70E-07	2.92E-07	9.52E-10
12	WSW	1.7	7.71E-06	7.63E-06	6.45E-06	9.69E-09
13	W	5	5.35E-07	5.15E-07	3.93E-07	4.49E-10
16	NNW	4.2	3.15E-07	3.06E-07	2.37E-07	4.68E-10

SPECIAL RECEPTOR LOCATIONS

SECTOR NUMBER	AFFECTED SECTOR	LOCATION	MILES	X/Q(1)	X/Q DEC(2)	X/Q DEC+DEP(3)	DEPOSITION(4)
3	NE	Riverlands/EIC	0.7	4.17E-06	4.16E-06	3.73E-06	2.61E-08
12	WSW	Towers Club	0.5	3.88E-05	3.87E-05	3.54E-05	5.96E-08
5	E	East Gate	0.5	1.79E-06	1.79E-06	1.63E-06	1.39E-08

*Locations use the 2003 Land Use Census Locations. Only sectors with animals or dairy within 5 miles are shown.

(1) Relative concentration (sec/m^3)
(2) Decayed and undepleted, half-life 2.26 days (sec/m^3)
(3) Decayed and depleted, half-life 8 days (sec/m^3)
(4) Relative deposition rate (1/m^2)

TABLE 2.3-127

2001 ATMOSPHERIC DISPERSION ESTIMATES
FOR NEAREST MEAT ANIMAL, DAIRY LOCATIIONS
AND SPECIAL RECEPTORS*

ANIMAL RAISED FOR MEAT CONSUMPTION WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
2	NNE	2.3	9.17E-07	9.06E-07	7.46E-07	2.75E-09
4	ENE	2.4	2.93E-07	2.91E-07	2.38E-07	2.30E-09
5	E	1.4	2.71E-07	2.70E-07	2.31E-07	1.85E-09
10	SSW	3	4.46E-07	4.39E-07	3.52E-07	1.24E-09
10	SSW	3.5	3.13E-07	3.08E-07	2.42E-07	8.22E-10
12	WSW	1.7	8.49E-06	8.40E-06	7.10E-06	1.02E-08
15	NW	1.8	2.03E-06	2.01E-06	1.69E-06	3.63E-09

ALL DAIRY LOCATIONS WITHIN A 5-MILE RADIUS OF SSES

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
5	E	4.5	3.70E-08	3.64E-08	2.77E-08	2.14E-10
6	ESE	2.7	6.77E-08	6.71E-08	5.42E-08	3.93E-10
6	ESE	4.2	2.71E-08	2.67E-08	2.05E-08	1.40E-10
10	SSW	3	4.46E-07	4.39E-07	3.52E-07	1.24E-09
10	SSW	3.1	4.15E-07	4.08E-07	3.26E-07	1.14E-09
10	SSW	3.5	3.13E-07	3.08E-07	2.42E-07	8.22E-10
12	WSW	1.7	8.49E-06	8.40E-06	7.10E-06	1.02E-08
13	W	5	6.35E-07	6.11E-07	4.66E-07	4.89E-10
16	NNW	4.2	3.76E-07	3.66E-07	2.84E-07	5.95E-10

SPECIAL RECEPTOR LOCATIONS

SECTOR NUMBER	AFFECTED SECTOR	LOCATION	MILES	X/Q(1)	X/Q DEC(2)	X/Q DEC+DEP(3)	DEPOSITION(4)
3	NE	Riverlands/EIC	0.7	4.34E-06	4.33E-06	3.87E-06	2.77E-08
12	WSW	Towers Club	0.5	4.29E-05	4.28E-05	3.92E-05	6.28E-08
5	E	East Gate	0.5	1.42E-06	1.42E-06	1.30E-06	1.19E-08

*Locations use the 2003 Land Use Census Locations. Only sectors with animals or dairy within 5 miles are shown

(1) Relative concentration (sec/m^3)
(2) Decayed and undepleted, half-life 2.26 days (sec/m^3)
(3) Decayed and depleted, half-life 8 days (sec/m^3)
(4) Relative deposition rate (1/m^2)

TABLE 2.3-128

2002 ATMOSPHERIC DISPERSION ESTIMATES
FOR NEAREST MEAT ANIMAL, DAIRY LOCATIIONS
AND SPECIAL RECEPTORS*

ANIMAL RAISED FOR MEAT CONSUMPTION WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
2	NNE	2.3	9.70E-07	9.61E-07	7.90E-07	3.39E-09
4	ENE	2.4	3.48E-07	3.45E-07	2.82E-07	2.54E-09
5	E	1.4	3.55E-07	3.53E-07	3.02E-07	2.30E-09
10	SSW	3	5.49E-07	5.40E-07	4.33E-07	1.50E-09
10	SSW	3.5	3.87E-07	3.80E-07	2.99E-07	1.00E-09
12	WSW	1.7	7.78E-06	7.70E-06	6.51E-06	8.69E-08
15	NW	1.8	1.65E-06	1.63E-06	1.38E-06	3.05E-09

ALL DAIRY LOCATIONS WITHIN A 5-MILE RADIUS OF SSES

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
5	E	4.5	4.86E-08	4.79E-08	3.64E-08	2.66E-10
6	ESE	2.7	7.72E-08	7.66E-08	6.19E-08	4.85E-10
6	ESE	4.2	3.11E-08	3.07E-08	2.36E-08	1.72E-10
10	SSW	3	5.49E-07	5.40E-07	4.33E-07	1.50E-09
10	SSW	3.1	5.11E-07	5.03E-07	4.01E-07	1.38E-09
10	SSW	3.5	3.87E-07	3.80E-07	2.99E-07	1.00E-09
12	WSW	1.7	7.78E-06	7.70E-06	6.51E-06	8.69E-09
13	W	5	4.98E-07	4.78E-07	3.65E-07	3.87E-10
16	NNW	4.2	3.58E-07	3.49E-07	2.70E-07	5.93E-10

SPECIAL RECEPTOR LOCATIONS

SECTOR NUMBER	AFFECTED SECTOR	LOCATION	MILES	X/Q(1)	X/Q DEC(2)	X/Q DEC+DEP(3)	DEPOSITION(4)
3	NE	Riverlands / EIC	0.7	4.03E-06	4.02E-06	3.60E-06	3.03E-08
12	WSW	Tower's Club	0.5	3.97E-05	3.96E-05	3.63E-05	5.34E-08
5	E	East Gate	0.5	1.82E-06	1.82E-06	1.67E-06	1.48E-08

*Locations use the 2003 Land Use Census Locations. Only sectors with animals or dairy within 5 miles are shown.

(1) Relative concentration (sec/m^3)
(2) Decayed and undepleted, half-life 2.26 days (sec/m^3)
(3) Decayed and depleted, half-life 8 days (sec/m^3)
(4) Relative deposition rate (1/m^2)

TABLE 2.3-129

2003 ATMOSPHERIC DISPERSION ESTIMATES
FOR NEAREST MEAT ANIMAL, DAIRY LOCATIIONS
AND SPECIAL RECEPTORS*

ANIMAL RAISED FOR MEAT CONSUMPTION WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
2	NNE	2.3	8.48E-07	8.39E-07	6.90E-07	2.79E-09
4	ENE	2.4	3.62E-07	3.60E-07	2.94E-07	2.52E-09
5	E	1.4	3.53E-07	3.51E-07	3.01E-07	2.19E-09
10	SSW	3	5.71E-07	5.63E-07	4.51E-07	1.61E-09
10	SSW	3.5	4.00E-07	3.94E-07	3.10E-07	1.07E-09
12	WSW	1.7	6.85E-06	6.77E-06	5.73E-06	8.50E-09
15	NW	1.8	2.18E-06	2.16E-06	1.82E-06	4.44E-09

ALL DAIRY LOCATIONS NEAR SSES

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
5	E	4.5	4.99E-08	4.90E-08	3.73E-08	2.53E-10
6	ESE	2.7	7.74E-08	7.67E-08	6.20E-08	5.09E-10
6	ESE	4.2	3.10E-08	3.06E-08	2.35E-08	1.81E-10
10	SSW	3	5.71E-07	5.63E-07	4.51E-07	1.61E-09
10	SSW	3.1	5.30E-07	5.23E-07	4.17E-07	1.48E-09
10	SSW	3.5	4.00E-07	3.94E-07	3.10E-07	1.07E-09
12	WSW	1.7	6.85E-06	6.77E-06	5.73E-06	8.50E-09
13	W	5	4.42E-07	4.25E-07	3.25E-07	3.86E-10
16	NNW	4.2	3.20E-07	3.13E-07	2.42E-07	6.07E-10

SPECIAL RECEPTOR LOCATIONS

SECTOR NUMBER	AFFECTED SECTOR	LOCATION	MILES	X/Q(1)	X/Q DEC(2)	X/Q DEC+DEP(3)	DEPOSITION(4)
3	NE	Riverlands / EIC	0.7	3.70E-06	3.69E-06	3.30E-06	2.65E-08
12	WSW	Tower's Club	0.5	3.41E-05	3.40E-05	3.11E-05	5.22E-08
5	E	East Gate	0.5	1.82E-06	1.81E-06	1.66E-06	1.41E-08

*Locations use the 2003 Land Use Census Locations

(1) Relative concentration (sec/m^3)
(2) Decayed and undepleted, half-life 2.26 days (sec/m^3)
(3) Decayed and depleted, half-life 8 days (sec/m^3)
(4) Relative deposition rate (1/m^2)

TABLE 2.3-130

1999 - 2003 ATMOSPHERIC DISPERSION ESTIMATES
FOR NEAREST MEAT ANIMAL, DAIRY LOCATIIONS
AND SPECIAL RECEPTORS*

ANIMAL RAISED FOR MEAT CONSUMPTION WITHIN A 5-MILE RADIUS OF SSES BY SECTOR

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
2	NNE	2.3	9.07E-07	8.98E-07	7.39E-07	2.88E-09
4	ENE	2.4	3.29E-07	3.26E-07	2.67E-07	2.43E-09
5	E	1.4	3.30E-07	3.28E-07	2.81E-07	2.16E-09
10	SSW	3	5.14E-07	5.07E-07	4.06E-07	1.47E-09
10	SSW	3.5	3.61E-07	3.55E-07	2.80E-07	9.79E-10
12	WSW	1.7	7.75E-06	7.66E-06	6.48E-06	9.36E-09
15	NW	1.8	1.90E-06	1.88E-06	1.58E-06	3.49E-09

ALL DAIRY LOCATIONS NEAR SSES

SECTOR NUMBER	AFFECTED SECTOR	MILES	X/Q	X/Q DEC	X/Q DEC+DEP	DEPOSITION
5	E	4.5	4.59E-08	4.52E-08	3.44E-08	2.50E-10
6	ESE	2.7	7.55E-08	7.49E-08	6.05E-08	4.73E-10
6	ESE	4.2	3.03E-08	2.99E-08	2.29E-08	1.68E-10
10	SSW	3	5.14E-07	5.07E-07	4.06E-07	1.47E-09
10	SSW	3.1	4.78E-07	4.71E-07	3.76E-07	1.35E-09
10	SSW	3.5	3.61E-07	3.55E-07	2.80E-07	9.79E-10
12	WSW	1.7	7.75E-06	7.66E-06	6.48E-06	9.36E-09
13	W	5	5.32E-07	5.12E-07	3.90E-07	4.29E-10
16	NNW	4.2	3.28E-07	3.19E-07	2.47E-07	5.46E-10

SPECIAL RECEPTOR LOCATIONS

SECTOR NUMBER	AFFECTED SECTOR	LOCATION	MILES	X/Q(1)	X/Q DEC(2)	X/Q DEC+DEP(3)	DEPOSITION(4)
3	NE	Riverlands / EIC	0.7	4.13E-06	4.12E-06	3.69E-06	2.75E-08
12	WSW	Tower's Club	0.5	3.92E-05	3.91E-05	3.58E-05	5.75E-08
5	E	East Gate	0.5	1.71E-06	1.71E-06	1.56E-06	1.39E-08

*Locations use the 2003 Land Use Census Locations. Only sectors with animals or dairy within 5 miles are shown.

(1) Relative concentration (sec/m^3)
(2) Decayed and undepleted, half-life 2.26 days (sec/m^3)
(3) Decayed and depleted, half-life 8 days (sec/m^3)
(4) Relative deposition rate (1/m^2)

TABLE 2.3-131

1999 ATMOSPHERIC DISPERSION ESTIMATES
AT SELECTED LOCATIONS

SECTOR NUMBER	AFFECTED SECTOR	LOCATION	MILES	X/Q(1)	X/Q DEC(2)	X/Q DEC+DEP(3)	DEPOSITION(4)
12	WSW	Maximum (X/Q) Site Boundary	1.22	1.28E-05	1.27E-05	1.10E-05	1.66E-08
9	S	Closest (X/Q) Site Boundary	0.38	6.71E-06	6.70E-06	6.25E-06	4.66E-08
12	WSW	Maximum (X/Q) Residence	1.3	1.16E-05	1.15E-05	9.90E-06	1.49E-08
3	NE	Maximum (D/Q) Residence	0.9	3.07E-06	3.05E-06	2.69E-06	1.77E-08
12	WSW	Maximum (D/Q) Garden	1.3	1.15E-05	9.90E-06	1.49E-08	1.16E-05
12	WSW	Maximum (D/Q) Dairy	1.7	7.90E-06	7.82E-06	6.61E-06	9.71E-09
12	WSW	Maximum (D/Q) Meat Producer	1.7	7.90E-06	7.82E-06	6.61E-06	9.71E-09
3	NE	Riverlands / EIC	0.7	4.40E-06	4.39E-06	3.93E-06	2.70E-08
12	WSW	Tower's Club	0.5	4.06E-05	4.05E-05	3.71E-05	5.97E-08
5	E	East Gate	0.5	1.70E-06	1.70E-06	1.55E-06	1.48E-08

(1) Relative concentration (sec/m^3)
(2) Decayed and undepleted, half-life 2.26 days (sec/m^3)
(3) Decayed and depleted, half-life 8 days (sec/m^3)
(4) Relative deposition rate (1/m^2)

TABLE 2.3-132

2000 ATMOSPHERIC DISPERSION ESTIMATES
AT SELECTED LOCATIONS

SECTOR NUMBER	AFFECTED SECTOR	LOCATION	MILES	X/Q(1)	X/Q DEC(2)	X/Q DEC+DEP(3)	DEPOSITION(4)
12	WSW	Maximum (X/Q) Site Boundary	1.22	1.24E-05	1.23E-05	1.07E-05	1.65E-08
9	S	Closest (X/Q) Site Boundary	0.38	6.21E-06	6.20E-06	5.78E-06	4.11E-08
12	WSW	Maximum (X/Q) Residence	1.3	1.13E-05	1.12E-05	9.65E-06	1.49E-08
3	NE	Maximum (D/Q) Residence	0.6	2.44E-06	2.44E-06	2.20E-06	1.94E-08
12	WSW	Maximum (D/Q) Garden	1.3	1.13E-05	1.12E-05	9.65E-06	1.49E-08
12	WSW	Maximum (D/Q) Dairy	1.7	7.71E-06	7.63E-06	6.45E-06	9.69E-09
12	WSW	Maximum (D/Q) Meat Producer	1.7	7.71E-06	7.63E-06	6.45E-06	9.69E-09
3	NE	Riverlands / EIC	0.7	4.17E-06	4.16E-06	3.73E-06	2.61E-08
12	WSW	Tower's Club	0.5	3.88E-05	3.87E-05	3.54E-05	5.96E-08
5	E	East Gate	0.5	1.79E-06	1.79E-06	1.63E-06	1.39E-08

(1) Relative concentration (sec/m^3)
(2) Decayed and undepleted, half-life 2.26 days (sec/m^3)
(3) Decayed and depleted, half-life 8 days (sec/m^3)
(4) Relative deposition rate (1/m^2)

TABLE 2.3-133

2001 ATMOSPHERIC DISPERSION ESTIMATES
AT SELECTED LOCATIONS

AFFECTED SECTOR	LOCATION	MILES	X/Q(1)	X/Q DEC(2)	X/Q DEC+DEP(3)	DEPOSITION(4)
12/WSW	Maximum (X/Q) Site Boundary	1.22	1.37E-05	1.36E-05	1.18E-05	1.74E-08
9/S	Closest (X/Q) Site Boundary	0.38	6.07E-06	6.06E-06	5.65E-06	3.90E-08
12/WSW	Maximum (X/Q) Residence	1.3	1.24E-05	1.23E-05	1.06E-05	1.57E-08
3/NE	Maximum (D/Q) Residence	0.9	3.00E-06	2.99E-06	2.63E-06	1.82E-08
12/WSW	Maximum (D/Q) Garden	1.3	1.24E-05	1.23E-05	1.06E-05	1.57E-08
12/SW	Maximum (D/Q) Dairy	1.7	8.49E-06	8.40E-06	7.10E-06	1.02E-08
12/WSW	Maximum (D/Q) Meat Producer	1.7	8.49E-06	8.40E-06	7.10E-06	1.02E-08
3/NE	Riverlands / EIC	0.7	4.34E-06	4.33E-06	3.87E-06	2.77E-08
12/WSW	Tower's Club	0.5	4.29E-05	4.28E-05	3.92E-05	6.28E-08
5/E	East Gate	0.5	1.42E-06	1.42E-06	1.30E-06	1.19E-08

(1) Relative concentration (sec/m^3)
(2) Decayed and undepleted, half-life 2.26 days (sec/m^3)
(3) Decayed and depleted, half-life 8 days (sec/m^3)
(4) Relative deposition rate (1/m^2)

TABLE 2.3-134

2002 ATMOSPHERIC DISPERSION ESTIMATES
AT SELECTED LOCATIONS

AFFECTED SECTOR	LOCATION	MILES	X/Q(1)	X/Q DEC(2)	X/Q DEC+DEP(3)	DEPOSITION(4)
12/WSW	Maximum (X/Q) Site Boundary	1.22	1.26E-05	1.25E-05	1.08E-05	1.48E-08
9/S	Closest (X/Q) Site Boundary	0.38	5.71E-06	5.70E-06	5.31E-06	3.78E-08
12/WSW	Maximum (X/Q) Residence	1.3	1.14E-05	1.13E-05	9.74E-06	1.33E-08
3/NE	Maximum (D/Q) Residence	0.9	2.78E-06	2.77E-06	2.44E-06	1.99E-08
12/WSW	Maximum (D/Q) Garden	1.3	1.14E-05	1.13E-05	9.74E-06	1.33E-08
12/SW	Maximum (D/Q) Dairy	1.7	7.78E-06	7.70E-06	6.51E-06	8.69E-09
12/WSW	Maximum (D/Q) Meat Producer	1.7	7.78E-06	7.70E-06	6.51E-06	8.69E-09
3/NE	Riverlands / EIC	0.7	4.03E-06	4.02E-06	3.60E-06	3.03E-08
12/WSW	Tower's Club	0.5	3.97E-05	3.96E-05	3.63E-05	5.34E-08
5/E	East Gate	0.5	1.82E-06	1.82E-06	1.67E-06	1.48E-08

(1) Relative concentration (sec/m^3)
(2) Decayed and undepleted, half-life 2.26 days (sec/m^3)
(3) Decayed and depleted, half-life 8 days (sec/m^3)
(4) Relative deposition rate (1/m^2)

TABLE 2.3-135

2003 ATMOSPHERIC DISPERSION ESTIMATES
AT SELECTED LOCATIONS

AFFECTED SECTOR	LOCATION	MILES	X/Q(1)	X/Q DEC(2)	X/Q DEC+DEP(3)	DEPOSITION(4)
11WS	Maximum (X/Q) Site Boundary	0.61	1.45E-05	1.45E-05	1.31E-05	3.25E-08
9/S	Closest (X/Q) Site Boundary	0.38	4.69E-06	4.69E-06	4.37E-06	2.97E-08
12/WSW	Maximum (X/Q) Residence	1.3	1.00E-05	9.93E-06	8.56E-06	1.30E-08
15/NW	Maximum (D/Q) Residence	0.8	7.72E-06	7.67E-06	6.82E-06	1.86E-08
12/WSW	Maximum (D/Q) Garden	1.3	1.00E-05	9.93E-06	8.56E-06	1.30E-08
12/SW	Maximum (D/Q) Dairy	1.7	6.85E-06	6.77E-06	5.73E-06	8.50E-09
12/WSW	Maximum (D/Q) Meat Producer	1.7	6.85E-06	6.77E-06	5.73E-06	8.50E-09
3/NE	Riverlands / EIC	0.7	3.70E-06	3.69E-06	3.30E-06	2.65E-08
12/WSW	Tower’s Club	0.5	3.41E-05	3.40E-05	3.11E-05	5.22E-08
5/E	East Gate	0.5	1.82E-06	1.81E-06	1.66E-06	1.41E-08

(1) Relative concentration (sec/m^3)
(2) Decayed and undepleted, half-life 2.26 days (sec/m^3)
(3) Decayed and depleted, half-life 8 days (sec/m^3)
(4) Relative deposition rate (1/m^2)

TABLE 2.3-136

1999 - 2003 ATMOSPHERIC DISPERSION ESTIMATES
AT SELECTED LOCATIONS

AFFECTED SECTOR	LOCATION	MILES	X/Q(1)	X/Q DEC(2)	X/Q DEC+DEP(3)	DEPOSITION(4)
12/WSW	Maximum (X/Q) Site Boundary	1.22	1.25E-05	1.24E-05	1.07E-05	1.60E-08
9/S	Closest (X/Q) Site Boundary	0.38	5.88E-06	5.87E-06	5.47E-06	3.88E-08
12/WSW	Maximum (X/Q) Residence	1.3	1.14E-05	1.13E-05	9.70E-06	1.43E-08
3/NE	Maximum (D/Q) Residence	0.9	2.86E-06	2.85E-06	2.51E-06	1.81E-08
12/WSW	Maximum (D/Q) Garden	1.3	1.14E-05	1.13E-05	9.70E-06	1.43E-08
12/WSW	Maximum (D/Q) Dairy	1.7	7.75E-06	7.66E-06	6.48E-06	9.36E-09
12/WSW	Maximum (D/Q) Meat Producer	1.7	7.75E-06	7.66E-06	6.48E-06	9.36E-09
3/NE	Riverlands / EIC	0.7	4.13E-06	4.12E-06	3.69E-06	2.75E-08
12/WSW	Tower's Club	0.5	3.92E-05	3.91E-05	3.58E-05	5.75E-08
5/E	East Gate	0.5	1.71E-06	1.71E-06	1.56E-06	1.39E-08

(1) Relative concentration (sec/m^3)
(2) Decayed and undepleted, half-life 2.26 days (sec/m^3)
(3) Decayed and depleted, half-life 8 days (sec/m^3)
(4) Relative deposition rate (1/m^2)

TABLE 2.3-137

1999-2003 SSES RELATIVE CONCENTRATIONS NO DECAY, UNDEPLETED X/Q
X/Q ACCUMULATION FOR GROUND AVERAGE (seconds per cubic meter)

Direction From	0.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
N	4.13E-06	7.83E-07	3.25E-07	1.71E-07	1.10E-07	4.05E-08	1.11E-08	5.34E-09	3.35E-09	2.37E-09
NNE	8.11E-06	1.63E-06	7.28E-07	3.91E-07	2.53E-07	9.40E-08	2.59E-08	1.27E-08	8.10E-09	5.80E-09
NE	1.71E-05	3.22E-06	1.47E-06	8.32E-07	5.51E-07	2.18E-07	6.61E-08	3.32E-08	2.14E-08	1.55E-08
ENE	4.96E-05	9.15E-06	4.48E-06	2.66E-06	1.78E-06	7.05E-07	2.06E-07	9.99E-08	6.47E-08	4.76E-08
E	2.24E-05	4.08E-06	1.80E-06	1.02E-06	6.83E-07	2.78E-07	8.81E-08	4.46E-08	2.88E-08	2.10E-08
ESE	1.27E-05	2.45E-06	1.11E-06	6.23E-07	4.13E-07	1.67E-07	4.64E-08	2.04E-08	1.31E-08	9.49E-09
SE	1.26E-05	2.48E-06	1.13E-06	6.41E-07	4.25E-07	1.74E-07	4.35E-08	1.61E-08	1.03E-08	7.42E-09
SSE	9.08E-06	1.77E-06	7.87E-07	4.42E-07	2.98E-07	1.28E-07	3.37E-08	1.21E-08	7.71E-09	5.55E-09
S	7.81E-06	1.65E-06	8.08E-07	4.70E-07	3.23E-07	1.50E-07	4.18E-08	1.44E-08	9.23E-09	6.63E-09
SSW	8.36E-06	1.69E-06	7.78E-07	4.42E-07	2.94E-07	1.22E-07	3.21E-08	1.23E-08	7.80E-09	5.59E-09
SW	6.65E-06	1.34E-06	6.36E-07	3.65E-07	2.45E-07	1.08E-07	2.80E-08	9.42E-09	5.96E-09	4.23E-09
WSW	3.41E-06	6.56E-07	3.05E-07	1.79E-07	1.23E-07	5.80E-08	1.82E-08	6.82E-09	3.49E-09	1.91E-09
W	1.58E-06	2.99E-07	1.30E-07	7.11E-08	4.67E-08	1.91E-08	5.18E-09	2.10E-09	1.31E-09	9.15E-10
WNW	1.20E-06	2.19E-07	8.80E-08	4.60E-08	2.93E-08	1.08E-08	2.93E-09	1.39E-09	8.58E-10	5.96E-10
NW	2.03E-06	3.78E-07	1.50E-07	7.66E-08	4.86E-08	1.75E-08	4.62E-09	2.18E-09	1.34E-09	9.32E-10
NNW	2.58E-06	4.83E-07	2.04E-07	1.08E-07	6.83E-08	2.38E-08	5.94E-09	2.82E-09	1.75E-09	1.22E-09

TABLE 2.3-138

1999-2003 SSES RELATIVE CONCENTRATIONS, 2.26-DAY DECAY UNDEPLETED X/Q
X/Q ACCUMULATION FOR GROUND DECAYED SECTOR AVERAGE (seconds per cubic meter)

Direction From	MILES									
	0.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
N	4.13E-06	7.79E-07	3.22E-07	1.69E-07	1.08E-07	3.94E-08	1.04E-08	4.84E-09	2.92E-09	1.98E-09
NNE	8.09E-06	1.62E-06	7.19E-07	3.84E-07	2.47E-07	9.06E-08	2.40E-08	1.12E-08	6.80E-09	4.63E-09
NE	1.71E-05	3.19E-06	1.45E-06	8.15E-07	5.37E-07	2.08E-07	6.03E-08	2.85E-08	1.73E-08	1.18E-08
ENE	4.95E-05	9.06E-06	4.41E-06	2.60E-06	1.73E-06	6.72E-07	1.87E-07	8.55E-08	5.21E-08	3.60E-08
E	2.23E-05	4.03E-06	1.77E-06	9.95E-07	6.60E-07	2.62E-07	7.85E-08	3.69E-08	2.21E-08	1.49E-08
ESE	1.26E-05	2.43E-06	1.09E-06	6.06E-07	3.99E-07	1.57E-07	4.12E-08	1.67E-08	9.93E-09	6.65E-09
SE	1.26E-06	2.45E-06	1.11E-06	6.25E-07	4.12E-07	1.64E-07	3.90E-08	1.34E-08	7.95E-09	5.33E-09
SSE	9.05E-06	1.75E-06	7.75E-07	4.33E-07	2.89E-07	1.22E-07	3.06E-08	1.03E-08	6.16E-09	4.16E-09
S	7.79E-06	1.64E-06	7.97E-07	4.61E-07	3.15E-07	1.44E-07	3.84E-08	1.26E-08	7.59E-09	5.16E-09
SSW	8.34E-06	1.68E-06	7.69E-07	4.35E-07	2.88E-07	1.18E-07	2.99E-08	1.09E-08	6.64E-09	4.54E-09
SW	6.64E-06	1.33E-06	6.31E-07	3.61E-07	2.41E-07	1.05E-07	2.65E-08	8.60E-09	5.25E-09	3.59E-09
WSW	3.41E-06	6.53E-07	3.03E-07	1.78E-07	1.21E-07	5.67E-08	1.73E-08	6.30E-09	3.13E-09	1.66E-09
W	1.58E-06	2.98E-07	1.29E-07	7.03E-08	4.60E-08	1.86E-08	4.91E-09	1.92E-09	1.16E-09	7.79E-10
WNW	1.20E-06	2.18E-07	8.73E-08	4.55E-08	2.89E-08	1.06E-08	2.80E-09	1.28E-09	7.67E-10	5.16E-10
NW	2.03E-06	3.76E-07	1.49E-07	7.58E-08	4.80E-08	1.72E-08	4.42E-09	2.03E-09	1.22E-09	8.19E-10
NNW	2.57E-06	4.81E-07	2.02E-07	1.07E-07	6.74E-08	2.33E-08	5.67E-09	2.61E-09	1.57E-09	1.06E-09

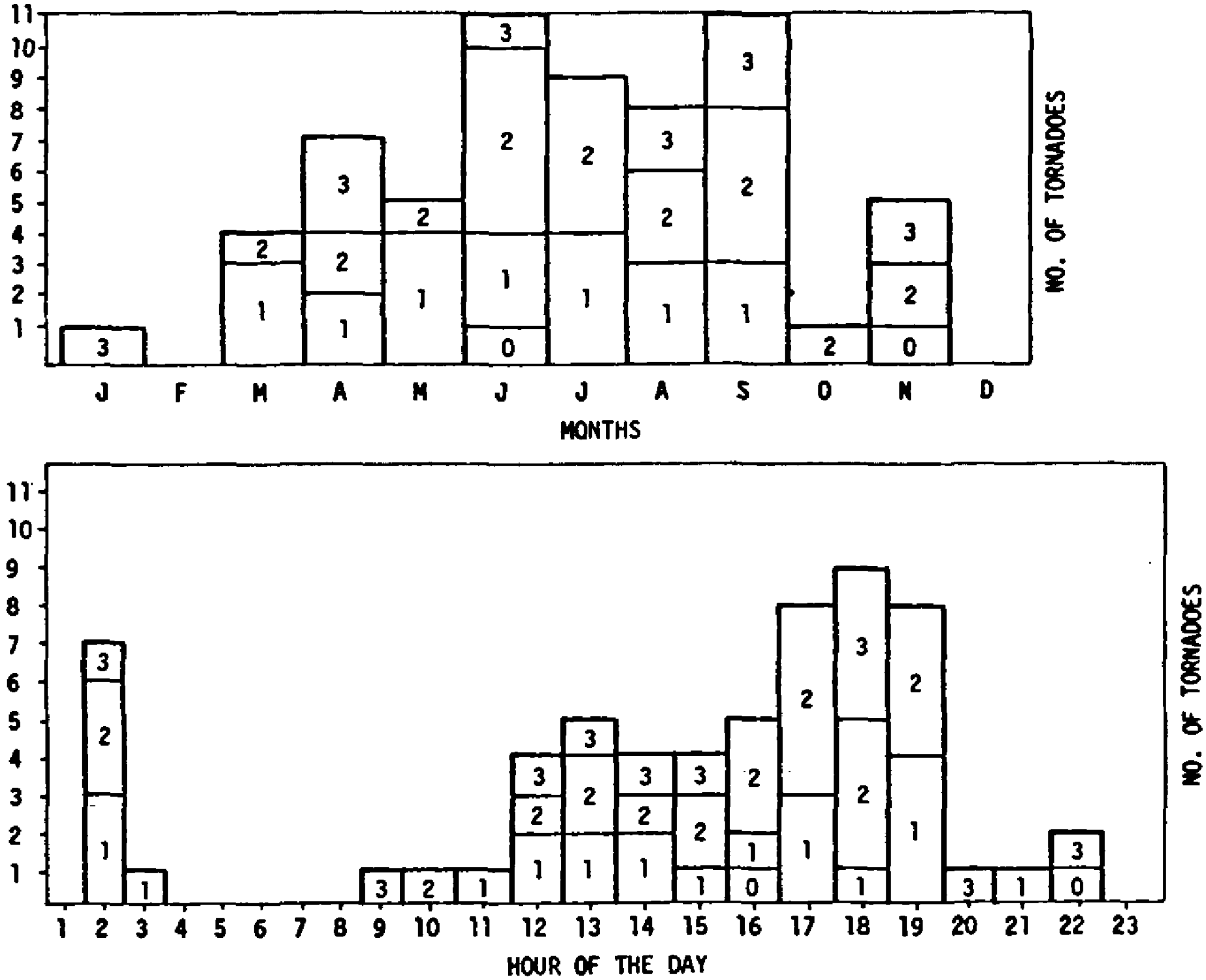
TABLE 2.3-139

1999-2003 SSES RELATIVE CONCENTRATIONS 8-DAY DECAY, DEPLETED X/Q
X/Q ACCUMULATION FOR DECAYED DEPLETION (seconds per cubic meter)

	MILES									
Direction From	0.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
N	3.78E-06	6.63E-07	2.63E-07	1.33E-07	8.22E-08	2.82E-08	6.81E-09	2.87E-09	1.61E-09	1.03E-09
NNE	7.41E-06	1.38E-06	5.88E-07	3.02E-07	1.89E-07	6.52E-08	1.59E-08	6.80E-09	3.86E-09	2.50E-09
NE	1.56E-05	2.73E-06	1.19E-06	6.43E-07	4.12E-07	1.51E-07	4.04E-08	1.76E-08	1.01E-08	6.61E-09
ENE	4.53E-05	7.74E-06	3.61E-06	2.05E-06	1.33E-06	4.88E-07	1.25E-07	5.28E-08	3.05E-08	2.02E-08
E	2.04E-05	3.45E-06	1.46E-06	7.89E-07	5.09E-07	1.92E-07	5.34E-08	2.34E-08	1.34E-08	8.78E-09
ESE	1.16E-05	2.07E-06	8.96E-07	4.80E-07	3.08E-07	1.15E-07	2.81E-08	1.07E-08	6.07E-09	3.94E-09
SE	1.15E-05	2.10E-06	9.15E-07	4.95E-07	3.17E-07	1.20E-07	2.64E-08	8.44E-09	4.79E-09	3.10E-09
SSE	8.29E-06	1.50E-06	6.35E-07	3.42E-07	2.22E-07	8.87E-08	2.05E-08	6.38E-09	3.62E-09	2.35E-09
S	7.13E-06	1.40E-06	6.53E-07	3.64E-07	2.41E-07	1.04E-07	2.56E-08	7.67E-09	4.37E-09	2.84E-09
SSW	7.63E-06	1.43E-06	6.29E-07	3.42E-07	2.20E-07	8.50E-08	1.97E-08	6.55E-09	3.73E-09	2.42E-09
SW	6.07E-06	1.13E-06	5.15E-07	2.83E-07	1.84E-07	7.50E-08	1.72E-08	5.07E-09	2.88E-09	1.86E-09
WSW	3.12E-06	5.56E-07	2.47E-07	1.39E-07	9.20E-08	4.04E-08	1.12E-08	3.68E-09	1.69E-09	8.43E-10
W	1.45E-06	2.54E-07	1.05E-07	5.51E-08	3.50E-08	1.33E-08	3.20E-09	1.13E-09	6.33E-10	4.01E-10
WNW	1.10E-06	1.86E-07	7.12E-08	3.57E-08	2.20E-08	7.53E-09	1.81E-09	7.49E-10	4.16E-10	2.63E-10
NW	1.86E-06	3.20E-07	1.22E-07	5.94E-08	3.64E-08	1.22E-08	2.86E-09	1.18E-09	6.54E-10	4.13E-10
NNW	2.35E-06	4.09E-07	1.65E-07	8.40E-08	5.12E-08	1.66E-08	3.67E-09	1.52E-09	8.48E-10	5.37E-10

TABLE 2.3-140

1999-2003 SSES RELATIVE DEPOSITION D/Q X/Q ACCUMULATION FOR DEPOSITION (per square meter)										
	MILES									
Direction From	0.5-1	1-2	2-3	3-4	4-5	5-10	10-20	20-30	30-40	40-50
N	2.57E-08	3.78E-09	1.55E-09	7.35E-10	4.34E-10	1.38E-10	3.29E-11	1.21E-11	6.47E-12	4.06E-12
NNE	3.31E-08	5.13E-09	2.23E-09	1.06E-09	6.23E-10	1.94E-10	4.53E-11	1.67E-11	8.89E-12	5.59E-12
NE	4.05E-08	6.11E-09	2.61E-09	1.27E-09	7.51E-10	2.42E-10	5.93E-11	2.18E-11	1.16E-11	7.31E-12
ENE	7.28E-08	1.13E-08	5.04E-09	2.46E-09	1.46E-09	4.61E-10	1.05E-10	3.68E-11	1.96E-11	1.23E-11
E	3.40E-08	4.96E-09	2.02E-09	9.65E-10	5.76E-10	1.90E-10	4.81E-11	1.77E-11	9.45E-12	5.94E-12
ESE	2.36E-08	3.57E-09	1.52E-09	7.36E-10	4.40E-10	1.46E-10	3.31E-11	1.07E-11	5.71E-12	3.59E-12
SE	3.04E-08	4.62E-09	2.02E-09	9.97E-10	5.98E-10	2.02E-10	4.21E-11	1.15E-11	6.14E-12	3.86E-12
SSE	2.52E-08	3.76E-09	1.60E-09	7.92E-10	4.83E-10	1.73E-10	3.79E-11	1.01E-11	5.38E-12	3.38E-12
S	2.61E-08	4.19E-09	1.97E-09	1.01E-09	6.30E-10	2.44E-10	5.69E-11	1.46E-11	7.80E-12	4.90E-12
SSW	3.58E-08	5.48E-09	2.46E-09	1.24E-09	7.53E-10	2.63E-10	5.84E-11	1.67E-11	8.89E-12	5.59E-12
SW	4.76E-08	7.56E-09	3.56E-09	1.84E-09	1.14E-09	4.28E-10	9.66E-11	2.46E-11	1.31E-11	8.25E-12
WSW	3.14E-08	4.84E-09	2.26E-09	1.21E-09	7.69E-10	3.17E-10	8.93E-11	2.58E-11	1.13E-11	5.54E-12
W	1.29E-08	1.93E-09	8.36E-10	4.17E-10	2.54E-10	9.10E-11	2.22E-11	6.96E-12	3.71E-12	2.33E-12
WNW	9.68E-09	1.40E-09	5.63E-10	2.69E-10	1.60E-10	5.17E-11	1.28E-11	4.71E-12	2.51E-12	1.58E-12
NW	1.67E-08	2.45E-09	9.82E-10	4.57E-10	2.70E-10	8.57E-11	2.06E-11	7.57E-12	4.04E-12	2.54E-12
NNW	2.15E-08	3.19E-09	1.35E-09	6.51E-10	3.81E-10	1.16E-10	2.62E-11	9.63E-12	5.14E-12	3.23E-12



Fugita Intensity Classification

F-Scale	Wind Speed (MPH)	Description
0	40-75	Very Weak Tornado (light damage)
1	73-112	Weak Tornado
2	113-154	Strong Tornado
3	155-206	Severe Tornado
4	207-260	Devasting Tornado
5	261-318	Incredible Tornado
6	319 and higher	Inconceivable Tornado

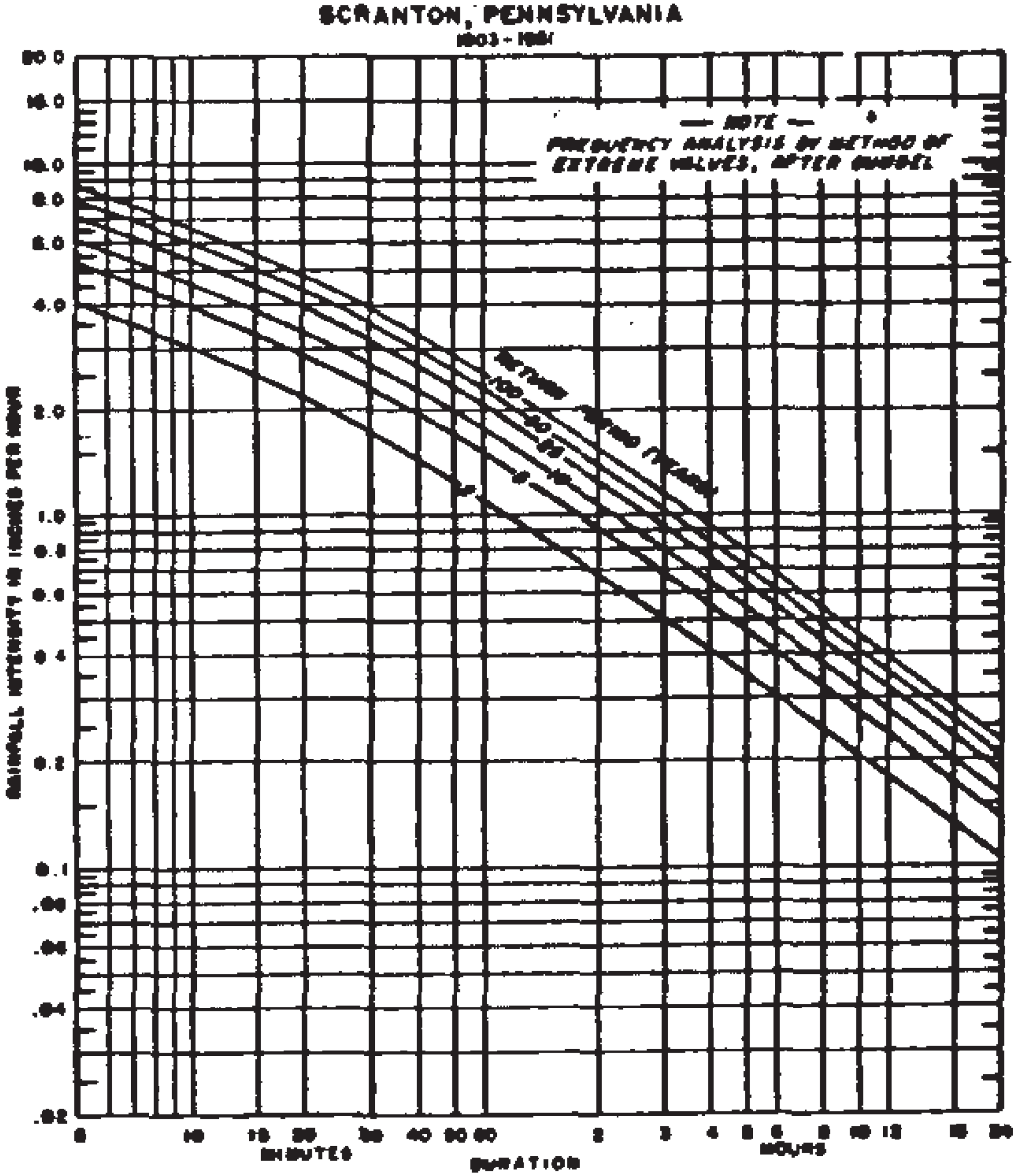
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

TORNADO OCCURRENCE
AND INTENSITY IN
SUSQUEHANNA REGION

FIGURE 2.3-1, Rev 47

AutoCAD: Figure Fsar 2_3_1.dwg



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UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

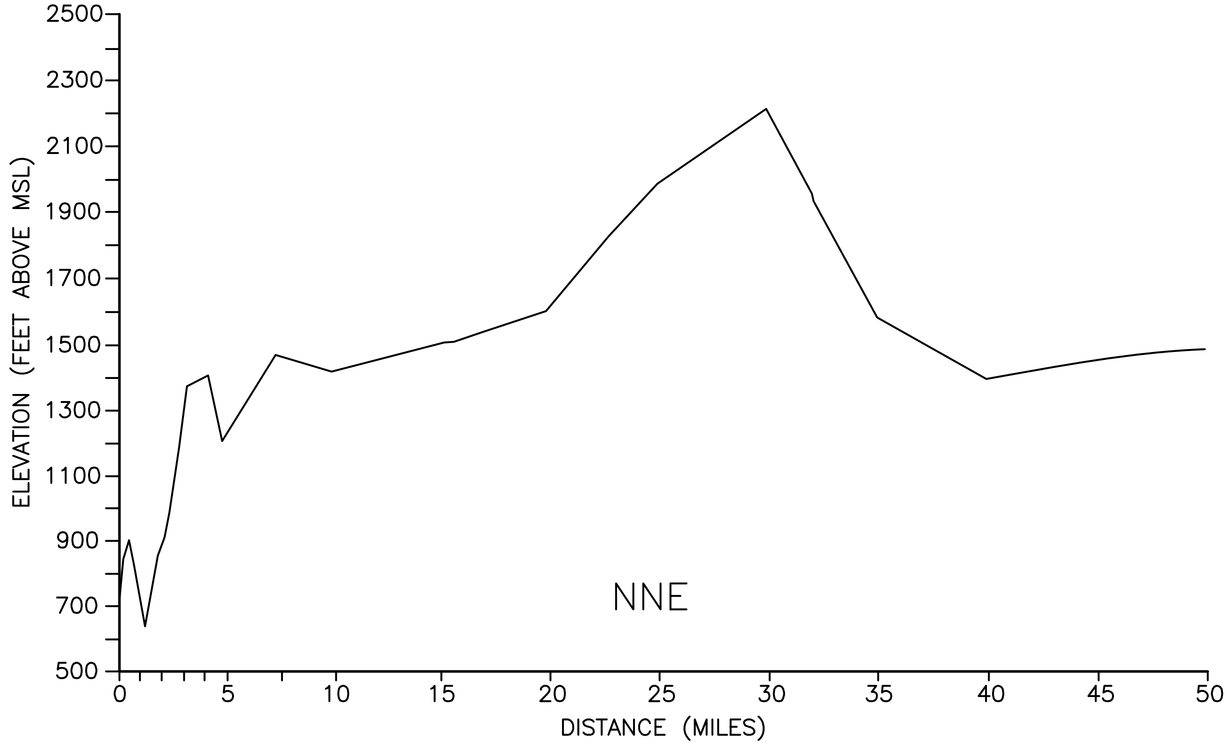
TP 25
RAINFALL INTENSITY-DURATION
FREQUENCY CURVES
USDC WB, 1955

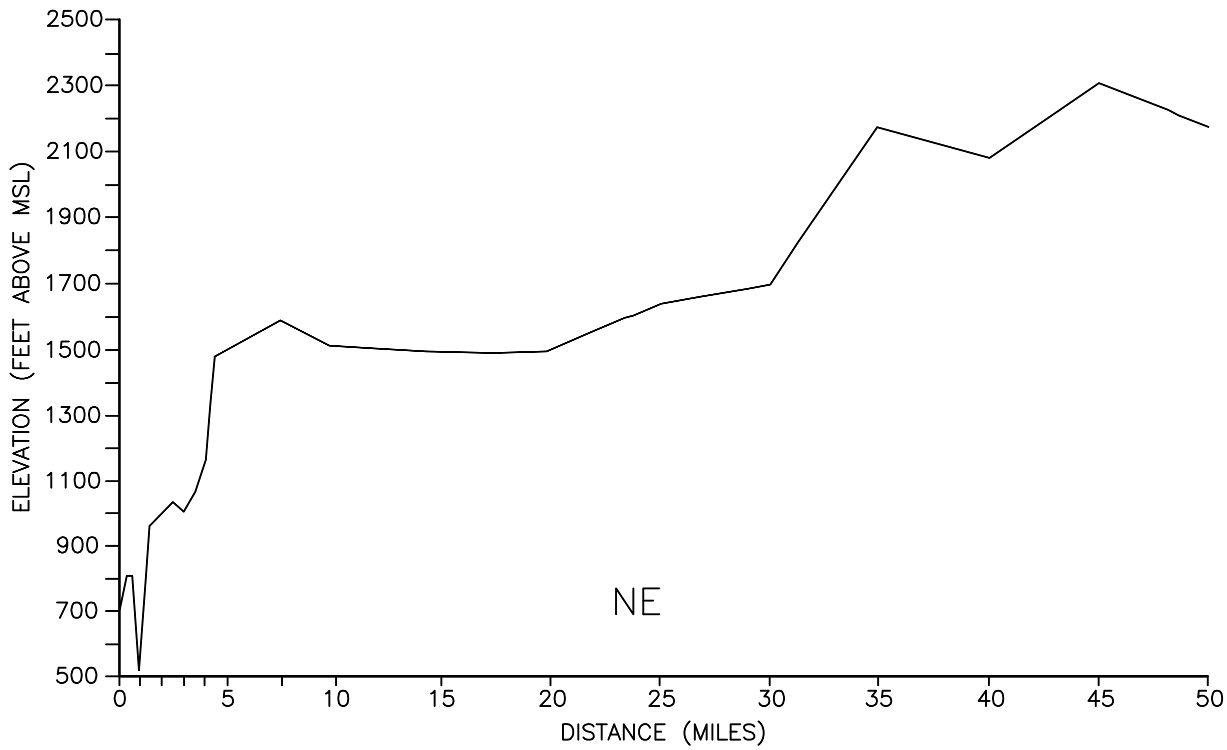
FIGURE 2.3-2, Rev 47

AutoCAD: Figure Fsar 2_3_2.dwg

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
TOPOGRAPHY WITHIN 5 MILES FIGURE 2.3-3





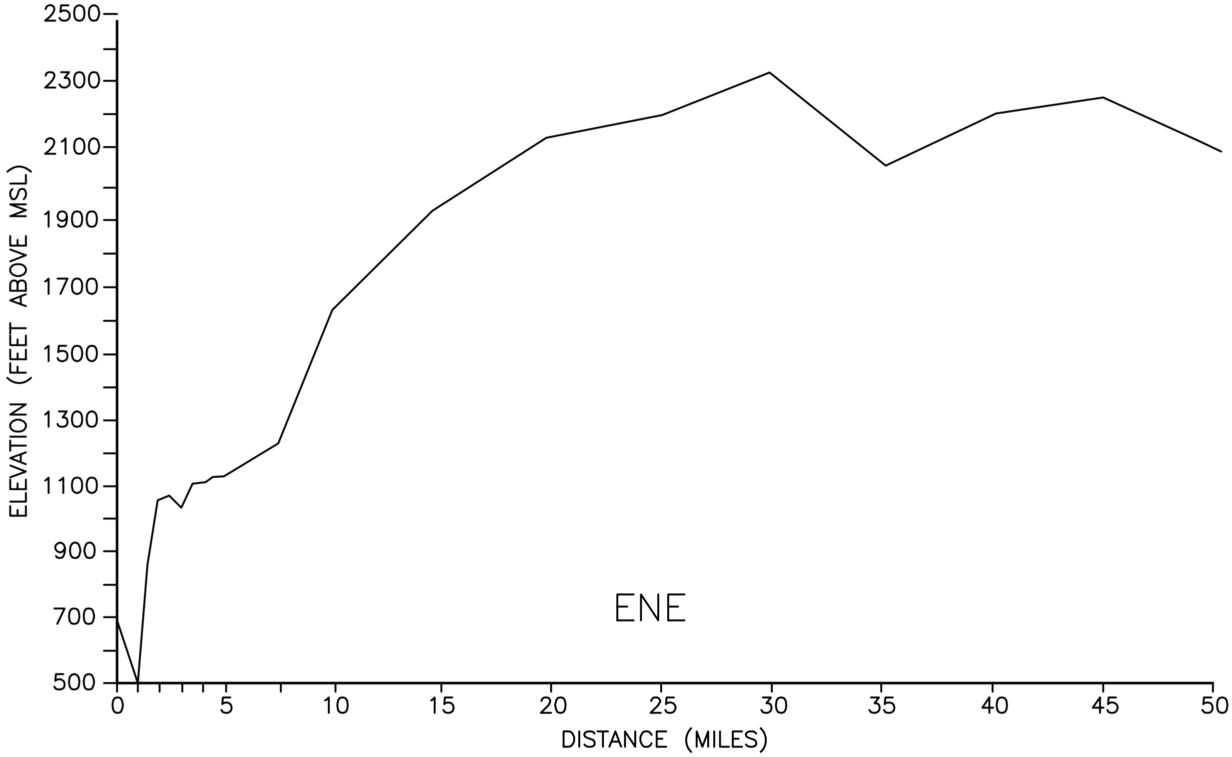
FSAR REV. 65

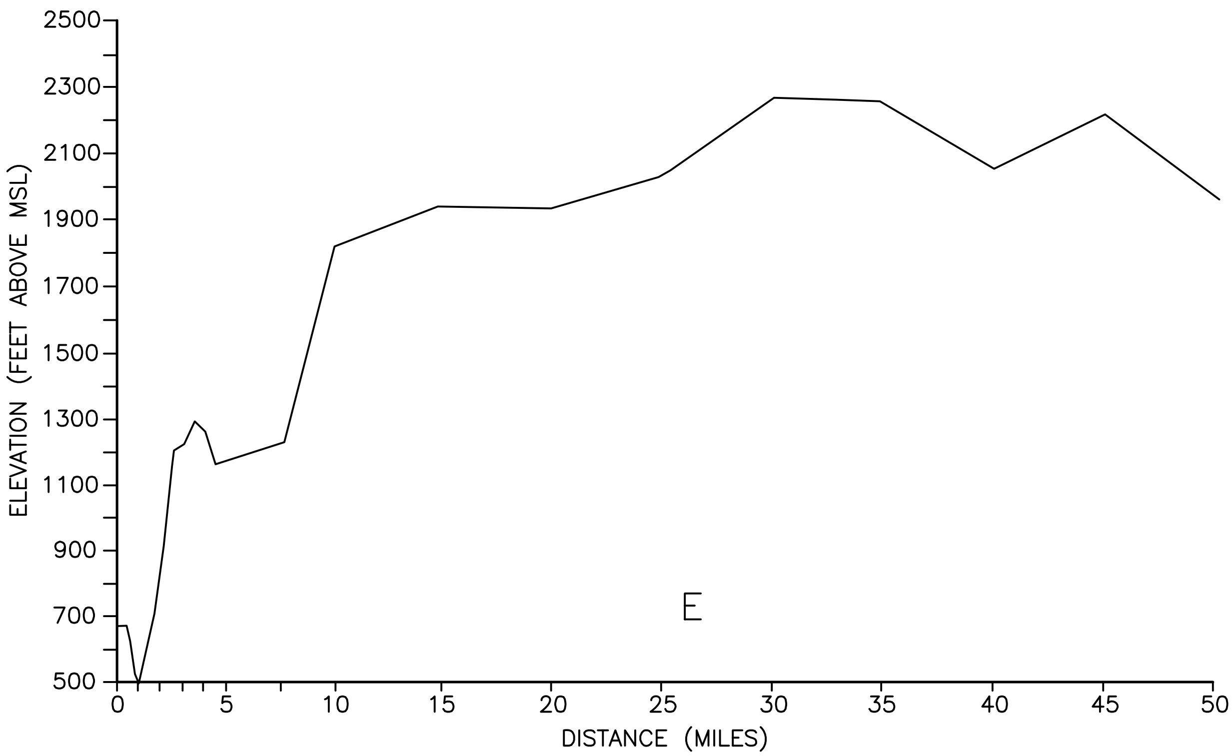
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

MAXIMUM TERRAIN ELEVATION
VERSUS
DISTANCE BY SECTOR

FIGURE 2.3-4-1, Rev 47 Sh. 1 of 8

AutoCAD: Figure Fsar 2_3_4_1.dwg





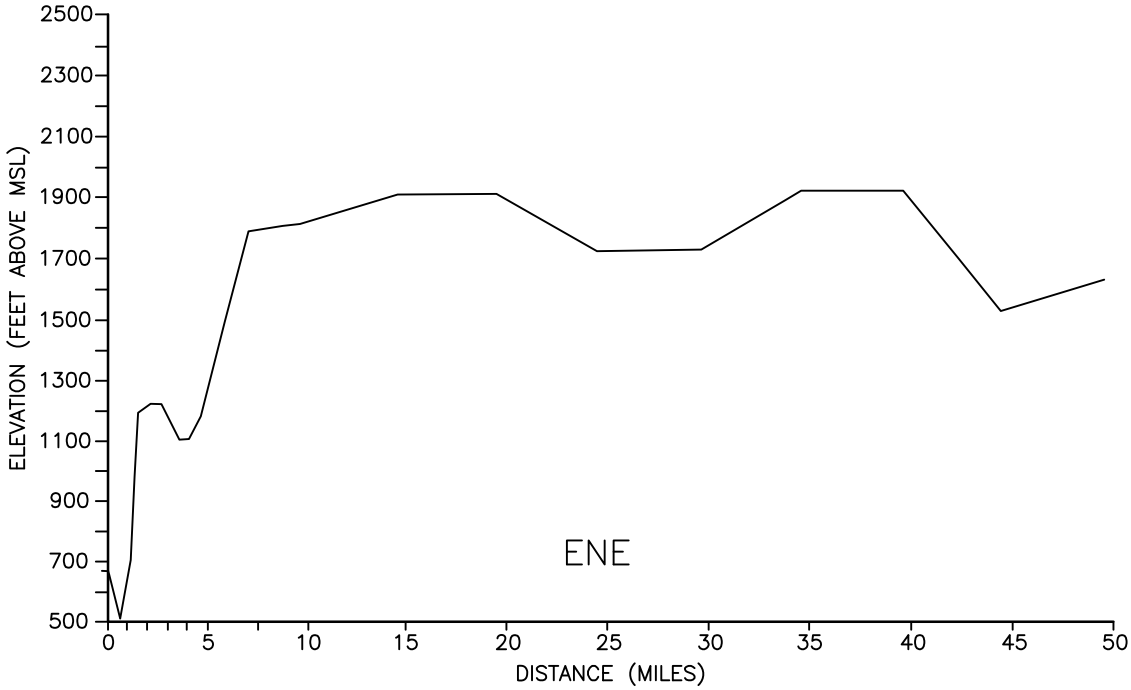
FSAR REV. 65

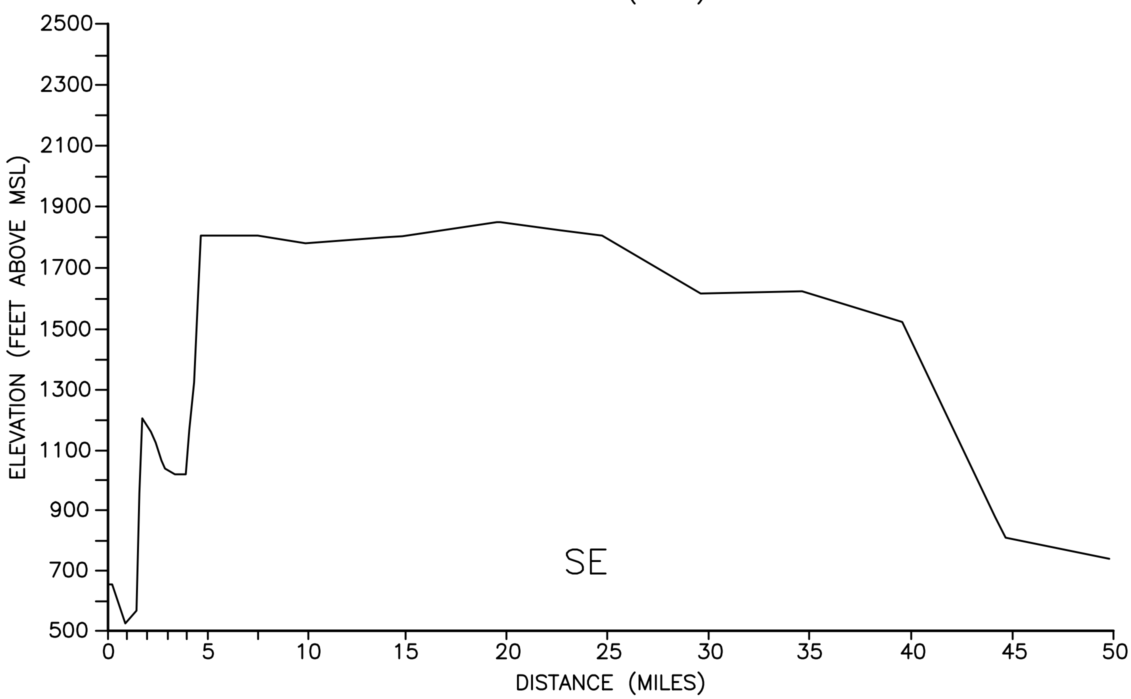
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

MAXIMUM TERRAIN ELEVATION
VERSUS
DISTANCE BY SECTOR

FIGURE 2.3-4-2, Rev 47 (Cont'd) Sh. 2 of 8

AutoCAD: Figure Fsar 2_3_4_2.dwg





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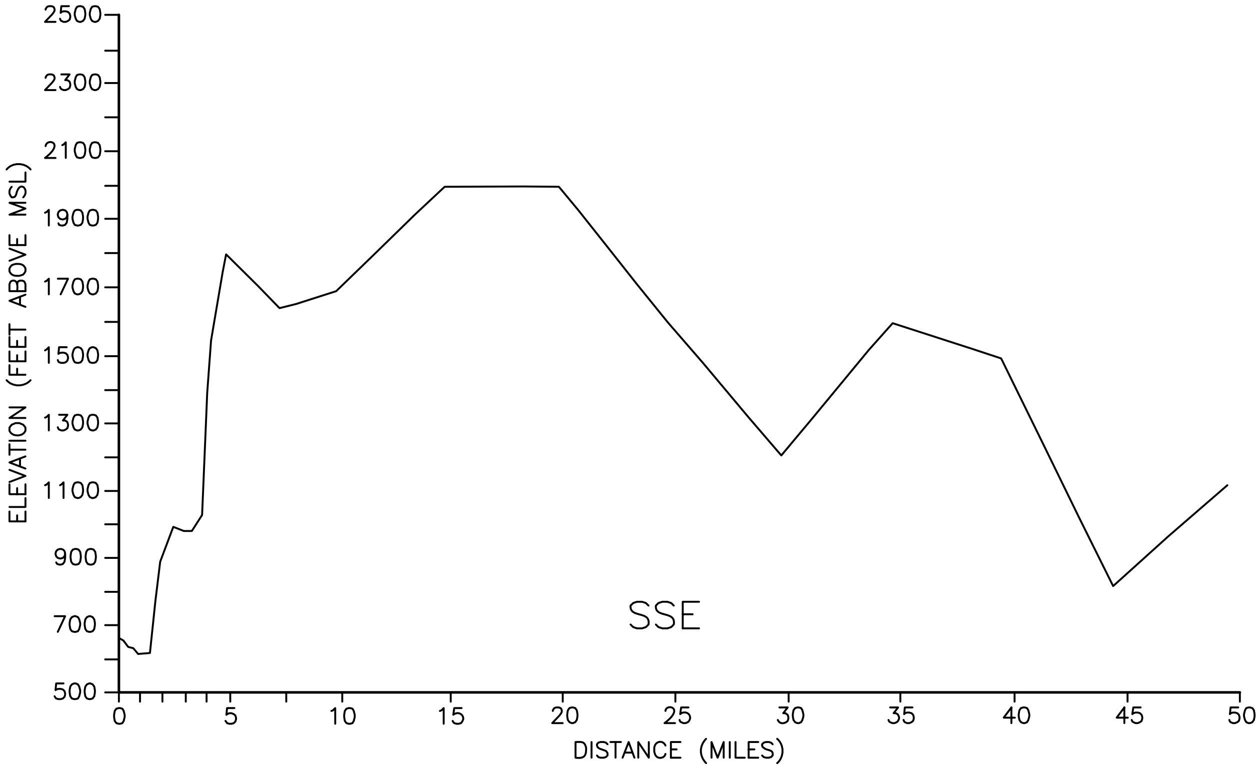
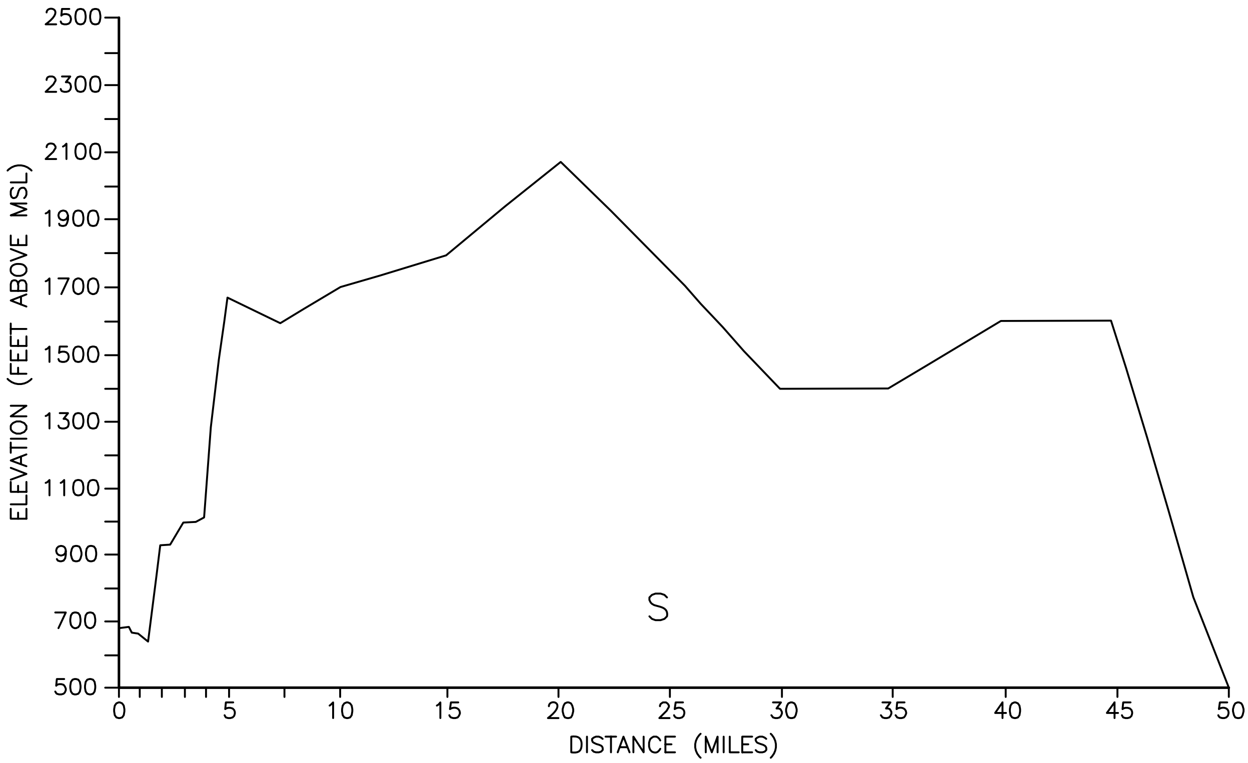
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

MAXIMUM TERRAIN ELEVATION
VERSUS
DISTANCE BY SECTOR

FIGURE 2.3-4-3, Rev 47 Sh. 3 of 8

AutoCAD: Figure Fsar 2_3_4_3.dwg





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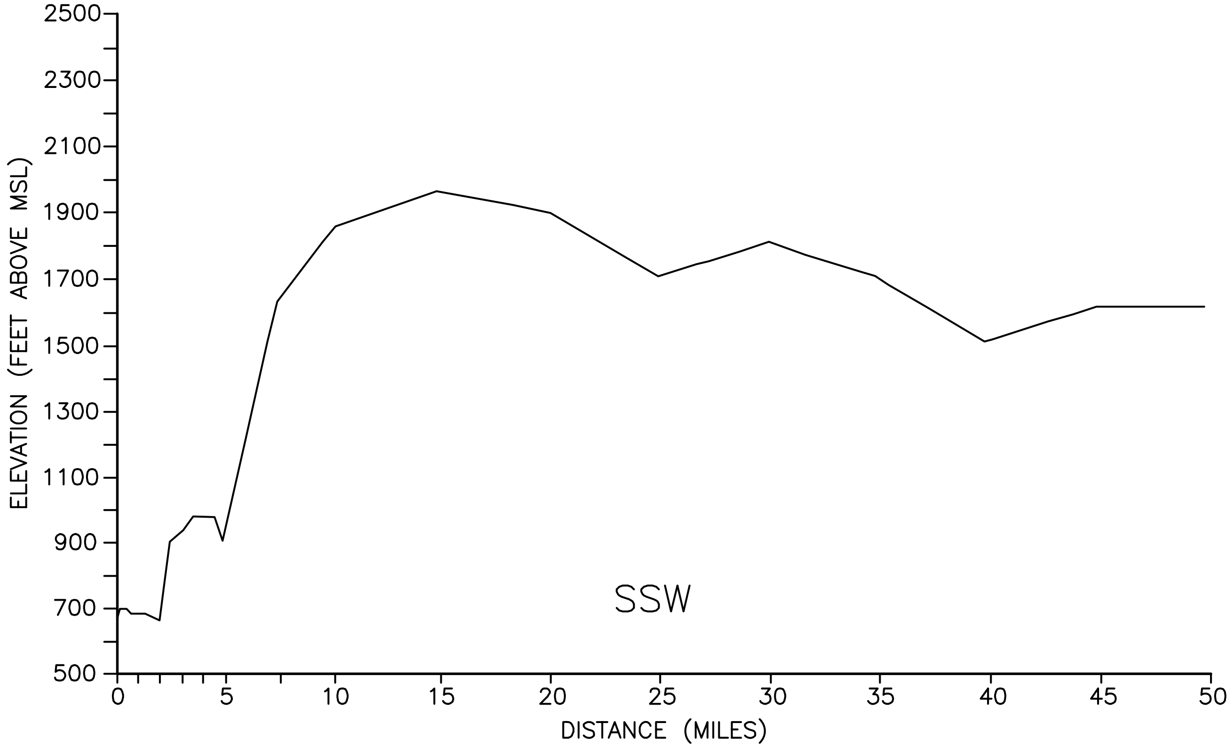
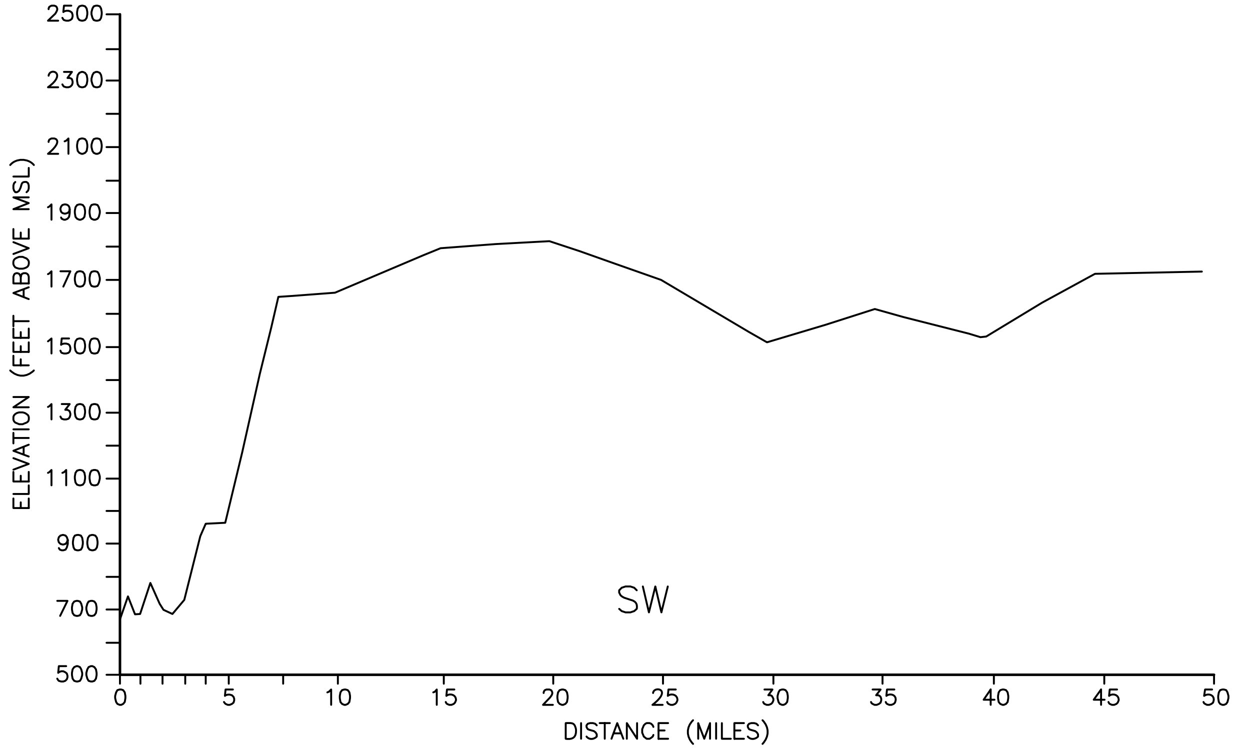
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

MAXIMUM TERRAIN ELEVATION
VERSUS
DISTANCE BY SECTOR

FIGURE 2.3-4-4, Rev 47 Sh. 4 of 8

AutoCAD: Figure Fsar 2_3_4_4.dwg





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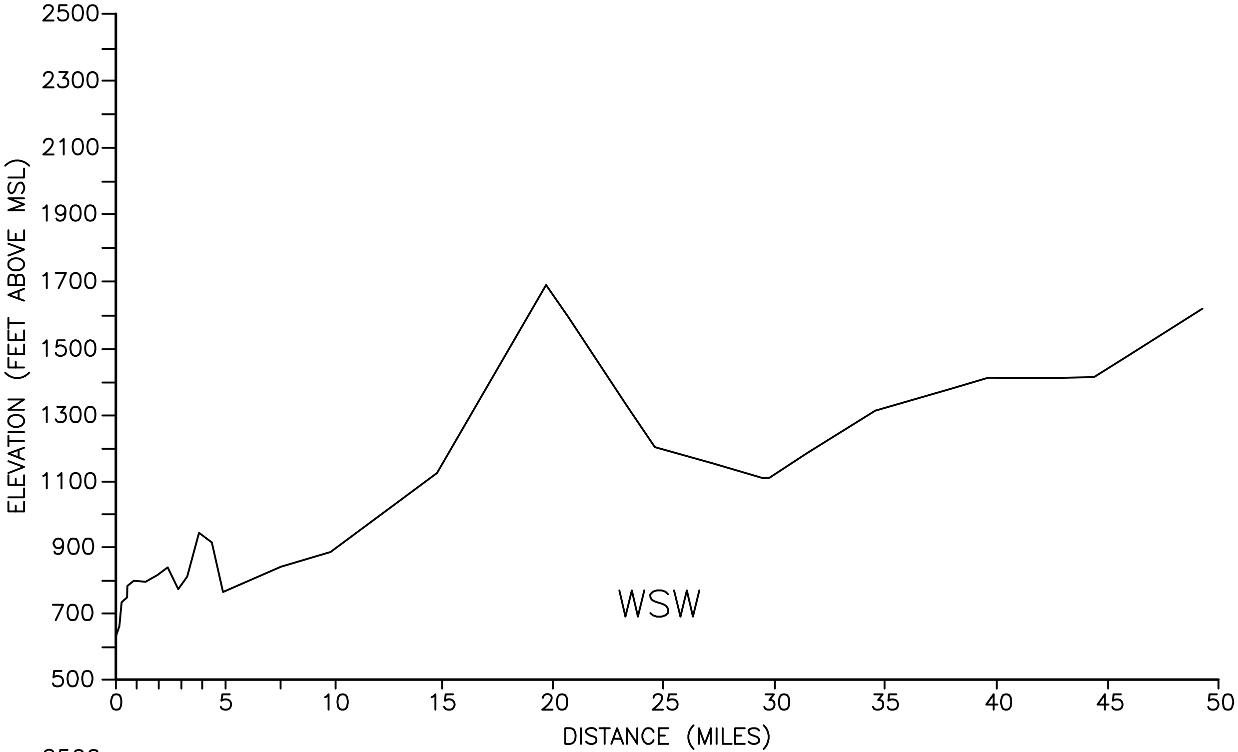
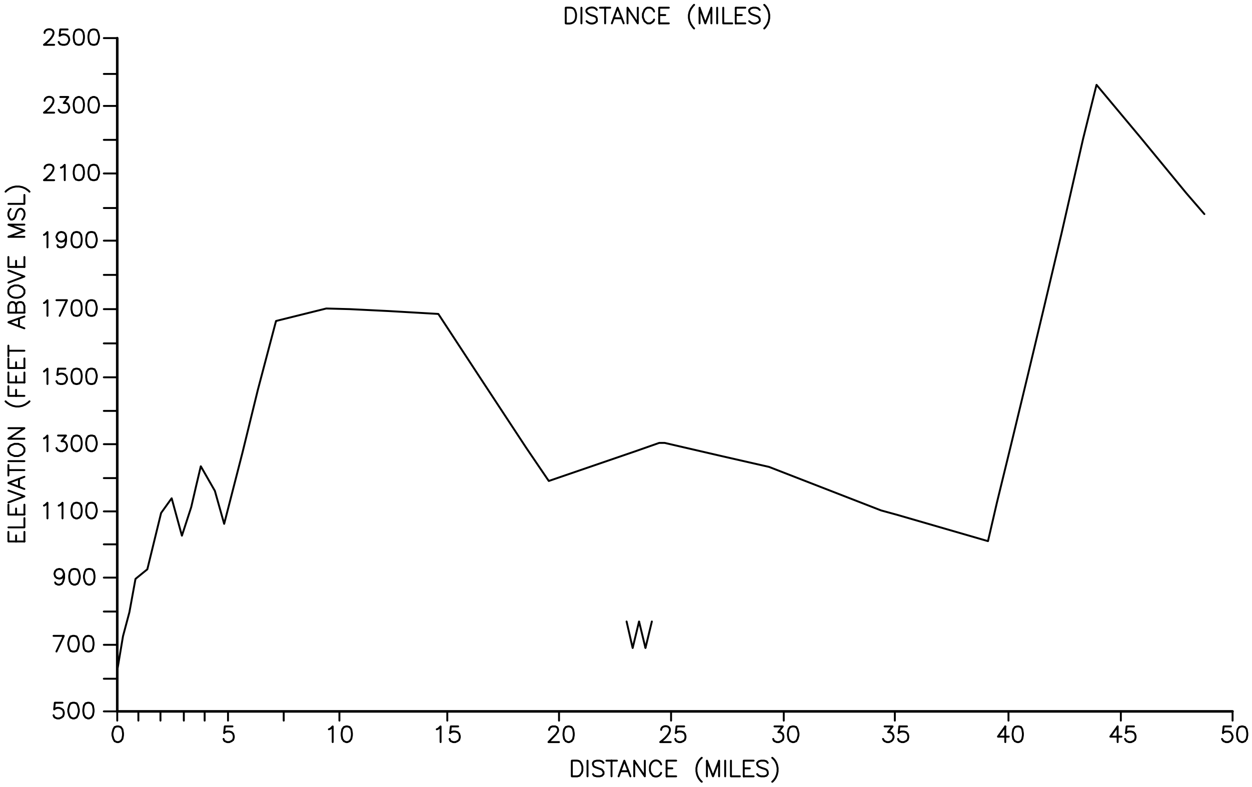
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

MAXIMUM TERRAIN ELEVATION
VERSUS
DISTANCE BY SECTOR

FIGURE 2.3-4-5, Rev 47 Sh. 5 of 8

AutoCAD: Figure Fsar 2_3_4_5.dwg





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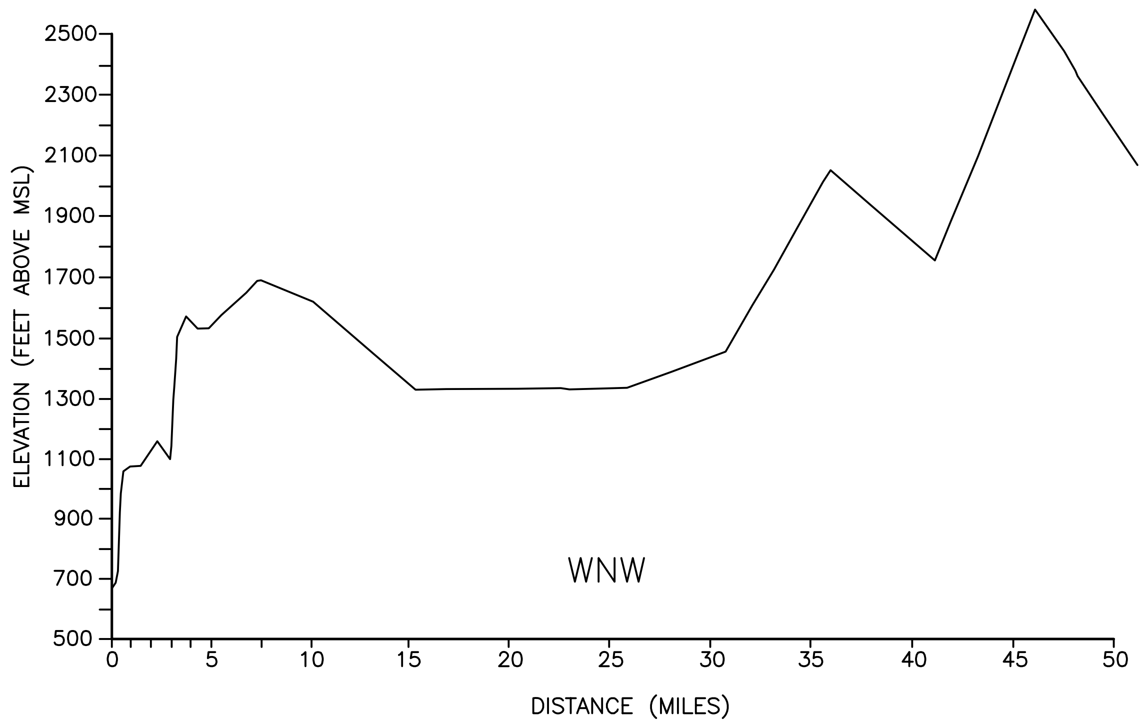
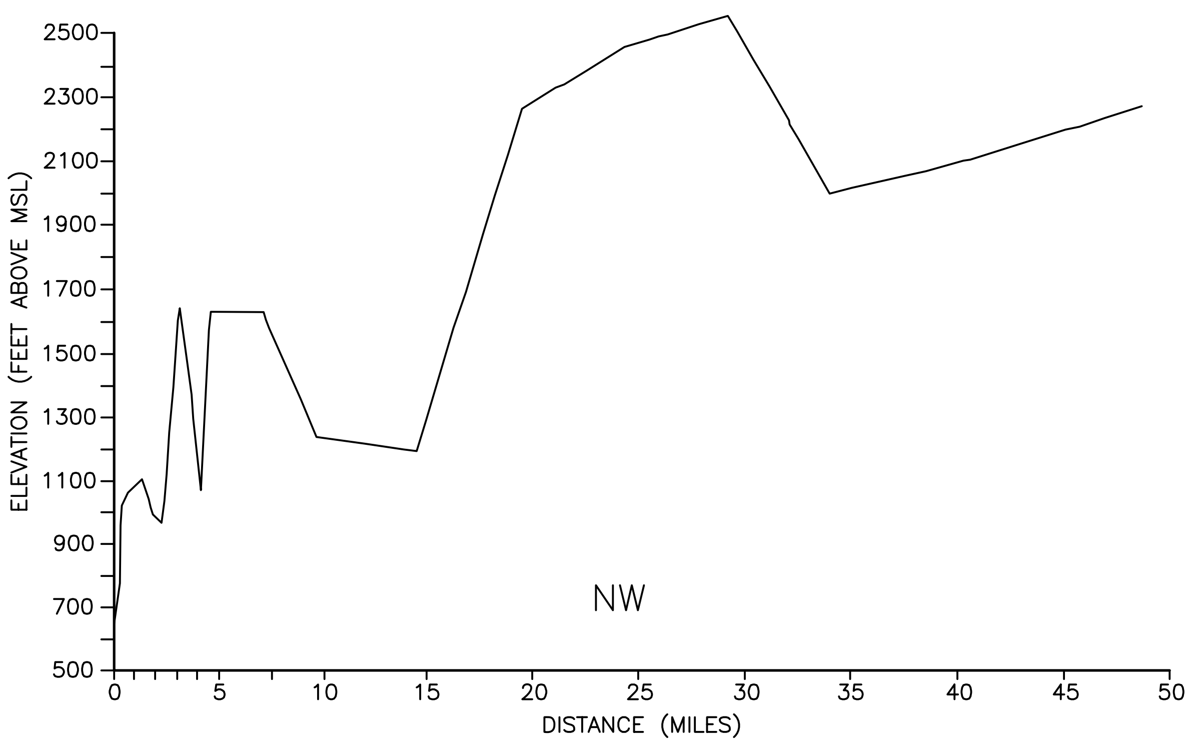
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

MAXIMUM TERRAIN ELEVATION
VERSUS
DISTANCE BY SECTOR

FIGURE 2.3-4-6, Rev 47 Sh. 6 of 8

AutoCAD: Figure Fsar 2_3_4_6.dwg





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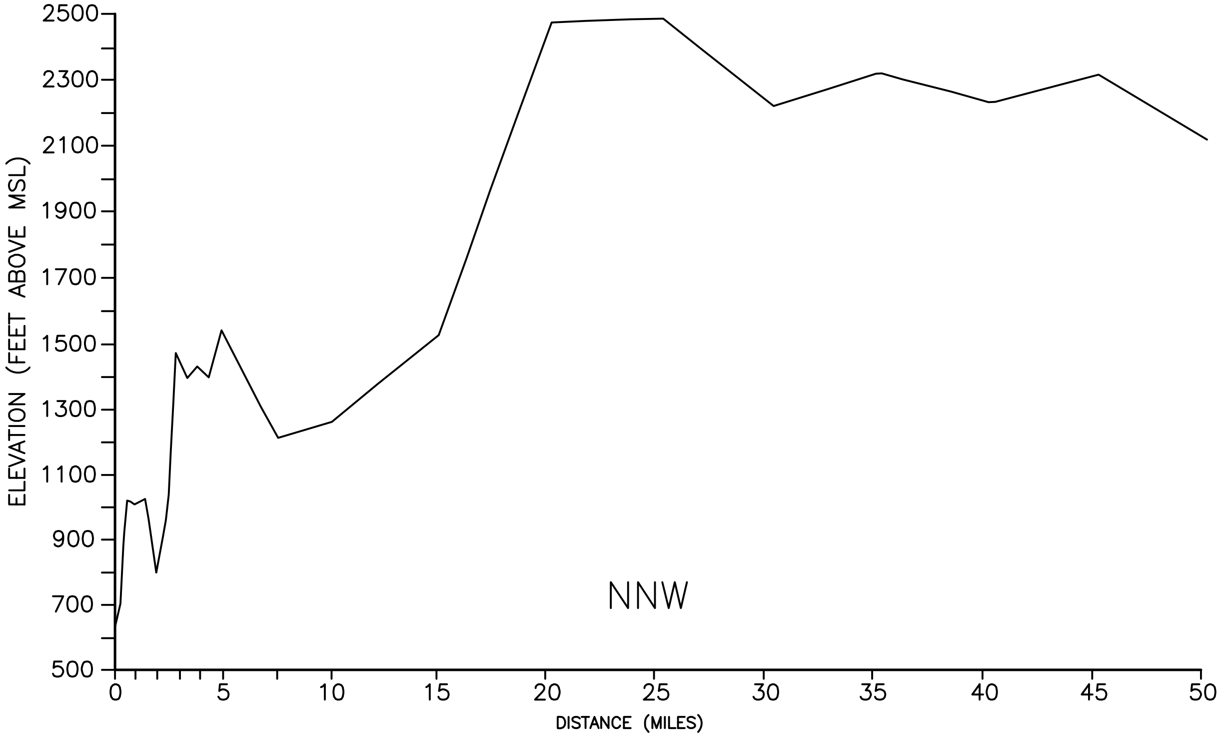
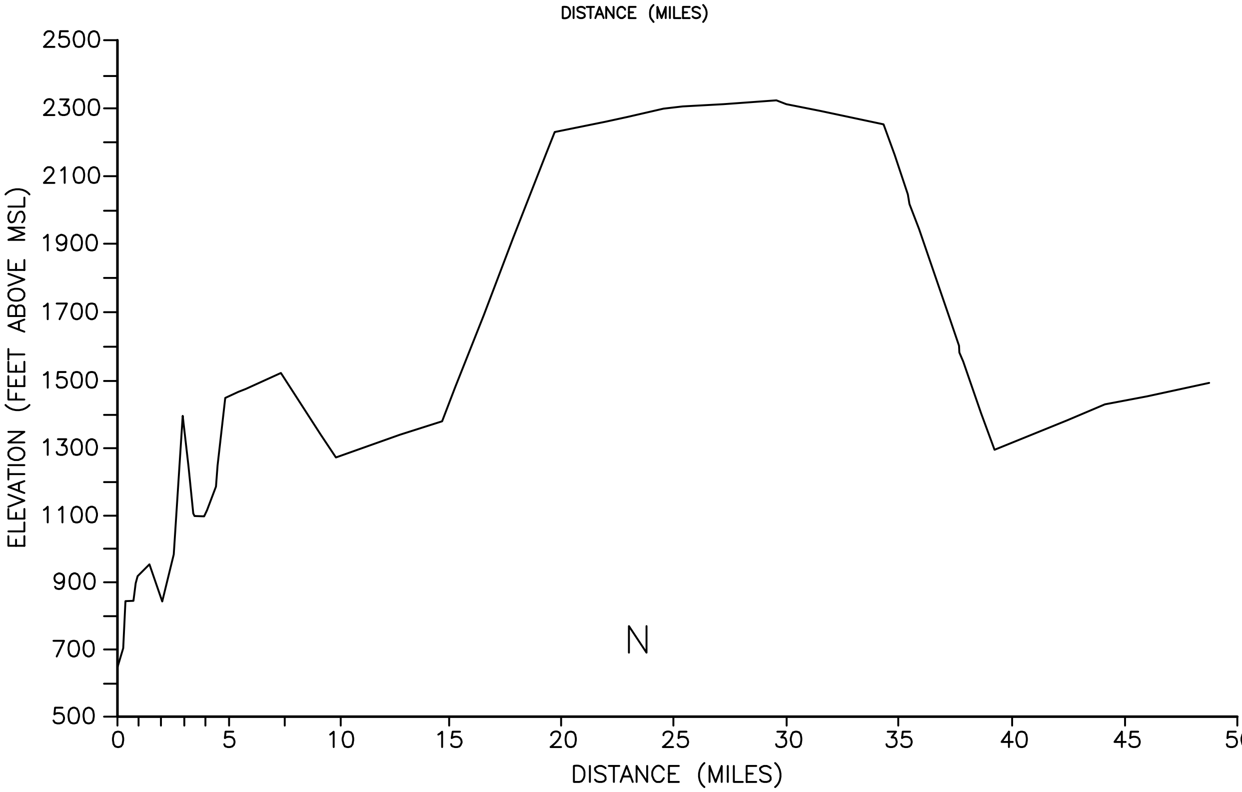
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

MAXIMUM TERRAIN ELEVATION
VERSUS
DISTANCE BY SECTOR

FIGURE 2.3-4-7, Rev 47 Sh. 7 of 8

AutoCAD: Figure Fsar 2_3_4_7.dwg





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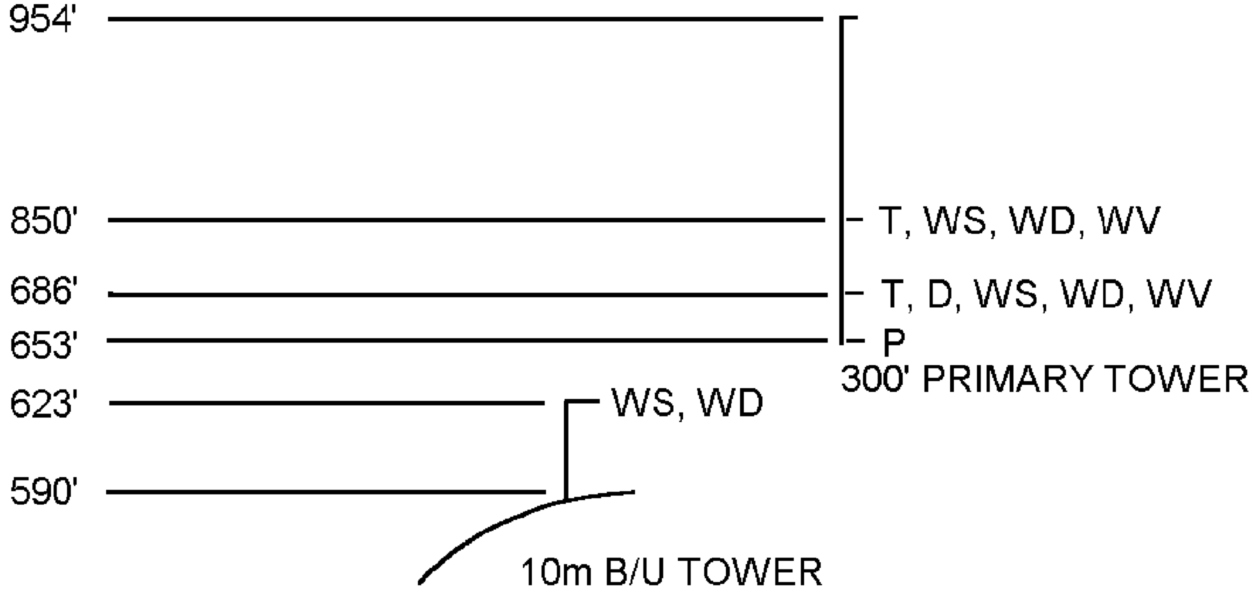
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

MAXIMUM TERRAIN ELEVATION
VERSUS
DISTANCE BY SECTOR

FIGURE 2.3-4-8, Rev 47 Sh. 8 of 8

AutoCAD: Figure Fsar 2_3_4_8.dwg



KEY TO INSTRUMENTATION

T = TEMPERATURE
D = DEW POINT TEMPERATURE
WS = WIND SPEED
WD = WIND DIRECTION
WV = WIND VARIABILITY (NOTE 1)
P = PRECIPITATION

NOTE:
1. WIND VARIABILITY IS NOT A DIRECT MEASUREMENT AT THE TOWER, BUT TRANSLATED AT THE MAINFRAME IN THE TOWER BUILDING FROM WD DATA.

FSAR REV. 65

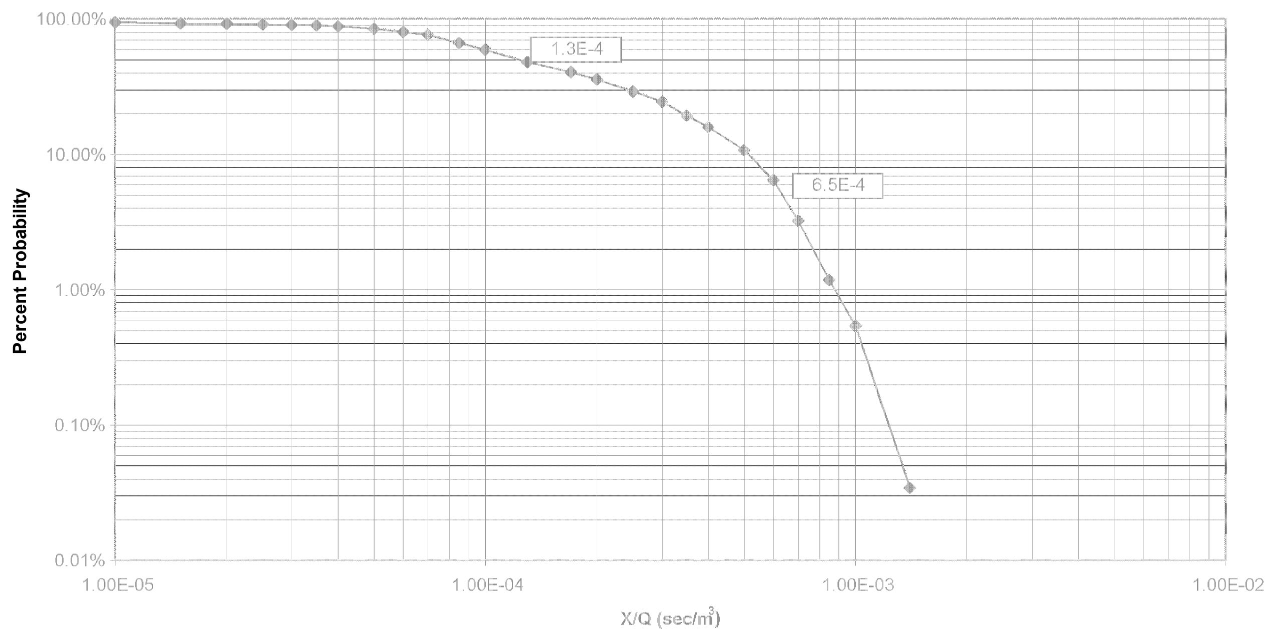
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SCHEMATIC OF INSTRUMENTATION

FIGURE 2.3-5, Rev 54

AutoCAD: Figure Fsar 2_3_5.dwg

Figure 2.3-6 - 1 Hour Direction Independent X/Q at the EAB
(Weighted Average of 1999, 2000, 2001, 2002, 2003 Calculations)



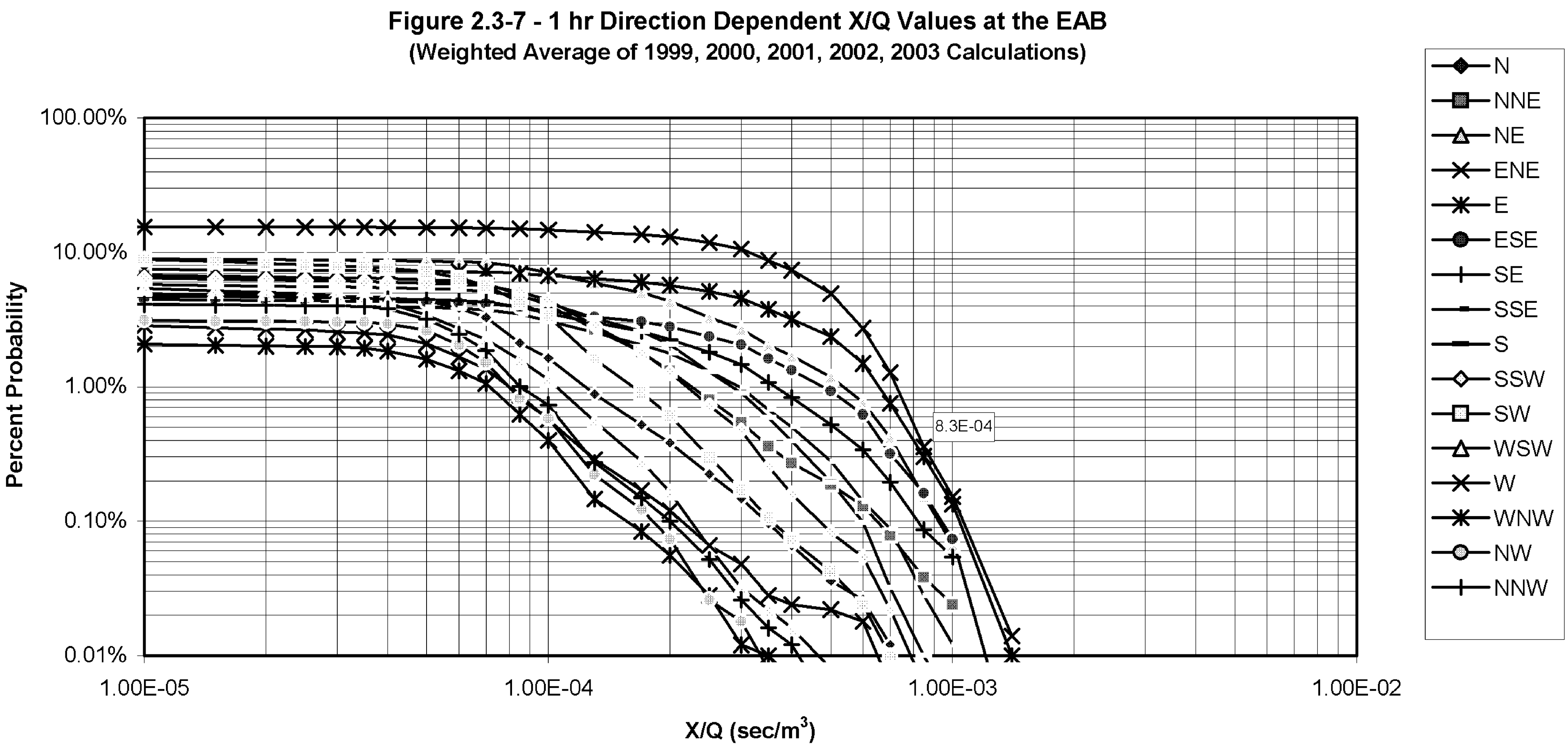
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

ONE HOUR DIRECTION INDEPENDENT X/Q
AT THE EAB
(WEIGHTED AVERAGE OF 1999, 2000, 2001,
2002, 2003 CALCULATIONS)

FIGURE 2.3-6, Rev 1

AutoCAD: Figure Fsar 2_3_6.dwg



FSAR REV. 65

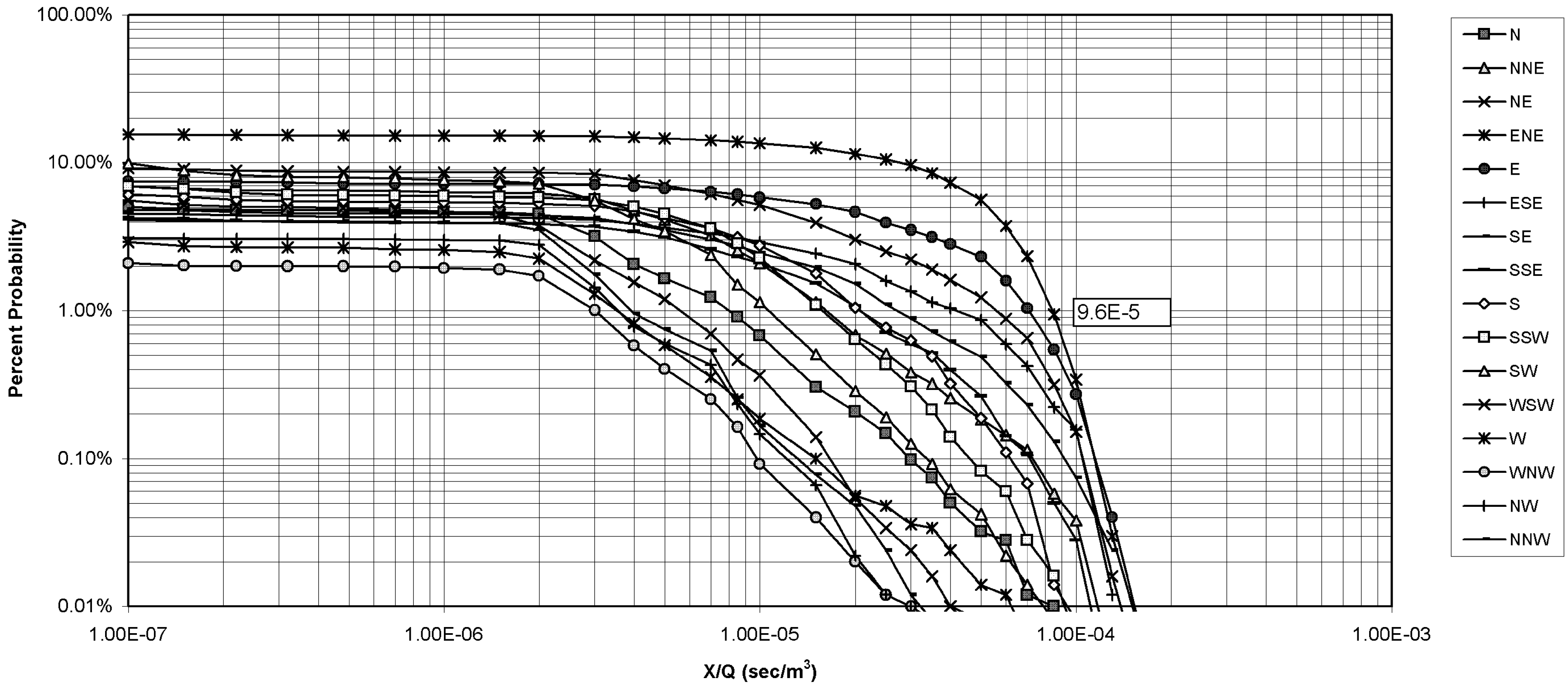
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

ONE HOUR DIRECTION DEPENDENT X/Q VALUES
AT THE EAB
(WEIGHTED AVERAGE OF 1999, 2000, 2001,
2002, 2003 CALCULATIONS)

FIGURE 2.3-7, Rev 1

AutoCAD: Figure Fsar 2_3_7.dwg

Figure 2.3-8 -1 Hour Direction Dependent X/Q Values at the LPZ
(Weighted Average of 1999, 2000, 2001, 2002, 2003 Calculations)

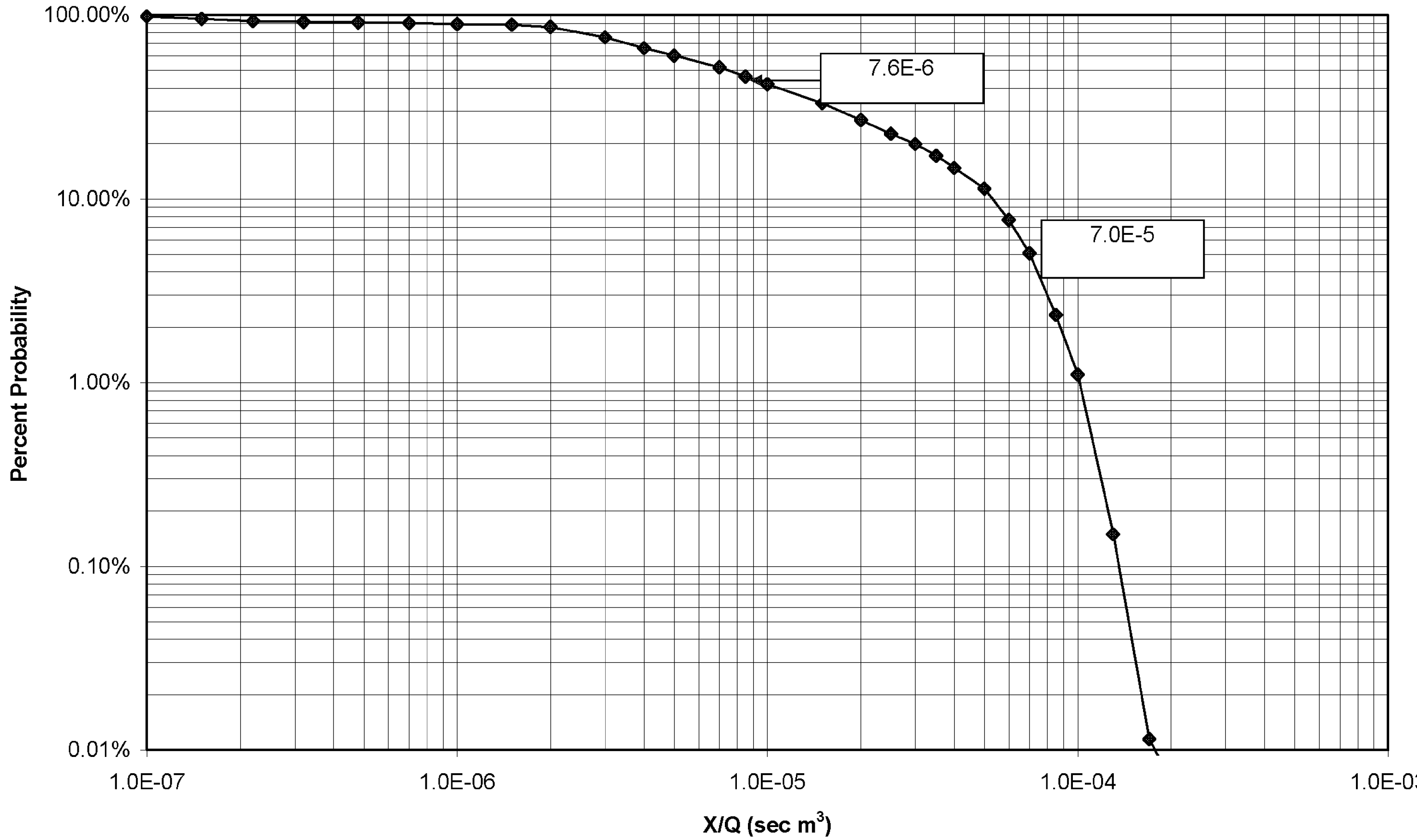


FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
ONE HOUR DIRECTION DEPENDENT X/Q VALVES AT THE LPZ (WEIGHTED AVERAGE OF 1999, 2000, 2001, 2002, 2003 CALCULATIONS) FIGURE 2.3-8, Rev 1

AutoCAD: Figure Fsar 2_3_8.dwg

Figure 2.3-9 - 1 Hour Direction Independent X/Q at the LPZ
(weighted average of 1999, 2000, 2001, 2002, 2003 Calculations)

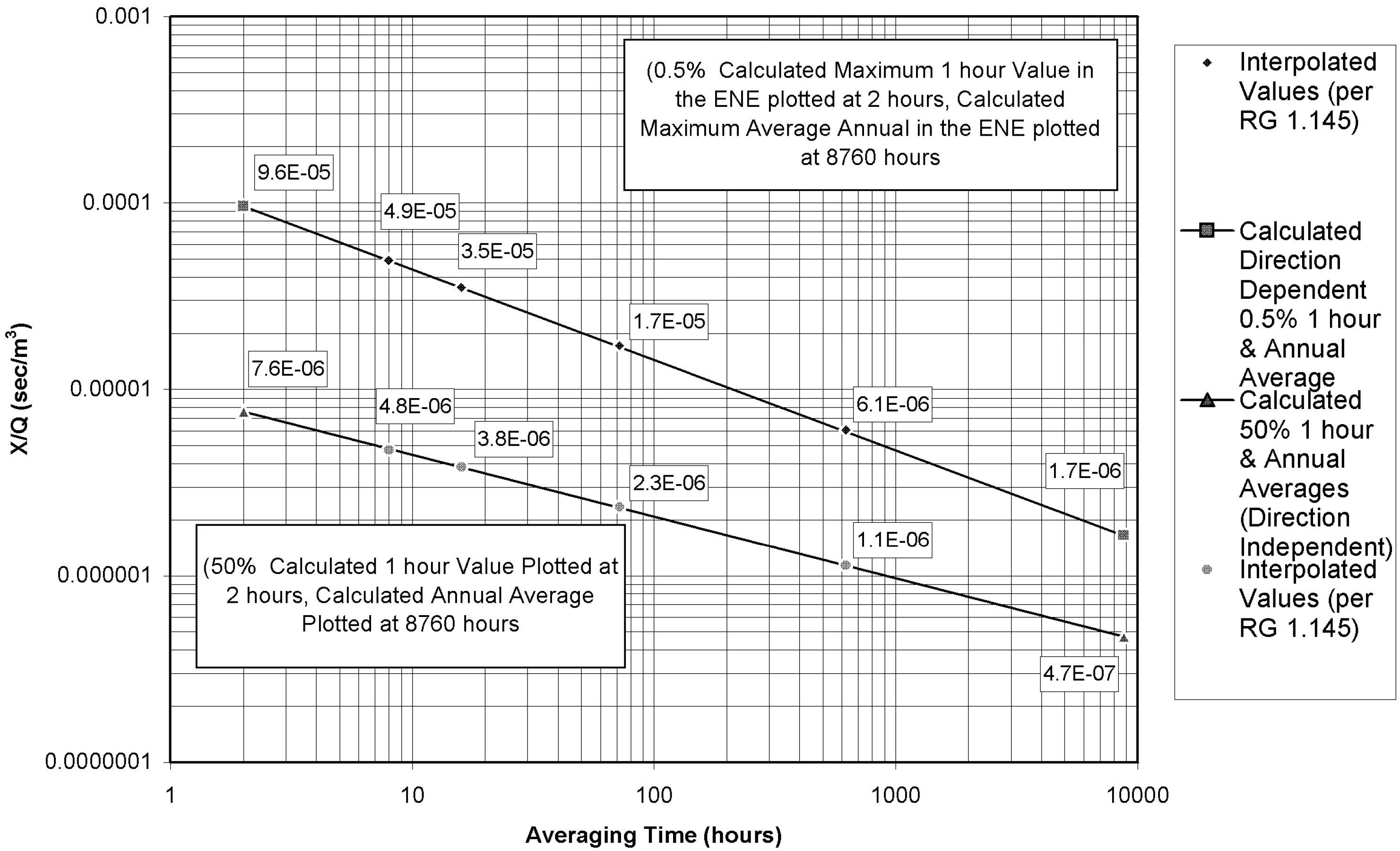


FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
ONE HOUR DIRECTION DEPENDENT X/Q AT THE LPZ (WEIGHTED AVERAGE OF 1999, 2000, 2001, 2002, 2003 CALCULATIONS) FIGURE 2.3-9, Rev 1

AutoCAD: Figure Fsar 2_3_9.dwg

Figure 2.3-10 - Interpolated X/Q Values at the LPZ
(Weighted Average of 1999, 2000, 2001, 2002, 2003 Calculations)



FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

INTERPOLATED X/Q VALUES AT THE LPZ
(WEIGHTED AVERAGE OF 1999, 2000, 2001, 2002, 2003 CALCULATIONS)

FIGURE 2.3-10, Rev 1

AutoCAD: Figure Fsar 2_3_10.dwg

2.4 HYDROLOGIC ENGINEERING

2.4.1 HYDROLOGIC DESCRIPTION

2.4.1.1 Site and Facilities

**Security-Related Information
Text Withheld Under 10 CFR 2.390**

2.4.1.2 Hydrosphere

Included in this section is a description of the location, size, shape and other hydrologic characteristics of the streams and reservoirs comprising the surface water hydrosphere. A description of the groundwater environments influencing plant siting is included in Subsection 2.4.13.1.

2.4.1.2.1 Rivers and Streams

Security-Related Information Text Withheld Under 10 CFR 2.390

2.4.1.2.2 Dams and Reservoirs

**Security-Related Information
Text Withheld Under 10 CFR 2.390**

START HISTORICAL

**Security-Related Information
Text Withheld Under 10 CFR 2.390**

Security-Related Information
Text Withheld Under 10 CFR 2.390

2.4.1.2.3 Downstream Water Uses

Information has been collected for known points of water use within a 50 mile water route distance from the Susquehanna SES site as required by 10 CFR 50 Appendix I. Included are users on or near the Main Branch downstream of the site, since these are the only locations where detectable amounts of radioactivity could possibly affect such use. The types of use found are municipal water supply, industrial use, and recreation. Most of this information was obtained from the Pennsylvania Department of Environmental Resources (DER) (Ref. 2.4-9 through 2.4-13). Listed in Table 2.4-3 are known water users on or near the Main Branch of the Susquehanna with their location, type of use, radial and water route distance from the station site, present total and consumptive use and, where available, projected use with sources and dates of projections. The locations of the water users in Table 2.4-3 are indicated on the map of the river presented in Figure 2.4-7. Users on the map can be identified by the column entitled "map code" in Table 2.4-3.

Information on municipal water users was provided by the Pennsylvania DER through Water Company Consolidated Inventory Report (Ref. 2.4-9), Surface Water Use Summary Reports (Ref. 2.4-10) and Personal Communication (Ref. 2.4-11). Four municipal water supply companies withdrawing directly from the reach of the Susquehanna downstream of the station site have been identified. These four companies serve the towns of Berwick, Danville, Sunbury, and Shamokin Dam. Projected use for these water companies is presented in Table 2.4-3; however, many water companies in the area have multiple sources and it is not possible to project that their entire use will be drawn from the Susquehanna River. Groundwater is presently the primary source of water supply in the region. The Susquehanna and its tributaries provide a secondary source, but as water demand increases, direct withdrawal from the Susquehanna is expected to increase.

Information concerning industrial water users is also supplied by the Pennsylvania DER (Ref. 2.4-12). Name and type of company, location, water source, and total withdrawal is given in Table 2.4-3. Information on water return to the river or future water demand for these companies is not generally available. The Pennsylvania DER does have estimates of consumptive use vs. total use and also future water demand estimates on an area wide basis for various types of use (municipal, manufacturing, mining, etc.) (Ref. 2.4-13). For the region under study, the consumptive use for manufacturing is about eight percent of the total water use. Total and consumptive use for manufacturing is projected to increase 15 percent by 1970 to 1990. Therefore, a rough estimate of 30 percent may be used for industrial water use increase in the general areaover the station life. Of course, these values should be used only as guidelines, as they pertain to industry for the area

in general and not necessarily to those companies withdrawing water from the Susquehanna. A tabulation of groundwater users is included in Subsection 2.4.13.2.

Points of known recreational use along the Main Branch of the Susquehanna within 50 miles of the station site are listed in Table 2.4-3. Five of the locations are considered to be good fishing locations. Four of these five are listed in the "100 Best Bass Spots in Pennsylvania," a brochure distributed by the Pennsylvania Fish Commission (Ref. 2.4-14). The remaining recreational area is Shikellamy State Park and Marina. The marina is located on the southern tip of Packer's Island at the confluence of the West and Main Branches of the Susquehanna. This portion of the river is called Lake Augusta, a 3,000 acre lake created by the Sunbury Fabridam located 3 miles downstream of the confluence. The Lake Augusta is heavily used for boating activities, including water skiing.

END HISTORICAL

2.4.2 FLOODS

2.4.2.1 Flood History

This Subsection discusses the historical flood events which have occurred on the Main Branch of the Susquehanna River in the vicinity of the Susquehanna SES site. The most severe flood event for this region occurred in June 1972 as the result of the passage of Tropical Storm Agnes through Pennsylvania. In spite of the fact that this flood produced discharges of nearly 1.4 times as great as those of the previous flood on record, the Susquehanna SES site remained over 150 feet above the flood crest. This very substantial margin of safety is additionally reinforced by findings of the flood mechanism evaluations discussed in the following sections. Therefore the classification of the Susquehanna SES as a "dry" site is therefore justified.

2.4.2.1.1 Flood Records

Detailed records of historical floods in the immediate vicinity of the Susquehanna SES do not exist. Data is available, however, from USGS Gaging Stations located at Wilkes-Barre (about 22 miles upstream of the site) and at Danville (about 31 miles downstream). The Corps of Engineers has compiled flood stage and discharge information for the Susquehanna River at Wilkes-Barre (Ref. 2.4-15). These data are based on records of flood stages dating from 1891. Data for the four most severe floods on record are presented in Table 2.4-4. Table 2.4-4 also includes the stages and discharges for floods at Danville. Discharges at the Susquehanna SES site are linearly interpolated between the reported Wilkes-Barre and Danville discharges on the basis of drainage areas. Corresponding river stages at the site are estimated by use of the stage-discharge curve presented as Figure 2.4-8. The development of this curve is discussed in Subsection 2.4.3.5.

2.4.2.1.2 Tropical Storm Agnes

The passage of Tropical Storm Agnes through Pennsylvania on June 22 and 23, 1972 resulted in record flood levels in the Susquehanna River Basin. Flood crests exceeded the previous record

flood level of 1936 at Wilkes-Barre by 7.5 feet. At Danville, a local maximum gage level resulting from a 1904 ice jam was exceeded by 1.6 feet. Peak discharge at Wilkes-Barre was an estimated 345,000 cfs or a unit discharge of 34.6 cubic feet per second per square mile (cfsm). Accumulated runoff for the drainage area above Wilkes-Barre for the period of 0000 hours, June 21, 1972 through 2200 hours, June 27, 1972 totaled 4.32 inches (Ref. 2.4-16 through 2.4-18).

High water marks were recorded at two locations in the vicinity of the site (river mile 165.6). A downstream flood mark of 506.0 ft msl was noted at Beach Haven (river mile 161.5). An upstream flood mark of 525.6 ft msl was noted at Shickshinny (river mile 170.1) (Ref. 2.4-16). A profile of the June 1972 flood in the vicinity of the site is provided as Figure 2.4-9. This profile shows the site to be at least 150 feet above the high water level. The river gaging station maintained at the site biological laboratory recorded a flood crest elevation of 516.6 ft. msl on June 23, 1972.

2.4.2.1.3 Flood Resulting from Ice Jams

The only recorded instance of ice related-flooding on the Susquehanna River in the vicinity of the Susquehanna SES occurred near Danville on March 9, 1904. This jam resulted in a local maximum flood level of 462.0 ft msl which was not exceeded until the June 1972 flood (Ref. 2.4-19 and 2.4-20). Additional discussion of ice jams is found in Subsections 2.4.7 and 2.4.11.2.

2.4.2.2 Flood Design Considerations

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2.4.2.3 Effects of Local Intense Precipitation

The effects of local intense precipitation were investigated to ensure that flooding at the plant site, if any, produced by a probable maximum precipitation (PMP) would not endanger the integrity of the safety-related facilities and that adequate drainage systems are provided for the roofs of all safety related buildings. Drainage systems for the roofs are designed so that hydrostatic loadings on the roofs resulting from a local PMP are within the design limit.

The all-season 24-hr PMP was derived using the procedures suggested by the National Weather Service (formerly US Weather Bureau) (Ref. 2.4-21). The maximum 6-hr precipitation was disaggregated into one-half hour increments in accordance with a time distribution proposed by the US Army Corps of Engineers (Ref 2.4-22) and is presented in Table 2.4-5. For storms less than one-half hour, the rainfall increments were determined using the ratios suggested by the National Weather Service (Ref. 2.4-23), and are shown in Table 2.4-6.

The grading and natural topography of the plant site area are such that storm runoff is directed away from safety related buildings by a system of culverts, surface drainage channels, and underground storm drains. In the evaluation of the effects of the PMP relative to the flooding of the safety-related facilities, all the culverts and underground storm drains, except the culverts in the emergency spillway for the spray pond were assumed to be blocked by debris or ice accumulation. The runoff from the PMP was assumed to occur only as surface flows, traversing the plant site in drainage channels or over low sections in the roads (Figure 2.4-10).

Drainage areas are based from the “Existing Stormwater Report for PPL SSES in Salem Township, Luzerne County, Pennsylvania,” (Ref 2.4-100).
The peak flood discharges resulting from the PMP were computed at a number of locations and for the roofs of the safety related structures using the "rational" formula:

$$Q = CIA \qquad \text{(Equation 2.4-1)}$$

where:

Q is the peak rate of runoff in cubic feet per second at the section of interest

C is the runoff coefficient depending on the characteristics of the drainage area: conservatively assumed to be 1.0 for all impervious surfaces such as a paved area or roof surface, and 0.9 for all other types of surfaces for PMP conditions

I is the rainfall intensity in inches per hour for a storm duration equal to the time of concentration for the location of interest

A is the drainage area, in acres, contributing to the flow at the point of interest

The points of interest and their corresponding flow cross-sections are shown on Figures 2.4-11, 2.4-12 and 2.4-13.

The flow depth in a drainage channel was calculated using Manning's equation:

$$AR^{2/3} = \frac{Qn}{1.49\ (S)} 1/2 \qquad \text{Equation (2.4-2)}$$

where:

A is the cross-sectional area of the flow in square feet, at the check location perpendicular to the flow direction

R is the hydraulic radius at the check location in feet

Q is the peak flood discharge in cubic feet per second

n is the Manning's roughness coefficient

S is the slope of the energy gradient in feet per foot.

The peak discharges resulting from the local PMP and the corresponding flow depths at the check sections are shown on Figures 2.4-11 through 2.4-13.

Pressure resisting doors are provided to prevent water from reaching safety-related equipment should any water build up or ponding occur adjacent to the power block. These doors are 1-3/4 inch thick, flush type, hollow steel doors with 12 ga. pressed steel frames and 12 and 14 ga. steel face sheets. Hinges are heavy duty, stainless steel, locksets are heavy duty mortise type and strikes are wrought box type. Gaskets are provided between doors and bearing (sealing) surfaces of frames and thresholds. Door and frame assemblies are designed to withstand various combinations of seating and unseating pressures depending on their locations in the plant. Prototype assemblies are tested to insure their conformance to the specified performance criteria. The performance criteria includes testing the seating and unseating pressure at the specified temperature as well as testing the leakage rate of each prototype. The results of these tests are documented. The flood flow is from the 61-acre area north of the spray pond and would be diverted away by a periphery channel. The channel is designed to accommodate the peak discharge of 1,259 cfs resulting from the local PMP as shown on Figure 2.4-13. Therefore, the possibility of flooding any of the safety-related facilities due to PMP is precluded.

The peak rates of runoff from the roofs of these buildings resulting from the all-season PMP are presented in Table 2.4-7.

Direct rainfall on the roofs of the reactor building, diesel generator buildings, and control structure is drained by a system of roof drains, supplemented by a series of scuppers and/or openings in the parapet walls forming the perimeters of the roofs of the buildings. The dimensions of the scuppers and parapet openings required are shown in Table 2.4-7.

In the evaluation of the effects of the local PMP, the roof drains were assumed to be blocked by debris or ice accumulation and only the scuppers or parapet openings remain functional. The rating curve for each parapet opening or unsubmerged scupper was derived using the equation:

$$Q = CLH^{1.5} \qquad \text{(Equation 2.4-3)}$$

where:

Q is the discharge capacity of the parapet opening or unsubmerged scupper in cubic feet per second

C is the discharge coefficient assumed to be 2.5 for a broad-crested weir condition

L is the effective length in feet of the parapet opening or scupper, taking flow contraction at entrance into consideration

H is the head in feet of water above the invert of the parapet opening or scupper

To compute the flow capacity of a submerged scupper, the flow equation for an orifice was used:

$$Q = CA\,(2gh)^{1/2} \qquad \text{(Equation 2.4-4)}$$

where:

Q is the discharge through the submerged scupper

C is the discharge coefficient conservatively assumed to be 0.45 because of the limited submergence conditions encountered

A is the cross-sectional area of the scupper inlet in square feet

g is the gravitational acceleration equal to 32.2 ft/sec/sec

h is the upstream head in feet of water measured to the centerline of the flow through the scupper

Ice accumulation could affect the site drainage by blocking drains and culverts. This effect has been considered in the overall evaluation of the effect of the local PMP described in the section.

2.4.3 PROBABLE MAXIMUM FLOOD (PMF) ON STREAMS AND RIVERS

The conditions producing the PMF are defined by the Corps of Engineers as the "hypothetical flood characteristics (peak discharge, volume, and hydrograph shape) that are considered to be the most severe reasonably possible at a particular location, based on a relatively comprehensive hydrometeorological analysis of critical runoff-producing precipitation (and snowmelt, if pertinent) and hydrologic factors favorable for maximum flood runoff" (Ref. 2.4-25). The PMF for the Susquehanna SES was derived for the only water system, except local runoff that could affect site flooding, the Susquehanna River. A maximum PMF water elevation on the Susquehanna River with coincident wind-generated waves of 548.0 ft msl was calculated in the site vicinity, which is over 120 feet below site grade elevation of 670 ft msl. There are no other adjacent streams that would have an impact on plant flooding.

The guidelines provided in Appendix A of Regulatory Guide 1.59 were followed throughout the analyses. Because the Susquehanna SES is a flood-dry site, conservative assumptions and baseline conditions were adopted to maximize the PMF water elevations.

2.4.3.1 Probable Maximum Precipitation

To determine the PMF for this study, the Probable Maximum Precipitation (PMP) storm location, magnitude and temporal distribution were taken directly from Corps data (Ref. 2.4-26).

The Corps had previously computed the Standard Project Flood (SPF) at Wilkes-Barre (Ref. 2.4-27). Both the storm pattern used on the basin (Ref. 2.4-26) and the magnitude and distribution of precipitation (Ref. 2.4-28) were derived by the Corps. The storm pattern thus obtained was laid over a map of the basin, the sub-basin outlines were drawn on the map and, by inspection, an average value of total rainfall on each sub-basin was estimated from the storm pattern isohyets. This is shown on Figure 2.4-14. The 12 six-hour time segments into which the 72-hour PMP storm was divided (Ref. 2.4-26) were converted into 18 four-hour time segments. This division allowed direct use of the available unit hydrographs previously derived by the Corps of Engineers (Ref. 2.4-29) as discussed in Subsection 2.4.3.3.

2.4.3.2 Precipitation Losses

In order to determine the PMP rainfall excess for the Susquehanna River drainage basin, an initial loss of 1.0 inch followed by an infiltration loss rate of 0.05 inches per hour were adopted (Ref. 2.4-30 and 2.4-31). These precipitation losses are consistent with values reported in the Susquehanna River Basin Study (Ref. 2.4-32). Since the maximum PMF water elevation computed in Subsection 2.4.3.6 is over 120 feet below the site, precipitation losses are not a critical factor in the derivation of the PMF. The assumed values represent reasonable conservative basin conditions appropriate for use with the PMP.

2.4.3.3 Runoff and Stream Course Models

In a previous study (Ref. 2.4-29), the Corps computed the SPF from actual storm data, synthesized four-hour unit hydrographs for 68 sub-basins and determined routing coefficients from observed flood hydrograph movement using the Coefficient Method of Routing (Ref. 2.4-28) for some 105 reaches in the Susquehanna River Basin above Wilkes-Barre, Pennsylvania. A tabulation of the drainage areas and unit hydrograph characteristics for each sub-basin is shown on Table 2.4-8. These sub-basins are delineated and numbered on Figure 2.4-12. Table 2.4-9 provides the flood routing coefficients used in the basin runoff model (Ref. 2.4-32).

Since the previously derived PMP was divided into 18 four-hour periods, this division allowed direct use of the available unit hydrographs previously derived (Ref. 2.4-29). These hydrographs were combined and routed in accordance with the procedures developed in Reference 2.4-29. The site is over 120 feet above the PMF water level as determined in Subsection 2.4.3.6. Because of this substantial margin of safety against river flooding, no adjustments of the unit hydrographs for non-linearity were made.

2.4.3.4 Probable Maximum Flood Flow

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2.4.3.5 Water Level Determination

Backwater computations have been carried out for the reach of the Susquehanna River in the vicinity of the site shown in Figure 2.4-16. The developed stage discharge curves for river cross-section 2 (Berwick Bridge) and upstream of river cross-section 7 (site) are shown on Figures 2.4-17 and 2.4-18 respectively. The procedures for developing the stage discharge relationships are described in the following Subsection.

2.4.3.5.1 Hydraulic Characteristics of Channel and Overbank

The excellent discharge records of the flood profile from the March 20, 1936 flood (Ref. 2.4-34), combined with the channel cross-section data (Ref. 2.4-35) and the USGS topographic maps (Berwick Quadrangle), allowed computation of the hydraulic characteristics of the channel and of the overbank area in the vicinity of the site. The profile data (Ref. 2.4-34) are shown on Figure 2.4-18. The river mile locations of the selected eight Sections (Ref. 2.4-35) are also shown on Figure 2.4-18 and on Table 2.4-10. The configurations of river cross-sections 1 through 8 are shown on Figure 2.4-19. The cross-section of the river at the site is shown on Figure 2.4-20. Since the 1937 survey (Ref. 2.4-35) there has been no construction (roads, bridges, excavation, etc.) which would have appreciably modified the cross-sections. Therefore, it is assumed that these sections are still representative. The 1936 flood level elevations of the Sections were taken from the plotted profile Figure 2.4-18. Elevations of the river bank edge, that is, the level which separates channel flow characteristics and overbank flow characteristics, were estimated by site inspection and from the data on Figures 2.4-19 and 2.4-20. Levels of about 12 to 16 feet above the main river channel bottom were selected on this basis. The bank elevations of the Sections are shown on the profile Figure 2.4-18 (Ref. 2.4-31).
The Corps of Engineers Method I backwater curve calculations (Ref. 2.4-36) were used to estimate Manning "n" values between pairs of river cross-sections using the 1936 flood profile data. This method is a trial and error computation procedure applicable to situations in which channel and overbank reach lengths may be assumed as equal (Ref. 2.4-31).

Discharge values of the sections were estimated by linear interpolation along the river between the maximum discharge values of 232,000 cfs and 250,000 cfs at Wilkes-Barre and Danville, respectively (Ref. 2.4-31).

The values for "n" thus estimated are shown on Table 2.4-10. Because of rather small contribution of the overbank flow to the total flow it was impractical to determine separately the "n" values for the overbank and the channel. Therefore, it was assumed that the overbank "n" was consistently 0.10 and the channel "n" values were then computed. Even doubling an assumed overbank "n" value to 0.20 did not result in significantly different channel "n" values. Table 2.4-10 shows that the channel "n" values range from 0.027 to 0.052. This range of values might be attributed to changes in the channel configuration. References to Figure 2.4-19 show river cross-sections 1, 2, 7 and 8 to have relatively flat river bank slopes. The channel between river cross-sections 3 through 6 has steep bank slopes (Ref. 2.4-31).

Backwater curve calculations using the Corps of Engineers Method III (Ref. 2.4-36), a graphic solution technique described below, reproduced the 1936 flood almost exactly when the computer

"n" values of Table 2.4-10 were used. Thus, these "n" values were adopted. The head loss of the 1936 flood passing through the Berwick Bridge, as computed by the Yarnell Method (Ref. 2.4-37), was only 0.05 ft. Thus, all head losses due to this channel construction were subsequently ignored (Ref. 2.4-31).

2.4.3.5.2 Backwater Curve Calculations

Upon initial inspection, it was apparent that the Susquehanna River would not reach a critical depth level, irrespective of discharge, at any point downstream of the site. Therefore, it was necessary to select some arbitrary point downstream from which to begin the backwater calculations. The point selected was river cross-section 1, downstream from the Berwick Highway Bridge. Two different sets of backwater curves were calculated; one with the Berwick Bridge completely washed out and the other with the bridge intact. Both profiles are shown on Figure 2.4-16. The procedure used was that suggested in Reference 2.4-36. That is, at several assigned values of discharge, including the PMF, backwater curve calculations were begun at river cross-section 1 using an assumed elevation and carried upstream to river cross-section 8. The error resulting from an incorrectly assumed trial starting elevation will tend to diminish as computations progress upstream. Additional sets of computations were made beginning at the same downstream location, but at a different trial starting elevation. If the starting location is sufficiently far downstream, and if the assumed trial starting elevations are reasonably near to the true elevation, the corresponding backwater curves will merge into one before the computations have progressed to the reach for which the back water curve is desired. This procedure was followed until the backwater curves conveyed, indicating that the derived water surface elevations are reasonably correct (Ref. 2.4-31).

The backwater curve calculations were performed in accordance with standard procedures (Ref. 2.4-36). The increase in wetted perimeter at river cross-section 2 was considered because of the bridge piers remaining intact. The backwater curve for PMF is shown on Figure 2.4-17 (Ref. 2.4-31).

It was conservatively assumed that the Berwick Highway Bridge (a truss) would remain intact and that the truss would be completely covered with debris. Thus, there would be no appreciable weir overflow over the bridge deck. This was considered to be the worst possible condition in creating high backwater at the site. On this basis, the water would pass as channel flow and as orifice flow (Ref. 2.4-31).

The Pennsylvania Department of Transportation has replaced the Berwick Bridge with a new structure at a higher level. The new structure results in lower head losses since the orifice flow situation is eliminated. Backwater calculations based on the intact old bridge structure become even more conservative since the bridge has been replaced.

The elevation of the lower edge of the existing bridge deck on the right bank of the river is 531.5 ft, and on the left bank is 509.0 ft. The length of the bridge between abutments is 1517 ft. The discharge at the bridge Section at a water surface elevation of 509.0 ft would be about 530,000 cfs. Any water passing under the bridge at an elevation above 509.0 ft would be partially open channel (some channel and some overbank) and partially orifice discharge. The capacity of the open channel portion was estimated from the stage-discharge relationships obtained from the curve developed with the bridge assumed to be washed out. The orifice discharge was calculated using an orifice coefficient of 0.7 from Reference 2.4-36. Because of the sloping of the bridge deck, the

maximum head at each discharge would be at the lowest point or the east end. The orifice discharge was added to the open channel discharge and the stage-discharge curve derived at river cross-section 2, the bridge, is shown on Figure 2.4-17 (Ref. 2.4-31).

At the PMF discharge of 1,100,000 cfs, the existence of the intact bridge raises the water surface elevation about 6 feet at the bridge. Backwater curves were generated upstream from Section 2 with the new elevations. The results for the PMF are shown on the profile Figure 2.4-18 (Ref. 2.4-31).

The profiles of the PMF are shown on Figure 2.4-18 for the two conditions of bridge washed out and bridge intact. The site is at mile 165.6; the elevations reached by the PMF are 544.8 ft for the bridge considered washed out, and 545.7 ft for the bridge intact (Ref. 2.4-31).

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2.4.3.6 Coincident Wind Wave Activity

The wind-generated significant waves on the probable maximum water surface elevation would be about 2.3 feet estimated from a crossriver 5,000 ft fetch with an average depth of 45 ft and a wind velocity of 45 mph along the fetch shown in Figure 2.4-21 (Ref. 2.4-31). The Susquehanna SES is located far above any potential flood level. The design basis for river flooding includes a PMF stillwater elevation of 545.7 ft, plus 2.3 ft for setup and wave runup effects for a total elevation of 548.0 ft msl (Ref. 2.4-31).

Consideration is also given to coincident wind wave activity with floods of a more frequent nature. Because of the plant's great safety margin against river flooding, an extremely conservative procedure was adopted to estimate water levels under the combined occurrence of frequent flooding, with coincident probable maximum gradient winds. The analysis procedure considers a 100-year flood level with the maximum supportable wave height, i.e., the breaking wave height and its associated wave runup.

Federal Insurance Administration, Type 15, Flood Insurance Study has been performed for the Susquehanna River Basin Commission (SRBC) in the vicinity of plant. Results of the study have been provided by the SRBC. These results are preliminary in nature until accepted by both the concerned municipalities and the Federal Insurance Administration. However, they represent the most up-to-date 100-year flood profile estimates.

Figure 2.4-9 shows the profile of the 100-year flood in the vicinity of the Susquehanna SES as estimated by the Flood Insurance Study. The 100-year flood level at the site is 513.6 ft. At this level, the water depth to the river bed is 33.6 ft. The maximum wave height which is physically possible is the breaking wave height. This wave height is 26.2 ft. based on a 33.6 ft. water depth (Ref. 2.4-42). The bank slope is estimated to be 1:30 (V:H). Wave runup associated with the breaking wave height on this slope is estimated to be 5.2 ft. (Ref. 2.4-42). The maximum water level at the site, under the effects of a 100-year flood with coincident wind wave activity resulting from the simultaneous occurrence of an extreme wind, is very conservatively estimated to be 539.8 ft. at the breaking wave height. This level is well below the plant grade elevation of 670 ft. The

simplified, albeit, very conservative approach is, therefore, justified. This water level is also below the PMF level of 548 ft.

Dynamic effects of wind waves are not considered in this section since there are no safety-related facilities in the Susquehanna Flood Plain.

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2.4.4 POTENTIAL DAM FAILURES SEISMICALLY INDUCED

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2.4.4.1 Dam Failure Permutations

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2.4.4.2 Unsteady Flow Analysis of Potential Dam Failures

2.4.4.2.1 Singular Dam Failures

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2.4.4.2.2 Multiple Dam Failures - Chemung River Basin

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2.4.4.2.3 Multiple Dam Failures - East Susquehanna River Basin

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2.4.4.2.4 Multiple Dam Failures - Lackawanna River Basin

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2.4.4.3 Water Level at Plant Site

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2.4.5 PROBABLE MAXIMUM SURGE AND SEICHE FLOODING

The Susquehanna River is the only major water body in the vicinity of the site. Consideration of seiche flooding potential is therefore not applicable in this case. The site is located about 165 miles upstream of the mouth of the Susquehanna River in Chesapeake Bay. Flooding through propagation of an open coast surge upstream to the site is also not applicable.

Wind waves and associated wave run-up acting in conjunction with a 100-year flood are discussed in Subsection 2.4.3.6. The very conservative analysis for these conditions results in a maximum wave height of 23.4 ft with an associated wave run-up of 9.4 ft. The maximum water level for the 100-year flood with coincident wind wave activity is 519.4 ft msl. This level is 28 ft below the PMF level and 150 ft below the plant grade. Based on the great margin of safety against flooding obtained through this simple but very conservative analysis, a detailed wave analysis including probable maximum hurricane winds is not considered necessary.

Consideration of flooding mechanisms in the spray pond are discussed in Subsection 2.4.8. The very short fetch lengths involved prevent the development of any significant wave activity in the spray pond.

2.4.6 PROBABLE MAXIMUM TSUNAMI FLOODING

Not applicable to the Susquehanna Site.

2.4.7 ICE EFFECTS

Portions of the Susquehanna River are subject to freezing during the months of November through April. Information on river freezing at Harrisburg for the period of 1870-1955 has been compiled by the Weather Bureau Airport Station at Harrisburg, Pennsylvania (Ref. 2.4-39). This information is provided in Table 2.4-11. The Susquehanna River remained open all winter during 22 of the 86 years of record. During the remaining years, 98 instances of freeze-over were noted. Thirty six of these freeze-overs were for periods lasting 14 consecutive days or less. There have been only 9 occasions when the river has remained frozen over for more than 60 consecutive days.

Flooding due to ice jams or "gorges" caused by ice break-up and subsequent re-freezing is sometimes a problem in the late winter months. Jamming may occur at locations where floating ice is retained and builds-up, such as at bridges, dams, narrow bends in the river, islands and reaches of the river with shallow rocky bottoms. Neither the Baltimore District Corps of Engineers nor the National Weather Service Mid-Atlantic River Forecast Center at Harrisburg currently have programs for systematically recording details of ice jam occurrences. Ice jams receive mention only when they cause flooding conditions.

Instances of ice jam-related flooding on the Susquehanna River have been recorded at Danville and at Wilkes-Barre. The dates of these occurrences and the resulting stages are provided in Table 2.4-12. Three such events have occurred at Danville over a 58 year period of record, or about once every 19 years. Seven ice-related flooding events have occurred at Wilkes-Barre over a 68 year period of record, or about once every 10 years. (Ref. 2.4-40.) Information on ice jams in the immediate vicinity of the site is not available. However, the regional data suggest an average recurrence on the order of 10 to 19 years. The most severe ice-related flooding occurred at Danville in 1904. Gage heights of 26.2 ft on January 25, 24.6 ft on February 10, and 30.7 ft on March 9 were recorded (Ref. 2.4-19, 2.4-20, and 2.4-40). All levels exceeded the Danville Flood Stage of 20 ft. The flood stage of March 9, 1904 remained the maximum gage height of record up until the flooding resulting from Tropical Storm Agnes in June 1972 (Ref. 2.4-41).

Remaining incidents of ice jam-related flooding have occurred substantially downstream of the Susquehanna SES. Probably the most damaging of these ice jam floods occurred in February 1963 at Duncannon, near the confluence of the Juniata and Susquehanna Rivers. Ice layers broke up and fused upstream of Duncannon causing a severe jam resulting in flood levels higher than the 1936 flood. Damage was reported from the mouth of the Juniata to Newport, 12 miles upstream. The jam was so severe that it diverted the flow of the Juniata River to the Susquehanna River upstream of the mouth.

The above discussion, along with reference to Tables 2.4-4 and 2.4-12, show that ice-related flooding in the general vicinity of the Susquehanna SES has resulted in flood stages comparable to precipitation-related flood stages. These stages are, however, appreciably below the estimated PMF water level, which is itself over 120 feet below the plant grade. Ice jam flooding is, therefore, no threat to any safety-related facilities.

Ice jam flooding or low water as a result of upstream jams on the river do not affect the availability of essential cooling water supplies. Any potential damage to the river intake structure from ice jamming in no way effects the safety of the plant. The plant can be safely shut down without the use of makeup water from the river. Design for potential icing conditions in the spray pond is discussed in Subsection 9.2.7.

2.4.8 COOLING WATER CANALS AND RESERVOIRS

2.4.8.1 General

The purpose of the spray pond system is to satisfy the ultimate heat sink criteria outlined in Regulatory Guide 1.27 (Rev. 2, 1/76). In this section, only the hydrologic and hydraulic design aspects of the spray pond system are considered.

2.4.8.2 General Description of the Spray Pond System

The spray pond system is located northwest of the cooling towers and the reactor-turbine building. The pond is freeform in shape. Embankments and ditches are provided to direct surface water runoff in a controlled manner. The bottom elevation of the pond is 668 ft msl. Under normal operating conditions, the water surface elevation in the pond is at elevation 679 ft msl, and is controlled by an overflow weir in the ESSW pumphouse with crest at elevation 678.5 ft msl. The pond and ESSW Pumphouse are described in more detail in Subsection 3.8.4.1 and shown on Dwgs. M-284, Sh. 1, C-64, Sh. 1, C-65, Sh. 1, C-66, Sh. 1, and C-67, Sh. 1. The water level is maintained by (a) rainfall (primary), (b) separate makeup pumped through a pipe adjacent to the ESSW pumphouse (secondary), or (c) cooling tower blowdown (backup as required). An uncontrolled spillway is located at the east end of the spray pond and has a bottom width of 30 ft and side slopes of 10 to 1. Invert elevation at the entrance is 680.5 ft and the longitudinal slope of the exit channel is 0.5 percent with side slopes of 2 to 1.

A railroad embankment is located some 159 feet downstream from the outlet of the spray pond. At this location, the channel is replaced by four 6 ft. x 3 ft. concrete box culverts with security gratings installed at both ends. The channel has a drop some 133 feet downstream of the culverts before merging with the drainage ditch located just to the north of the spray pond, and discharges into a natural waterway leading to the Susquehanna River. If this natural water course could become blocked, safety-related structures will not be affected. The drop in the spillway channel is designed to prevent any possible backwater effect which could develop from flows in the drainage ditch and which could affect the hydraulic performance of the spillway channel. The channel upstream of the box culverts is concrete-lined to a depth of 3.5 feet to prevent any possible erosion that could cause a blockage in the culverts. Downstream from the culvert, the channel is grass-lined. A longitudinal section of the spillway channel is shown in Figure 2.4-27.

2.4.8.3 Design Bases for the Capacity of the Spray Pond

The design bases for the capacity of the spray pond are addressed in Subsection 9.2.7.

2.4.8.4 Hydrologic Design Bases for the Spray Pond System

The spray pond system is designed to remain functional under the most adverse hydrometeorological conditions such as the probable maximum storm (Subsection 2.4.2.3) and

tornado, or the hydrodynamic loadings resulting from waves generated by the safe shutdown or operating basis earthquake (Section 3.7).

2.4.8.4.1 Design Basis Flood Level (DBFL)

The Design Basis Flood Level (DBFL) for the spray pond was determined in accordance with Regulatory Guide 1.59 (Rev. 1 4/76) by superimposing the effects of coincident wind-generated wave activity on the various flood levels; namely:

1) A sustained 40 mph wind on the probable maximum flood (PMF) level
2) The worst wind of record at Avoca on the standard project flood (SPF) level
3) A probable maximum gradient wind on a 10-year flood level.

The probable maximum flood was derived from the probable maximum storm (Subsection 2.4.2.3) assuming no rainfall losses on the land portion of the drainage area. Since the longest distance from the drainage divide to the edge of the pond is only about 400 ft with a slope of 3 to 1, no time lag between rainfall and runoff to the pond was assumed. The inflow probable-maximum-flood hydrograph was derived by assuming that the entire probable maximum precipitation (PMP) on the 18.6-acre drainage area runs off instantly. The resulting hydrograph is shown on Figure 2.4-24.

The probable maximum flood was routed through the spray pond under the following assumptions:

a) A normal operating water level of 679.0 ft is maintained in the spray pond.

b) Blowdown water from the cooling towers may be routed through the spray pond at a constant rate of 10,000 gpm (22.3 cfs).

c) The blowdown outlet conduit at the ESSW pumphouse, which serves as the exit for the excess water in the spray pond, has a maximum capacity of about 41 cfs.

Figure 2.4-25 shows the rating curve of the spillway channel, assuming no blockage of the culverts by debris or ice accumulation. The spillway rating curve was derived by assuming a discharge, computing the corresponding critical depth at the downstream control point, calculating a water surface profile upstream to the spray pond, adding an entrance velocity head and associated losses to obtain the proper spray pond water surface. The entrance loss for the channel was assumed to be 0.5 of the velocity head and that for the culverts was estimated using data suggested by the Bureau of Public Roads (Ref. 2.4-97). Manning's equation:

$$Q=\frac{1.486}{n}AR^{2/3}S^{1/2} \qquad \text{(Equation 2.4.8-1)}$$

was used to determine the channel resistance. In this equation,

Q is the discharge, in cfs

n is Manning's roughness coefficient assumed to be as follows:

0.06 for the grassed channel downstream from the culverts and upstream from the confluence with the drainage ditch. 0.015 for the concrete-lined spillway channel upstream from the culvert inlet. 0.013 for the concrete box culvert.

A is the cross-sectional area of the flow in sq ft.

R is the hydraulic radius in ft.

S is the slope of the energy gradeline.

The blowdown-water outlet in the ESSW pumphouse is a submerged orifice, 2 ft in diameter, with the centerline at elevation 674.75 ft and is located 8 ft downstream of the overflow weir. The rating curve of the blowdown water outlet shown on Figure 2.4-25 was derived using the equation:

$$Q = 0.6A\,(2gh)^{1/2} \qquad \text{(Equation 2.4.8-2)}$$

where

Q is the discharge in cfs.

A is the cross-sectional area of the submerged orifice in sq ft.

g is the gravitational acceleration equal to 32.2 ft/sec^2.

h is the upstream head in feet of water measured to the centerline of the submerged orifice.

The elevation-area-storage capacity curves of the spray pond are shown on Figure 2.4-26.

The results of the flood routing studies are shown in Table 2.4-13. The maximum water level under the PMF condition was found to be at elevation 682.3 ft.

The maximum water levels in the spray pond for the Standard Project Flood (SPF) defined as one-half of the PMF, and the 10-year flood conditions, were also calculated and found to be 681.8 ft and 679.6 ft msl, respectively (Table 2.4-13). In deriving the maximum water level under the SPF condition, a coincident earthquake was assumed (Regulatory Guide 1.59, Rev. 1, 4/76) causing failure of the blowdown discharge conduit downstream from the ESSW pumphouse. The 10-year flood level was derived assuming that the entire 10-year flood runoff from the spray pond watershed would be stored in the pond. In deriving the 10-year flood volume, the 24-hour 10-year rainfall suggested by the US Weather Bureau (Ref. 2.4-23) was used assuming that 50 percent of the rainfall on the level portion of the spray pond drainage area would run off.

The worst winds of record at Avoca were found to be 65 mph and the probable maximum gradient wind for the site area was estimated to be 80 mph, the derivation of which is presented in Section 2.3.

The effects of the coincident wind-generated wave activity were estimated in accordance with methods suggested by the US Army Corps of Engineers (Ref. 2.4-42). Wind setup in the spray pond was found to be negligible. The results of these computations are presented in Table 2.4-14.

Assuming a standing wave condition at the ESSW pumphouse, the DBFL resulting from a 1 percent wave will be at 684.8 ft. This water level does not represent a threat to any safety-related facility because all the safety-related equipment is located at elevation 685.5 ft. or higher and is protected from splash effects by the walls of the pumphouse and slab at top elevation 685.5 ft. (refer to Dwg. M-274, Sh. 1). The run-up elevation on the side of the spray pond from a 1 percent wave will be 684.6 ft. The side of the spray pond is protected by a concrete lining up to elevation 685.5 ft. This protection will preclude any erosion of the bank due to wind wave action.

Wave forces on the ESSW pumphouse and the pipe supports were also estimated for the different flood water levels under the assumed coincident wind wave activities using methods suggested by the US Army Corps of Engineers (Ref. 2.4-42). In estimating the wave forces on the ESSW pumphouse, the water level inside the pumphouse was assumed to be at the corresponding static water level in the spray pond. Table 2.4-15 presents the results of the wave force computations on the ESSW pumphouse and pipe supports. The force due to hydrostatic pressure is not included.

The peak outflow through the spillway channel during the design flood condition (Case 2, Table 2.4-13) is estimated to be 150 cfs. The calculated water surface profile along the spillway channel for the peak outflow is shown in Figure 2.4-27. It was derived using the standard-step method (Ref. 2.4-24), with the assumption that critical depth occurs at the drop located upstream from the confluence between the spillway channel and the drainage ditch north of the spray pond.

A minimum of 3 ft of freeboard is provided in the spillway channel. At most points along the chute, the actual freeboard is greater than 3 ft because it is governed by the elevation of the bottom of the chute relative to the adjacent ground surface. This amount of freeboard exceeds recommended practice by the US Bureau of Reclamation (Ref. 2.4-43) and the US Army Corps of Engineers (Ref. 2.4-44) for similar design conditions.

Consideration was also given to the possibility of having a wave propagated into the channel coincident with the occurrence of a probable maximum storm. The critical maximum breaking-wave height, coincident with a 40 mph wind, was found to be approximately 1 ft and would not affect the freeboard allowance stated previously.

2.4.8.4.2 Safe Shutdown and Operating Basis Earthquakes

The safe shutdown and operating basis earthquakes (SSE and OBE) for the project are presented in Section 3.7. In accordance with the design criteria set forth in Regulatory Guide 1.59 (Rev. 1, 4/76), the SSE and OBE were assumed to occur coincidentally with a 25-year flood and a standard project flood (one-half of a PMF), respectively. Since the spray pond discharge conduit is not designed to withstand an earthquake, it is conceivable that a portion of this conduit could collapse causing a blockage. It was, therefore, assumed that no water would pass through the discharge conduit following a design earthquake condition.

Assuming failure of the discharge conduit, the maximum water levels in the spray pond during a 25-year and a standard project flood (SPF) event were estimated to be at elevation 681.4 and 681.8 ft, respectively. The 25-year flood level was derived by routing the 25-year flood (peak flow 45 cfs) through the spray pond. The peak discharge through the spillway channel was estimated to be 65 cfs. In deriving the 25-year flood peak, the rainfall duration-frequency atlas published by the US Weather Bureau (Ref. 2.4-23) was used assuming that 50 percent of the rainfall on the land portion

of the spray pond drainage area would run off. The derivation of the SPF level was presented in Subsection 2.4.8.4.1.

At the ESSW pumphouse, the wave heights generated during the SSE and OBE under the stipulated hydrologic conditions were estimated to be 2.9 and 1.6 ft, respectively. These wave heights were computed using the equation developed by Biesel (Equation 2.4.8-3) for a piston type of wave generator (Ref. 2.4-45).

$$H = 2S\frac{2Sinh^2(2\pi d / L)}{(2\pi\, d / L) + (Sinh(2\pi d / L))Cosh(2\pi d / L)} \qquad \text{(Equation 2.4.8-3)}$$

where:

H	is the wave height generated in ft.

S	is the design displacement (amplitude) caused by the earthquake in ft.

d	is the initial depth of water in ft.
L	is the wave length in ft and is a function of the period of the design basis earthquake.

In this computation, the design displacement (amplitude) and period were derived from Figures 2.4-28 and 2.4-29 as 10.1 in. and 2.7 sec for the SSE, and 5.4 in. and 2.7 sec for the OBE, respectively.

Assuming a standing wave condition at the ESSW pumphouse, the maximum forces and moments about the base of the structure were estimated to be 75.6 kips and 1156 ft-kips during the SSE and 61.7 kips and 871.5 ft-kips during the OBE, respectively, using the methods suggested by the US Army Corps of Engineers (Ref. 2.4-42).

At the pipe supports near the center of the pond, it is possible that the earthquake-generated waves coming from the opposing sides of the spray pond could be in-phase. For this condition, the maximum wave heights during the SSE and OBE were conservatively estimated to be 5.5 ft and 3.2 ft, respectively. The wave forces and moments at the base of each support would be 1.1 kips and 13.3 ft-kips for the SSE and 0.5 kips and 5.2 ft-kips for the OBE conditions, respectively.

The resultant maximum hydrodynamic force acting on the pipe supports as a result of earthquake shaking was estimated using the equation (Ref. 2.4-45):

$$F = C_m \rho V a_x \qquad \text{(Equation 2.4.8-4)}$$

where:

F	is the maximum hydrodynamic force due to earthquake.

C_m is the virtual mass coefficient assumed to be 1.5 (Ref. 2.4-44).

ρ is the mass density of the water equal to 1.94 slugs/cu ft.

V is the volume of the submerged structure (displaced water) in cu ft.

a_x is the maximum horizontal acceleration due to earthquake in ft/sec/sec.

For the case of the ESSW pumphouse, the equation used is that suggested by Tennessee Valley Authority (Ref. 2.4-46) for a rigid structure with water fronting on one side:

$$F = 36.5H^2 \frac{a_x}{g} \qquad \text{(Equation 2.4.8-5)}$$

where:

F is the maximum hydrodynamic loading in lb/lf.
H is the depth of water fronting the pumphouse in ft.

a_x is the maximum horizontal bedrock acceleration due to earthquake in ft/sec/sec.

g is the gravitational acceleration equal to 32.2 ft/sec/sec.

For earthquakes with motion along the east-west axis, the ESSW pumphouse was analyzed as a rigid body. The maximum hydrodynamic loading exerted on the wing-walls adjacent to the embankment was estimated using Equation 2.4.8-4. In this case, the virtual mass coefficient (C_m) adopted was 0.32 as given by Sarpkaya (Ref. 2.4-47).

The maximum hydrodynamic loadings resulting from the design basis earthquakes are presented in Table 2.4-16. These loadings do not include those due to hydrostatic or earth pressures, or impact from the earthquake-generated waves originating from the sides of the pond.

Since the natural period of the water body in the spray pond is substantially larger than that of the design earthquakes, the formation of seiches in the spray pond due to earthquakes is not possible.

START HISTORICAL

2.4.9 CHANNEL DIVERSIONS

The drainage basin of the Susquehanna River upstream of the site lies within the physiographic provinces of the Appalachian Plateau, and the Appalachian Valley and Ridge. Within the Appalachian Plateau Province, the terrain is characterized by deeply eroded, steep-sided flat bottom valleys and flat to gently rolling plateaus. At Pittston near the mouth of the Lackawanna River, the Susquehanna River enters the Appalachian Valley and Ridge Province and flows through the Wyoming Valley which is lined by even crested ridges on both sides (Ref. 2.4-39). Near

Wilkes-Barre, the Susquehanna River flows through a broad, flat plain which is bounded by moderately steep mountains. In the general vicinity of the site, the terrain is steeply sloped on both banks with dense forests and wooded areas (Ref. 2.4-15). The Upper Susquehanna is thus characterized as possessing a stable stream course flowing through well defined ridge and valley topography. As such, this portion of the Susquehanna River is not subject to major meandering realignment and diversion by natural causes.

END HISTORICAL

2.4.10 FLOODING PROTECTION REQUIREMENTS

As discussed in Subsections 2.4.1.1 and 2.4.2.2, the safety-related structures and facilities are secure from flooding. Hence, flooding protection requirements are not necessary.

2.4.11 LOW WATER CONSIDERATIONS

2.4.11.1 Low Flow in Rivers and Streams

Security-Related Information Text Withheld Under 10 CFR 2.390

2.4.11.1.1 Low Flow Resulting from Hydrometeorological Events

The low flow and water level design bases consider the fact that the Susquehanna River is used as a source for non-essential water supplies only. Essential water supplies are provided for the Engineered Safeguards Service Water System from the spray pond located on the site. The statistically derived one day low flow with a 100-year recurrence interval is taken as a satisfactory definition of the low flow resulting from a 100-year drought. This value is taken to be the low flow design basis for operation.

For purposes of this study, the available flow data for the USGS Wilkes-Barre stream gage (station 01536500) were used. The drainage area above the Susquehanna SES is some 2.4 percent greater than the drainage area above the Wilkes-Barre gage. Use of the Wilkes-Barre flow data thus provides a conservative estimate the low flows at the Susquehanna SES Site. Frequency analysis of the Wilkes-Barre gage data for the years 1900-1967 yield a one day 100-year low flow of 520 cfs (Ref. 2.4-31). Recent log Pearson Type III frequency analysis, performed by the USGS using flow data for the years 1900-1972, resulted in a one day 100-year low flow of 520.7 cfs at Wilkes-Barre. Peak consumptive use for the Susquehanna SES as described in Subsection 2.4.11.4.2 amounts to 74.7 cfs. This usage represents less than 15 percent of the one day 100-year low flow. The Susquehanna River is thus an adequate source of non-safety related water during the 100-year drought. The stage-discharge relationship for the Susquehanna River in the vicinity of the site is provided in Figures 2.4-5 (0-3,000 cfs) and 2.4-6 (1,000-37,000 cfs). Stage levels were measured at the site by means of a gage installed for this particular purpose. Discharges corresponding to the measured stages were obtained by direct

interpolation between the mean daily discharges at Danville and Wilkes-Barre as reported by the USGS. As shown on the Figure 2.4-5, the stage discharge curve was extrapolated down to zero discharge by constructing the curve so as to intersect the zero discharge at an elevation of 480 ft msl which is the bottom of the stream channel at the site. There were no observations at interpolated discharge values of less than 1200 cfs (Ref. 2.4-30 and 2.4-31).

Based on the stage-discharge relationship of Figure 2.4-30, the stage elevation for the one day 100-year low flow at the Susquehanna SES site is 483.5 ft msl. This elevation is the low water level design basis for non safety-related water supplies. Essential water supplies are provided by the spray pond. The low water design basis for these supplies are discussed in Subsection 9.2.7.

2.4.11.1.2 Low Flow Resulting from Dam Failures

Security-Related Information Text Withheld Under 10 CFR 2.390

2.4.11.2 Low Water Resulting from Surges, Seiches, or Tsunami

The Susquehanna River serves only as the source of non-essential makeup water for the plant. Safety-related water supplies are drawn from the spray pond described in Subsection 9.2.7. Therefore, low water levels on the Susquehanna River resulting from the occurrence of probable maximum meteorological or geoseismic events, do not affect the ability of safety-related features to function adequately. Ice formation or possible ice jams on the Susquehanna River also affect only non-essential water supplies.

In order to demonstrate the adequacy of this non safety-related water supply even in extreme conditions, a conservative set down analysis was performed. The 100-year fastest mile wind of 80 mph as derived in Section 2.3 is taken as a steady wind blowing directly across the river away from the intake structure. This wind condition is assumed coincident with a 100-year low flow condition in the Susquehanna River. The resulting setdown amounts to 0.22 ft.

Even though makeup from the river is not required for any safety function, the intake is designed so that the top of the intake water passage is submerged 1 ft during the 100-year low flow condition (see Figure 2.4-52). The discharge diffusers are also below the river low flow level (see Figure 2.4-53).

Because of its location and small size, consideration of the effects of seiche and tsunami is not applicable to the spray pond. The very short effective fetch lengths which are available in the spray prevent the development of any significant wave and setdown. Thus, severe wind conditions as would result during a Probable Maximum Hurricane, would not create a low water condition in the spray pond which could affect the dependability of this safety-related water supply. Design features which assure the availability of safety-related water supplies are discussed in Subsection 9.2.7.

2.4.11.3 Historical Low Water

Flow data on historical low flows available in the records of the Wilkes-Barre gage (Station 01536500) is used in Subsection 2.4.11.1 to estimate the one day 100-year low flow at the Susquehanna SES site. The instantaneous minimum flow of record for this station is 528 cfs. This flow, as well as the lowest mean daily discharge of 532 cfs, occurred on September 27, 1964 (Ref. 2.4-6 and 2.4-49). Since statistical methods were not used to extrapolate flows and/or levels to provable minimum conditions, no further discussion is presented.

2.4.11.4 Future Controls

2.4.11.4.1 Legal Consumptive Use Restrictions

On September 30, 1976, an amendment to 18 CFR Part 803 (Susquehanna River Basin Commission, Part 803 - Review of Projects Consumptive Use of Water) was published in the Federal Register (Ref. 2.4-50). This amendment requires compensation in an amount equal to the projects total consumptive use when the stream flow at the intake equals or is anticipated to equal a specified low flow criterion. This criterion includes the 7-day 10-year low flow plus the projects total consumptive use and dedicated augmentation. Compensation may be provided by one or a combination of the following means:

1. Construction or acquisition of storage facilities
2. Purchase of available water supply storage in public or private facilities
3. Purchase of water to be released as required from a water purveyor
4. Releases from existing facilities owned and operated by the applicant
5. Other alternatives including reducing or halting consumptive water use and using alternative source unaffected by the compensation requirement

The provisions of this regulation apply to consumptive uses initiated since January 23, 1971. Consumptive uses beginning after this date must comply with the requirement within a time period to be determined by the Susquehanna River Basin Commission at the time of the permit application review. This compliance delay feature was included in the amendment with specific consideration of the Susquehanna SES project.

The low flow criterion value will be specified at the time of the permit application review. The Q7-10 flow, being a statistical quantity, will not vary substantially as additional years of base data are included in its computation. The Q7-10 value of 820 cfs for Wilkes-Barre (Ref. 2.4-51) can thus be taken as an approximation of the value which will be included in the low flow criterion. Dedicated augmentation and the plant consumptive use must also be added to determine low flow criterion.

2.4.11.4.2 Changes in Consumptive Use Upstream

Information on present and projected values of consumptive water use including inter-basin transfer is available from the New York State Department of Environmental Conservation (DEC) and the Pennsylvania DER (Ref. 2.4-13 and 2.4-52). Total consumptive use plus inter-basin transfer for the drainage area upstream of the Susquehanna SES for the period 1970-1974 was 81.4 cfs.

Projections for this same area for the period 2010-2020 set the consumptive use at 364.2 cfs or an increase of 282.8 cfs over the 1970's level. These projections included the originally-estimated average Susquehanna SES consumptive use of 62 cfs. A substantial increase in projected acreage of irrigated farm land in the Chemung and East Susquehanna River Basins account for about two-thirds of the estimated 2010-2020 consumptive water use.

The "Environmental Report - Operating License Stage" (ER-OL, Ref. 2.4-98) estimated a design maximum cooling tower evaporative loss of 28,700 gpm. The "Final Environmental Statement" (FES, Ref. 2.4-99) gave a conservative estimate of 600 gpm for all other consumptive uses, independent of power level. The maximum total consumptive use is the sum, 29,300 gpm or 65.3 cfs.

With power uprate the maximum cooling tower evaporative loss is expected to approach 32,900 gpm. Adding the 600 gpm FES estimate for other consumptive uses results in an uprated maximum total consumptive use of 33,500 gpm, or 74.7 cfs.

2.4.11.5 Plant Requirements

The safety-related cooling water is supplied by the ESW system and the RHRSW system. These systems are described in Subsections 9.2.5 and 9.2.6.

The minimum safety-related cooling water flow required is approximately 7,000 gpm for the ESW system and approximately 8000 gpm each for the Unit 1 and 2 RHRSW systems. Each of these systems has been designed with sufficient capacity and redundancy so that no single active or passive failure in either system will prevent the system from achieving its safety objective.

The cooling water for both the ESW and RHRSW systems is pumped from a concrete lined spray pond, the configuration of which is shown on Figure 2.4-2. This pond has a normal water surface area of approximately 8 acres and contains approximately 25×10^6 gal of water. The pond is designed to supply ESW and RHRSW for both units for 30 days after shutdown initiation without receiving makeup water. A complete discussion of pond design capability is given in Subsection 9.2.7.

The elevation of the bottom of the pond is 668 ft above msl and the minimum water level during normal operation is at elevation 678 ft-6 in. above msl. The ESSW pumphouse is located at the edge of the spray pond as shown on Figure 2.4-2. The top of the pumphouse foundation mat is at elevation 660 ft above msl. The minimum water level which will satisfy NPSH requirements at all flows of the vertical ESW and RHRSW pumps are at elevations 667 ft and 668 ft above msl, respectively. Therefore, sufficient NPSH is always available.

Details of the pumps are in Subsections 9.2.5 and 9.2.6.

2.4.11.6 Heat Sink Dependability Requirements

The water supply for normal shutdown is provided by:

a) The cooling tower pond, which supplies cooling water to the condensers and service water system by means of the circulating water pumps and service water pumps, respectively (see Subsection 10.4.5).

b) The spray pond, which supplies cooling water to the RHRSW system for dissipating reactor decay heat in the RHR heat exchangers.

The water supply for emergency shutdown is provided by the spray pond, which is the ultimate heat sink, and provides cooling water to both the ESW pumps and the RHRSW pumps as described in Subsections 9.2.5 and 9.2.6.

Subsection 9.2.7 describes the design bases for operation and normal or accident shutdown and cooldown under the following conditions:

a) The most severe natural and site-related accident phenomena

b) Reasonable combinations of less severe phenomena

c) Single failures of man-made structural components

The ultimate heat sink and the piping network located in it are designed to conform with Regulatory Guide 1.27 (Rev. 2, 1/76), which requires that the system operate both during and after the most severe natural phenomenon. Makeup water to both the cooling tower basin and the spray pond is provided by the Susquehanna River by pumps located in the river intake structure as described in Subsection 9.2.7.2.2.

Low level alarms are provided in the river intake structure, in the cooling tower basin, and in the RHRSW and ESW pump chambers.

The river water make-up pumps are tripped at the river low low level alarm setpoint of 485'-4". A low level alarm is provided and set at 485'-0" to alert the operator to a potential low river flow.

The volume of water to be contained within the pond was selected because various water losses (see Subsection 9.2.7) can be absorbed over a 30-day period without makeup. This absorption takes place when the pond is being used simultaneously to cool down one unit that has undergone a design basis accident and to safely shut down the second unit.

During the 30-day period, it is estimated that the decay heat generated for each core which has to be removed by the RHRSWS (Section 9.2.6) will be 2.5×10^{10} BTUs.

Table 9.2-3 lists all users of the ESWS (Subsection 9.2.5); Tables 2.4-18 and 9.2-4 relate users to time for two types of shutdown.

Tables 2.4-18 and 9.2-4 are based on one of the four aligned diesels being taken out of operation and placed on standby status after 24 hours of operation. The cooling load (Tables 2.4-18 and 9.2-4) is carried out to 30 days after the shutdown initiation; 30 days is the design life of the ultimate heat sink for operation without makeup water. The operation of all equipment listed at the cooling duty shown represents design conditions. Under actual operating conditions certain pieces of equipment may be shut down or operated under reduced loads.

The ultimate heat sink is used solely as a cooling water supply for the RHRSW and ESW systems. No interdependent water supply systems are used.

START HISTORICAL

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

The Susquehanna River is the only major surface water body in the vicinity of the station which could potentially be affected by the highly unlikely postulated spillage of liquid radwastes. The ability of the Susquehanna River to disperse, dilute as well as transport these wastes which reach it, is discussed with primary emphasis on the reach of the river extending from the station downstream to Danville, a channel distance of approximately 31 miles. The bulk of the potential dilution of such effluent releases occurs within this reach. In addition, standby and active uses of river water, as identified in Subsection 2.4.1.2.3, first occur within this reach.

Table 2.4-3 presents water users and uses within 50 miles downstream of the station. The location of these users is provided on Figure 2.4-7. Of principal importance to this discussion is the municipal water usage at Berwick (7 miles downstream), Bloomsburg (19 miles downstream) and Danville (31 miles downstream). Of these only Danville maintains active usage of the river water. Both Bloomsburg and Berwick maintain river intakes for use as standby water supplies. Five industrial users and one recreational usage have also been identified in this reach.

The following paragraphs provide a discussion of certain hydraulic characteristics of the Susquehanna River which are important to the dilution and transport of radionuclide releases. Accident conditions which result in such releases are postulated. Finally, estimates of the dilution of these wastes are provided.

END HISTORICAL

2.4.12.1 River Flow Characteristics

2.4.12.1.1 Flow Duration

The flow past a particular point represents a measure of the dilution potential of the stream. For the Susquehanna River the average flow past the station is about 13,600 cfs. A more complete description of the flow is provided in Figure 2.4-30. This figure shows the flow duration curves of daily discharge for the Susquehanna River gauging stations located at Wilkes-Barre and Danville. Flow duration characteristics at the stations can be interpolated from this figure. Such flow values are suitable for estimating the dilution of routine low level radioactive releases from the station. For

accidental releases, however, a more conservative approach must be taken. The determination of a suitable low flow value is described in the following sections.

START HISTORICAL

2.4.12.1.2 Extreme Low Flow

The minimum historic daily low flow rates were recorded on September 27, 1964 at both the Wilkes-Barre and Danville gages. The flows were 532 cfs and 558 cfs respectively (Ref. 2.4-49). The minimum historic daily low flow at the Susquehanna SES is estimated to be 538 cfs. This value is obtained by interpolation between the Wilkes-Barre and Danville values on the basis of drainage basin area. For comparison purposes, the 100-year low flow at the site is estimated to be 520 cfs (see Subsection 2.4.11.1.1).

No modification of this value was made for purposes of evaluation. Increased consumptive use of the Susquehanna River is projected to occur during the operational life of the station. Legislation described in Subsection 2.4.11.4.1, however, prohibits uncompensated consumptive water use when the flow rate approaches the 7 day, 10-year low flow value. The 7-day 10-year low flow value at the site is 820 cfs.

The major impact of new consumptive water uses initiated after regulation specified date of January 23, 1971 will essentially be limited to periods when the flow exceeds the 7-day, 10-year low flow value. Since no significant upstream changes in consumptive use occurred between the recorded historic low flows of 1964 and the controlling legislation date of 1971, the consumptive use situation which existed in 1964 is essentially preserved with respect to its influence on extreme low flows. Use of the unmodified historic low flows for purposes of discussion of dilutions of accidental liquid radwaste releases is considered to be reasonable.

2.4.12.1.3 Travel Times

Time-of-travel studies have been conducted by the USGS which include the reach of the Susquehanna River downstream of the station (Ref. 2.4-53). These dye studies were conducted between 1965 and 1967 during periods of low to medium flow. Data for the reach of the Susquehanna River between Shickshinny (about 4 miles upstream) and Danville (about 31 miles downstream) are presented in Figure 2.4-31.

Time-of-travel values for both the leading edge of the dye cloud, as well as for its peak concentration, are plotted. Discharge values are those for flow rates at Shickshinny, the dye injection point. For the historic low flow case with a flow of about 537 cfs at Shickshinny, Figure 2.4-31 indicates a range of travel times of about 135 through 155 hours. Proportioning these times on the basis of channel length, the travel times for the reach from the Susquehanna SES to Danville under historic low flow conditions range from 120 to 138 hours.

The flow velocity for this reach can be estimated through use of the peak concentration time-of-travel (138 hrs); the average flow velocity is 0.3 ft/sec. The 18 hour difference between the occurrence of the dye cloud leading edge and the peak concentration is a measure of the longitudinal dispersion which could contribute to the dilution of transient effluent releases.

END HISTORICAL

2.4.12.2 Accidental Releases

Because of the subsurface location of the radwaste tanks and processing facilities, as well as the procedures for handling radwastes at the Susquehanna SES, a direct release of liquid radioactive wastes via surface pathways to the Susquehanna River is not considered.

However, a highly improbable release of liquid radwastes into the Susquehanna River via a groundwater pathway has been postulated. A detailed discussion of the groundwater transport of the radionuclides is provided in Section 2.4.13.3. A brief description of the postulated accidental release along with the estimated radionuclide concentrations entering the river are provided in the following paragraphs.

The largest radionuclide concentrations in the radwaste system are found in the two 7,400-gallon Reactor Water Clean-Up (RWCU) Phase Separator Tanks. These tanks are located in the Radwaste Building and are entirely below grade. The postulated accident consists of a rupture of one of these tanks and a release of its contents into the groundwater system. The contaminated groundwater then moves downgradient toward the Susquehanna River. The location of the aquifer discharge into the river is shown on Dwg. FF62005, Sh. 1.

The aquifer rate of discharge to the river is estimated to be about 108 cubic feet/day per foot of aquifer width. Analysis performed under Section 2.4.13.3 indicated that the estimated radionuclide concentrations at the point of discharge into the river dropped off to below one percent of the peak centerline concentrations within a width of about 640 feet. Taking this value as the width of the contaminated flow, the inflow of contaminated groundwater to the river is calculated as 69,120 cubic feet/day (0.8 cfs).

Table 2.4-38 presents the estimated peak concentrations of radionuclides in the groundwater entering the Susquehanna River as a result of the postulated rupture of one of the RWCU Phase Separator tanks. As shown in the table the estimated peak concentrations for Sr-90 and Pu-239 at the point of entry into the river exceed the effluent concentration limits (ECL) for an unrestricted area as defined in 10 CFR 20 Appendix B. The remaining radionuclides analyzed in Section 2.3.13.3 have activity concentrations at the river that are at least an order of magnitude lower than their associated effluent concentration limits. When consideration is given to the downstream dilution effects discussed in the Subsection 2.4.12.3, in no case does the estimated peak concentration of any of the analyzed radionuclide exceed the effluent concentration limits given in 10 CFR 20 at the nearest downriver public potable water supply (Danville).

2.4.12.3 Effluent Dilution

The groundwater accident discussed above results in a release of contaminated water to the Susquehanna River over an extended period of time. For such a continuous release condition, lateral as opposed to longitudinal diffusion becomes the more important mixing mechanism. The maximum potential dilution occurs when cross-sectional homogeneity of concentration is achieved. For the case of the contaminated groundwater entering the river during the extreme low flow

occurrence, the maximum potential dilution ratio, is 1:650, which is the ratio of the groundwater flow (0.8 cfs) to the estimated 100-year low river flow at the site of 520 cfs (Section 2.4.12.1.2).

A relatively simple model was used to quantify the dilution downstream of the station. A steady state analytical streamtube model (Ref. 2.4-56) was employed for that purpose. The model is applicable to non-tidal rivers where the flow is assumed to be uniform and approximately steady. Such conditions occurred during the low flows of September 1964. Flow variation was within 10 percent of the minimums for 3 preceding days at Danville to 11 preceding days at Wilkes-Barre (Ref. 2.4-49). Similar flow behavior can be expected during future drought conditions severe enough to result in these low flow rates. The model is further limited to portions of the river removed from the influences of the discharge. For the groundwater release condition, the lack of momentum at the discharge location makes this model applicable for the entire reach downstream of the discharge.

Figure 2.4-33 shows a cross-section at the groundwater release point. The contaminated groundwater is seen to flow toward the river and flow into the river through the bank and bottom approximately to the mid-stream line. The contaminated groundwater inflow can be approximated as a line source perpendicular to the river flow. Equation 8 of Reference 2.4-56 provides the closed form solution for this type of release.

Additional conservative assumptions are made in the application of the model. The channel is taken to be straight, thereby removing any possible increase in cross stream diffusion at river bends. Effluent concentrations in the river are not reduced through any potential sorption of the radionuclides by suspended and bottom sediments. The analysis also conservatively neglects any additional dilution provided by tributary inflow at downstream locations along the Susquehanna River.

Flow characteristics at 32 cross-sections between the site and Danville were estimated from a HEC-2 computer simulation of the historic low flow condition as described in Subsection 2.4.12.1.2. These flow characteristics provide the basis for the determination of the diffusion factor D at each of these cross-sections. The longitudinal variation of D within this reach is relatively small. Therefore, the mean value of D is used to calculate the radioisotope concentration as a function of distance from the site.

The model results indicate that a fully mixed flow condition is approached within about 47 miles downstream of the station. Concentrations of the radionuclides at Berwick, Bloomsburg and Danville are reduced to 1.29, 1.02 and about 1.0 times the final fully mixed flow concentrations. The estimated concentrations at Danville of the three most important radionuclides are presented below relative to the limits presented in 10CFR20, Appendix B, Table 2

	Estimated (μ Ci/ml)	10 CFR 20 Effluent Concentrations (μ Ci/ml)
Sr-90	1.2×10^{-8}	5×10^{-7}
Cs-137	1.3×10^{-9}	1×10^{-6}
Pu-239	4.6×10^{-10}	2×10^{-8}

In summary, a simple analytical model was used together with conservative assumptions in order to roughly approximate the dilution of the contaminated groundwater entering the Susquehanna River. It was found that dilutions approaching the fully mixed flow limit of 1:650 were achieved at Danville where the first active municipal water usage downstream of the station is found. Concentrations of all radionuclides released in the postulated accident are substantially below their effluent concentration limits.

START HISTORICAL

2.4.13 GROUNDWATER

2.4.13.1 Description and Onsite Use

2.4.13.1.1 Regional Groundwater Conditions

From the point of view of groundwater, the region will be defined in this report to be the area within a 20-mile radius of the Susquehanna SES. Included in this area are the major portions of Luzerne and Columbia Counties, the northern portion of Schuylkill County, the northwestern corner of Carbon County, and the southeastern corner of Sullivan County.

The region lies in the Appalachian Highlands, which is made up of the Appalachian Plateau Province and the Valley and Ridge Province. The Valley and Ridge Province makes up almost the entire region, while the Appalachian Plateau occupies only the northernmost three percent of the area as shown on Figure 2.4-34.

In the region, the geologic formations of hydrologic significance are either consolidated formations of Paleozoic age or unconsolidated deposits laid down during the glacial age. In the Appalachian Plateau Province, the Paleozoic formations are nearly flat lying, while to the south in the Valley and Ridge Province, these formations have experienced pronounced folding. This folding, which occurred at the close of the Paleozoic Era, produced a number of northeast-southwest trending anticlines and synclines accompanied by the development of a number of normal and thrust faults.

As seen in Figure 2.4-34, seven major folds occur in the region. From north to south, they are the shallow syncline on the crest of North Mountain (in the Appalachian Plateau Province) the Milton anticline the Lackawanna syncline (including the Wyoming Valley) the Berwick anticline (on which the Susquehanna SES is located) the synclinorium of the Eastern Middle Basin in the vicinity of Hazleton the Selinsgrove anticline and the Mahanoy Basin, a synclinorium (Ref. 2.4-57). Faults, striking generally along the axis of the folds, occur within the Lackawanna syncline, the Berwick anticline, the Eastern Middle Basin and the Mahanoy Basin (Ref. 2.4-58).

With the exception of some of the Pleistocene deposits, no formation in the region has a high primary transmissivity. Both the primary porosity and permeability of the consolidated Paleozoic rocks are generally low. Thus, the joint systems, faults and solution channels caused by tectonic processes, weathering or solution activity subsequent to the deposition of these formations, take on considerable importance in enhancing the rocks' ability to transmit groundwater. Systems of

fractures or solution channels in bedrock can serve as groundwater pathways over distances of many miles, provided the openings have not been filled by precipitates or other solid matter.

In addition, the presence of sharply folded anticlines and synclines in the region in some cases provides special constraints on the flow of groundwater. Dips in the region range from 0^o to 40^o, with the maximum dips found on the rims and within the synclinal basins (Ref. 2.4-57). Groundwater will tend to flow within a specific formation to the extent that continuous pathways, fractures or solution channels occur preferentially in that formation. This would be particularly true of solution channels in limestone formations. In such cases, artesian or flowing wells are common, particularly in synclinal valleys (Ref. 2.4-57). However, to the extent that fracture systems extend across several adjacent formations, groundwater will not be confined to a particular formation, and the dip of the formation will provide no constraint on the flow of groundwater. In such a case, the alignment and interconnections of the joints or faults provides a major constraint on the flow, along with the direction and magnitude of the hydraulic gradient.

In general, groundwater in the Paleozoic rock formations of the Appalachian Highlands flows from the topographically higher areas (recharge areas) to the valleys (Ref. 2.4-57). It is believed that this groundwater discharges to springs and to the streams and rivers of the region, except at flood stage. However, no quantitative data in the form of piezometric contour maps are available to convey an accurate picture of the local or regional groundwater flow for any of the consolidated formations. In addition, there is no information at all on the flow of deep groundwater in the region.

An aquifer is defined as a rock unit or unconsolidated deposit that is saturated at least over a portion of its thickness, and is capable of transmitting groundwater through it readily. In the region around the Susquehanna SES, few of the bedrock formations have regularly yielded 100 gpm or more to an individual well. Yet, few, if any, of the formations can be considered to be aquitards, or non-aquifers. All the rock units to be described in this section are tapped by wells that provide, at the least, small domestic supplies of a few gallons per minute. This is because all the rock formations of the region contain to a greater or lesser extent the fracture systems or solution channels common to bedrock in the Valley and Ridge Province. To aid in the appraisal of the groundwater resources of the region, the discussion to follow will divide the geologic units into two groups, primary aquifers and secondary aquifers.

Primary aquifers are those generally tapped by the higher yielding industrial or municipal wells, and on the average, produce higher yields than secondary aquifers. Secondary aquifers generally provide water to only low-yielding domestic wells. The primary aquifers of the region include:

1) *Pleistocene-age outwash deposits and kame terrace deposits*
2) *The Pottsville Formation*
3) *The Mauch Chunk Formation*
4) *Upper Silurian Formations*

The secondary aquifers include:

1) *The Llewellyn Formation*
2) *The Pocono Formation*
3) *The Catskill Formation*
4) *Marine Beds (Devonian age)*
5) *The Mahantango, Marcellus and Onondaga Formations*

6) The Bloomsburg Formation

The only geologic units exposed in the region that are not included in the discussion to follow are those belonging to the Clinton Formation, the oldest group outcropping in the region. These units are exposed along the axis of the Berwick anticline about 11 miles southwest of the site, as seen in Figure 2.4-34. They are relatively unimportant with respect to groundwater, as they form a high ridge in outcrop (Ref. 2.4-57).

The extent of outcrop, or subcrop, of the consolidated rock units is shown on the bedrock geologic map (Fig. 2.4-34). The location of sand and gravel deposits laid down in the glacial age is shown in Figure 2.4-35. The stratigraphic relationships of the different geologic units of the region are given in Table 2.4-21, along with their groundwater yield characteristics.

2.4.13.1.1.1 Primary Aquifers of the Region Pleistocene - Age Deposits

Where there is sufficient saturated thickness, Pleistocene sand and gravel deposits generally serve as the highest yielding aquifers in the region. Figure 2.4-35 shows the location of the major surficial sand and gravel deposits in the region. These are, in general, part of the stratified drift resulting from the last (Wisconsin) ice invasion of the area. The location of the Wisconsin terminal moraine, indicating the farthest advance of the ice, is also shown on Figure 2.4-35.

From the point of view of groundwater, two types of stratified drift deposits generally serve as good aquifers, outwash sediments and kame terraces, both being confined to the valleys or low-lying areas. Outwash sediments were laid down by melt waters flowing ahead of the ice front. They consist of fine-grained well-sorted gravels (Ref. 2.4-59). Outwash sediments in the region are found chiefly in the valleys of Huntington and Fishing Creeks and along the Susquehanna River (Ref. 2.4-57). Kame terraces were formed by running water at the contact of the ice and the valley walls. They are commonly not as well sorted as the outwash sediments, and, hence may exhibit lower permeability. Kame terraces occur along the margins of the Susquehanna River valley (Ref. 2.4-60) and also in the smaller tributary valleys (Ref. 2.4-59). Not all of such small tributary deposits are shown in Figure 2.4-35.

The glacial sand and gravel deposits directly overlie the bedrock formations, or local colluvium. They are in places overlain by recent alluvium, which is generally either unsaturated or too thin to be considered important as a groundwater source. The thickness of the glacial sand and gravel varies widely from place to place. In some places, old deep valleys have been filled in primarily with glacial materials. In the Wyoming Valley of the Lackawanna Syncline (Figure 2.4-34), for example, such stratified deposits including clay layers, reach thicknesses of up to 300 feet (Ref. 2.4-57). In general, the stratified drift deposits usable as aquifers in the region range from 20 to 150 feet in thickness (Ref. 2.4-57).

Aquifer tests of four wells tapping the sand and gravel deposits of the Wyoming Valley indicated transmissivities ranging from 1,400 to 72,000 ft^2/day, and horizontal hydraulic conductivities ranging from 240 to 530 ft/day (Ref. 2.4-60). Storage coefficients obtained from the tests, with one exception, ranged from 0.01 to 0.13 indicating water-table conditions. In the one case, the aquifer was locally confined by a clay layer, and the storage coefficient obtained was $2.0x10^{-4}$.

Water levels in the sand and gravel deposits are responsive to both recharge from precipitation and the river or stream stage of the water body in the valley in which they are located. The water level in a well tapping a gravel and sand deposit in the Wyoming Valley, responded to the high stages of the Susquehanna River, during the Agnes storm in 1972 by rising from 16 feet below ground to 16 feet above ground (Ref. 2.4-61). Normally, in the deposits in the Wyoming Valley, the water table ranges from less than 10 feet below ground near the Susquehanna River to more than 30 feet below ground in the areas underlain by kame terraces or alluvial fans (Ref. 2.4-60). Seasonal water-level fluctuations were measured from 1965 to 1967 in shallow observation wells in this valley (Ref. 2.4-60). The amplitude ranged from 7 to 14 feet with the peaks occurring in the spring of the year. Water levels have been recorded in the USGS observation well (Lu-309) located in the Borough of Wyoming, north of the Susquehanna River and tapping outwash sand and gravel. In 1975, the water level in the well fluctuated between 9.8 and 20.3 feet below ground (Ref. 2.4-62). Groundwater discharge from these deposits to the stream or river generally tends to occur at most times except during flood stage.

Recharge to the sand and gravel deposits of the region occurs primarily by direct infiltration of precipitation and by infiltration from the stream and river beds during periods of high stage. The groundwater moves generally from areas of recharge to areas or points of discharge, whether a stream, spring, marsh, or a pumping well. The average hydraulic gradient in the glacial deposits over a section of the Wyoming Valley was determined to be 11 feet per mile (Ref. 2.4-60). Based on this and on an average transmissivity of about 8000 ft^2/day, it was estimated that the average rate of discharge from the aquifer to the Susquehanna River is approximately 15 inches per year, amounting to 39 percent of the average annual precipitation. The amount has been equated approximately to the average rate of recharge to these deposits (Ref. 2.4-60).

Because of the variability in the saturated thickness of the aquifer and in the quality of local well construction, well yields range widely. In Luzerne County yields from 6 to more than 1,000 gallons per minute (gpm) are reported for glacial sand and gravel deposits (Ref. 2.4-57 and 2.4-62). The gravel-packed wells near Pittston in the Wyoming Valley were tested at 1,280 gpm each with a drawdown of only nine to ten feet after eight hours of pumping (Ref. 2.4-57). In Columbia County, two wells along Fishing Creek tapping glacial sand and gravel deposits were reported to yield 140 and 830 gpm (Ref. 2.4-57). For the region as a whole, the median yield of wells tapping Pleistocene sand and gravel deposits is 100 gpm, based on 26 wells for which data were available (Ref. 2.4-57 and 2.4-62). Seaber's analysis indicates that where sufficient saturated thickness of these glacial deposits occur, 75 percent of properly constructed wells should yield 250 gpm or more (Ref. 2.4-63).

The Pottsville Formation

The Pottsville Formation is generally a hard quartzose unit consisting of gray conglomerate as well as white, gray or brownish sandstone (Ref. 2.4-57). Because of its resistance to weathering, it commonly forms ridges or mountains where it crops out. This is illustrated by the inner hills ringing the western part of the Wyoming Valley and by the hills aligned in an ENE-WSW direction in the Hazleton area. The Pottsville Formation underlies the Lackawanna syncline (Wyoming Valley), the Eastern Middle Basin in the vicinity of Hazleton and the Mahanoy Basin, but is absent elsewhere in the region. The Pottsville Formation, where it is not exposed in these basins, directly underlies the Llewellyn Formation, which is a Post-Pottsville formation and which is the primary coal-bearing unit of the region.

The Pottsville is of significantly greater thickness in the southern basins than in the Lackawanna syncline. In the Western Middle Basin, it is about 850 feet thick and in the Hazleton area it is about 500 feet thick, while in the Wyoming Valley its thickness ranges from only 150 to 300 feet (Ref. 2.4-57).

Of the 35 wells tapping the Pottsville in Luzerne County and for which recent data are available, 12 are reported to be flowing wells (Ref. 2.4-63). The static water level in the remaining wells is reported to range from 4 to 220 feet below ground, the wide range no doubt reflecting differences in topographic position and in the season of year when the measurement was taken. Of seven Pottsville wells studied in the 1930's in Schuylkill County, two were flowing and the static water levels in the remaining wells ranged from 10 to 32 feet below ground (Ref. 2.4-57). Large seasonal fluctuations in the water level are common in the region (Ref. 2.4-63).

No tests to estimate the aquifer parameters of the Pottsville Formation have been reported. Over a large part of the area where the formation occurs in the region, the fractured beds of sandstone and conglomerate are good water producers. In Luzerne County reported well yields range from less than 5 gpm to 160 gpm, with a median yield of 50 gpm (Ref. 2.4-63). The many flowing wells reportedly have large flows, but yield data are lacking. In Schuylkill County, yields ranging from 65 to 125 gpm have been reported, depending on the season. Here, a few deep wells in the Pottsville have been unsuccessful because of the absence of fractures in the formation at those locations (Ref. 2.4-57).

The Mauch Chunk Formation

The Mauch Chunk Formation consists of red, green, yellow or brown shale with some sandstones (Ref. 2.4-57). It is easily weathered and eroded and, consequently, has formed valleys or lowlands in the area where it outcrops in the region. As shown on Figure 2.4-34, its outcrops area comprises a large portion of the southern one-third of the region, surrounding the Eastern Middle Basin and the Mahanoy Basin. It crops out as a relatively narrow band around the Lackawanna syncline, making up a narrow valley between the hills of the Pottsville Formation and those of the Pocono Formation. These hills act as the double rim enclosing the western end of the Wyoming Valley. The Mauch Chunk Formation underlies the Pottsville Formation within the Synclinal basins, and is underlain by the Pocono Formation. In the region it is missing in the Berwick anticline area and north of the Lackawanna syncline.

The thickness of the Mauch Chunk Formation ranges from more than 2,000 feet in the southern part of the region to only 200 to 300 on the north side of the Wyoming Valley (Ref. 2.4-57). Northeast of Pittston, outside the region but still in the Lackawanna syncline, the Mauch Chunk is absent and the Pottsville Formation directly overlies the Pocono Formation.

Of the 51 wells tapping the Mauch Chunk in Luzerne County and for which recent data are available, seven are reported to be flowing wells (Ref. 2.4-63). The static water levels for the others are reported to range from 1 to 202 feet below ground, the wide range again reflecting differences in topographic position and the season of measurement. Data taken in the 1930's indicated that water levels in 48 wells tapping the Mauch Chunk in the portions of Carbon, Columbia, and Schuylkill Counties included in the region ranged from 1 to 130 feet below ground (Ref. 2.4-57). Only two of these had water levels at depths greater than 60 feet. In addition, six wells were reported as flowing.

No tests to determine the aquifer parameters of the Mauch Chunk Formation have been reported. The fractured beds of shale and sandstone in this formation yield moderate to relatively large supplies of water. The formation is particularly important as a source of groundwater because of the large areal extent of its outcrop in the southern part of the region. Well yields from the Mauch Chunk in Luzerne County range from 5 to 250 gpm (Ref. 2.4-63). Most wells in the county of more than 200-foot depth yield 25 gpm or more. The sandstone beds appear to be more productive than the fractured shale. In 50 wells tapping the Mauch Chunk in the portions of Carbon, Columbia, and Schuylkill Counties included in the region, yields in the 1930's were reported to range from 4 to 375 gpm (Ref. 2.4-57). The median yield based on 101 wells in the region for which data were available is 22 gpm (Ref. 2.4-57 and 2.4-63).

Upper Silurian Formations

Included in the Upper Silurian Formations in the region, in order of increasing age, are the Keyser Formation the Tonoloway Formation and the Wills Creek Formation. The Keyser Formation consists of alternating beds of sandy limestone and calcareous sandstone, some conglomeritic sandstone and a bed of soft shaly limestone (Ref. 2.4-57). The Tonoloway Limestone is about 100 to 150 feet thick and consists primarily of platy, laminated and argillaceous limestones with thick beds occurring locally at the top (Ref. 2.4-57 and 2.4-58). The Wills Creek Formation is made up of about 300 feet of alternating limestone, limy shales, and fissile shales (Ref. 2.4-57 and 2.4-58).

Some of the units included under these three formations and underlying the Onondaga Formation have been mapped as the Helderberg Formation (Ref. 2.4-57 and 2.4-58) assumed to be lower Devonian. More recent work by the Pennsylvania Geological Survey does not use the term Helderberg (for more information, refer to Subsection 2.5.1).

These formations crop out within the Berwick anticline from Berwick through Bloomsburg. They are underlain by the Bloomsburg Formation. Outside the Berwick anticline, the Upper Silurian Formations are overlain by the Onondaga Formation and the Marcellus Shale.

Groundwater in the Keyser and Tonoloway Formations occurs chiefly in solution channels and in some places in bedding planes and fractures enlarged by solution (Ref. 2.4-57). Static water levels of wells tapping these formations in the region are reported to range from 12 to 42 feet below ground in the 1930's (Ref. 2.4-57).

Some large yields have been recorded for wells tapping these formations. Within the region, recorded yields in four wells range from 16 to 250 gpm, and three of them yielded 125 gpm or more (Ref. 2.4-57). In addition, two wells tapping these formations near Berwick are reported to yield large, although unmeasured, supplies, with small drawdowns. A large spring, probably issuing from either the Tonoloway limestone or the Keyser Formation, is reported to occur in the bed of the Susquehanna River at the foot of the cliff below Berwick (Ref. 2.4-57).

2.4.13.1.1.2 Secondary Aquifers of the Region Llewellyn Formation

The Llewellyn Formation consists of sandstone, conglomerate, shale, fire clay, slate and numerous anthracite coal beds (Ref. 2.4-63). Beds of conglomerate and, in places, fireclay occur between the coal beds, which are the primary source for coal in the region (Ref. 2.4-57). The Llewellyn Formation in the region occurs only in the central portions of the Lackawanna syncline, the Eastern

Middle Basin and the Mahanoy Basin. It is directly underlain by the Pottsville Formation. The thickness of the formation is about 700 feet in the Hazleton area, 2,000 feet in the Mahanoy Basin, and nearly 2,200 feet in the Wyoming Valley (Ref. 2.4-57 and 2.4-60). The reported depths to water level in the Llewellyn range widely, from 1 to 342 feet, with four of the eight wells having reported water levels at depth greater than 150 feet (Ref. 2.4-63). No tests to determine aquifer parameters have been reported.

Small to moderate yields are obtainable from the fractured sandstone and conglomerate beds. Yields of wells tapping the Llewellyn Formation in the portions of Luzerne and Schuylkill Counties in the region range from 2 to 80 gpm (Ref. 2.4-57 and 2.4-63). The median yield of the 11 wells for which data were reported is 10 gpm. In the vicinity of the mining operations, some of the formation water drains into the mines. In addition, because of proximity to the mining operations, the quality of the groundwater is commonly poor. Highly acidic water results from the oxidation of pyrite found in the coal (Ref. 2.4-57).

Pocono Formation

The Pocono Formation consists of a hard massive gray sandstone and conglomerate, including some shale layers (Ref. 2.4-57). It is highly resistant to weathering and, consequently, over its outcrop area makes up the predominant ridges or hills of the region. These include, from north to south, North Mountain, Huntington Mountain/Shickshinny Mountain, Lee Mountain/Penobscot Mountain, Catawissa Mountain, Nescopeck Mountain, Little Mountain, and Broad Mountain. Huntington Mountain/Shickshinny Mountain and Lee Mountain/Penobscot Mountain serve as the outer rim of the Wyoming Valley. The Pocono Formation underlies most of the southern part of the region as well as the Lackawanna syncline. It is absent in the Berwick anticline Formation underlies the Mauch Chunk Formation and directly overlies the Catskill Formation. The Pocono Formation ranges in thickness from over 1,000 feet in the southern part of the region to about 600 feet in the north (Ref. 2.4-57).

Data from 10 wells tapping the Pocono Formation in the region indicate that four of the wells were flowing (Ref. 2.4-57 and 2.4-63). The depth to the static water level in the remaining wells, with one exception, ranges from 14 to 80 feet. One well yielding 133 gpm had a reported water-level depth of 300 feet (Ref. 2.4-63) which may more properly represent a pumping water level.

According to the available literature, no tests have been performed to estimate the aquifer parameters of the Pocono Formation. Moderate yields are obtainable when wells penetrate well-fractured saturated zones. Most all the wells tapping the Pocono in the region are in Luzerne County, and many of these are located along the north rim of the Wyoming Valley (Ref. 2.4-57).

The formation is reported to be a productive aquifer on the Appalachian Plateau when it occurs below drainage level and has a significant saturated thickness (Ref. 2.4-92). In this area, but probably outside of the region to the northwest, yields from the Pocono of more than 200 gpm are likely (Ref. 2.4-92). Within the region, reported yields range from 3 to 133 gpm (Ref. 2.4-63). Neglecting the one high flow of 133 gpm, the average yield is about 10 gpm.

The Catskill Formation

The Catskill Formation consists of red to brownish shales, red and gray crossbedded sandstone, and gray to green sandstone tongues (Ref. 2.4-58 and 2.4-63). As seen in outcrop (Figure 2.4-34)

it forms the outer limbs of the Milton, Berwick and Selinsgrove anticlines, and underlies about 75 percent of the region. It directly underlies the Pocono Formation, and is, in turn, underlain by the Devonian Marine Beds. The maximum exposed thickness of the formation over the major part of the region is about 1,700 feet. There is evidence, however, that the thickness may increase to 3,000 to 4,000 feet in the southernmost part of the region (Ref. 2.4-57).

Out of 75 wells tapping the Catskill Formation in Luzerne County for which data are available, six wells were reported as flowing (Ref. 2.4-63). Static water levels in the remainder ranged from 6 to 215 feet below ground, with 62 of the wells having water levels within 70 feet of the surface. During 1976, water levels in a USGS observation well (LU-243) tapping the Catskill Formation in the northern part of Luzerne County fluctuated between 49.5 and 55.1 feet below ground (Ref. 2.4-62). Water levels in Catskill wells located in Columbia and Carbon Counties were reported in the 1930's to range from 6 to 60 feet below ground (Ref. 2.4-57).

In general, the hard fractured sandstones of the Catskill Formation yield more water than do the shale beds of the formation (Ref. 2.4-57). The range of reported yields of wells tapping Catskill beds in Luzerne County is 2 to 325 gpm (Ref. 2.4-63). For the 63 wells in the county for which data are available, the median yield is 12 gpm, and 75 percent of the wells yield 25 gpm or less (Ref. 2.4-63). Seventeen Catskill wells in Columbia County and the portion of Carbon County included in the region were reported in the 1930's to yield from 1 to 75 gpm (Ref. 2.4-57). Seventy-five percent of these wells yielded 10 gpm or less.

<u>Marine Beds</u>

The Devonian Marine Beds, together with the Catskill Formation, has in the past been mapped as an undifferentiated unit termed the Susquehanna Group (Ref. 2.4-58). Within the region, the primary constituent of the Marine Beds is Trimmers Rock, which consists principally of hard gray to greenish-gray massive to flaggy sandstone containing little shale (Ref. 2.4-57). Brallier Shale and Harrell Shale are minor members of the Marine Beds and they appear to be missing over at least a portion of the region. The Marine Beds are present in most of the region, and are overlain by the Catskill Formation except within the Milton and Berwick anticlines. The Marine Beds overlie the Mahantango Formation. The total known thickness of the Marine Beds in the region ranges from about 1,500 to 3,000 feet, of which nearly the entire thickness of Trimmers Rock (Ref. 2.4-57).

Out of 16 wells tapping the Marine Beds in Luzerne County for which data are available, two were flowing wells (Ref. 2.4-63). The remaining wells have static water levels ranging from 18 to 63 feet below ground. Static water levels for wells tapping Marine Beds in Columbia County were reported in the 1930's to range from 3 to 50 feet below ground, with one of the 15 wells studied being a flowing well (Ref. 2.4-57).

Low yields are obtainable from wells tapping fracture zones in the Marine Beds. The range of the measured yields of 21 wells tapping Marine Beds in the region ranged from less than 1 to 15 gpm, with a median yield of 5 gpm (Ref. 2.4-57 and 2.4-63). Newport states that some of the wells tapping the Marine Beds are reported to yield large supplies, however, no measurement has been made (Ref. 2.4-62).

<u>The Mahantango, Marcellus and the Onondaga Formations</u>

On a regional scale, the Mahantango Formation and the Marcellus Shale have been mapped together as the Hamilton Group (Ref. 2.4-58). The Mahantango Formation is the youngest unit, and overlies the Marcellus Shale, which in turn overlies the Onondaga Formation.

Within the region, the Mahantango Formation is about 1,100 feet thick and consists chiefly of bluish-gray to brownish sandy shale, with some interbedded sandstones, and locally thin bluish-gray limestone (Ref. 2.4-57 and 2.4-58). The underlying Marcellus Shale consists of about 400 feet of black, gray or dark-blue fissile shale (Ref. 2.4-57). The Onondaga Formation generally consists of a non-cherty limestone member overlying a gray calcareous shale (Ref. 2.4-57). It is reported to be 140 feet thick in the Selinsgrove anticline (Ref. 2.4-57).

The Mahantango Formation crops out in the vicinity of the Susquehanna SES and underlies almost the entire region, with the exception of the central portion of the Berwick anticline between Berwick and Bloomsburg (Ref. 2.4-58). These formations are underlain by Upper Silurian Formations, and except within the central portions of the Milton and Berwick anticlines, are overlain by the Marine Beds.

Water levels in the Hamilton Group Formations have been reported to range from 7 to 40 feet below ground (Ref. 2.4-57 and 2.4-63). One well in Columbia County was reported in the 1930's to be flowing (Ref. 2.4-57). Yields have been reported to range from 2 to 21 gpm (Ref. 2.4-57 and 2.4-63) although one well in Columbia County tapping the Mahantango Formation (or possibly Marine Beds) was reported to have a "large" yield at a large drawdown (Ref. 2.4-57).

Bloomsburg Formation

The Bloomsburg Formation is about 800 feet thick in the region. It consists of dark-red sandy shale with a few thin layers of bright-green shale and a few beds of red sandstone (Ref. 2.4-57). The underlying McKenzie Formation is about 150 feet thick and consists of red to green shale, gray calcareous shale and some dark blue limestone (Ref. 2.4-57). It underlies essentially the entire region and immediately overlies units of the Clinton Formation. Except in the core of the Berwick anticline, the Upper Silurian formations overlie the Bloomsburg Formation.

Static water levels in the 1930's of four wells tapping the Bloomsburg in the region ranged between 12 and 55 feet below ground. During 1976, water levels in a USGS observation well (Co-45) tapping the Bloomsburg Formation and located near the Town of Bloomsburg, fluctuated between 81.0 and 86.3 feet below ground (Ref. 2.4-62). Yields of the Bloomsburg Formation range from 5 to 20 gpm, although one well was reported to give a "large" though unmeasured supply (Ref. 2.4-57).

2.4.13.1.2 Local Groundwater Conditions

The local area is herein defined as the area within a two-mile radius of the Susquehanna SES. Within a two-mile radius of the Susquehanna SES, three rock formations crop out and are tapped for groundwater supply. These are, from south to north, the Mahantango Formation, the Trimmers Rock Formation and the Catskill Formation, shown on Figure 2.4-36. In addition, several wells tap unconsolidated deposits, including Pleistocene sand and gravel, Holocene alluvium and residual soil. Most of these are located on the Susquehanna River flood plain. No withdrawal greater than 3,000 gallons per day is made from any existing well within two miles of the station. The general description of these formations and deposits is given in Subsection 2.4.13.1.1.

A door-to-door inventory of wells and springs utilized for water supply within two miles of the Susquehanna SES was performed in March 1977. Details of the results of this inventory are presented in Tables 2.4-22 and 2.4-23. The locations of the wells and springs are shown on Figures 2.4-37 and 2.4-38, respectively.

The Mahantango Formation, a blue-gray siltstone, underlies more than half of the two-mile radius area and is found immediately beneath the Susquehanna SES (Figure 2.4-36). On the north side, along its contact with the Trimmers Rock Formation, it is commonly a limey siltstone. In the vicinity of the station, boring log and pressure test information indicate the rock to be moderately well fractured in the upper 10 to 20 feet, with significantly fewer fractures at greater depth. Thus, in many locations in the local area, one would expect that wells tapping the Mahantango may obtain most of their supply from the upper 10 to 20 feet of rock.

Table 2.4-22 indicates that out of a total of 185 wells inventoried within the two-mile radius, 125 tap the Mahantango Formation. Of 114 Mahantango wells for which data were obtained, the range of depths is 20 to 354 feet with a median depth of 90 feet. Neglecting two large questionable values, reported yields from Mahantango wells range from 2 to 130 gpm with a median value of 15 gpm. Reported estimates of depth to static water level indicate a range of 1 to 100 feet with a median of 20 feet. Eighty local residents having wells tapping the Mahantango Formation (comprising nearly 70 percent of those giving water quality information) report their well water to be hard. Of these, 14 stated the water also contained iron, a sulfide, or both. The quality of water in three of these wells is so poor it cannot be used for drinking.

Table 2.4-23 indicates that out of a total of 33 springs used for water supply in the local area, only six are believed to issue from the Mahantango Formation.

The Trimmers Rock Formation in the local area consists of thinly laminated siltstone or silty shale and hard, often flaggy, fine-grained sandstone. Groundwater occurs primarily in the rock fractures, as the primary porosity of the rock is essentially nil. The contact between the Mahantango and the Trimmers Rock Formation is located about 1,500 feet north of the center of the Susquehanna SES plant area.

Forty-five of the 185 wells inventoried in the two-mile radius are believed to tap Trimmers Rock, and 15 of the 33 springs utilized for water supply are believed to issue from this formation. As taken from Table 2.4-22, the range in well depths is 20 to 460 feet, and the median depth is 150 feet, significantly greater than that for Mahantango wells (90 feet). The difference may be due in part to the fact that the area underlain by Trimmers Rock is topographically higher than that underlain by the Mahantango Formation.

The data given in Table 2.4-22 indicate that of seven wells for which data were reported, the well yields from Trimmers Rock range from 6 to 60 gpm with the median value 9 gpm. The largest yielding developed spring in the local area is owned by the Citizens Water Company of Wapwallopen and is given as No. 7 in Table 2.4-23. It is believed to issue from the Trimmers Rock Formation and supplies about 8,200 gpd.

The reported depths to static water level in Trimmers Rock wells in the local area range from 0 to 50 feet with a median of 22 feet. Water from approximately 55 percent of the Trimmers Rock wells is

reported to be hard; and of these, 40 percent are reported to contain iron, a sulfide or both. The quality of water in three of the wells is so poor that it cannot be used for drinking.

The Catskill Formation in the local area consists of reddish-brown to maroon sandstone, siltstone or mudrock, and greenish-gray or olive-gray fine-grained sandstone, siltstone, silty shale or shale. The size of the area underlain by this formation within the two-mile radius is small (Figure 2.4-36). None of the wells inventoried in the area appear to tap the Catskill Formation. One spring believed to be issuing from the Catskill Formation is utilized for water supply (No. 33 in Table 2.4-23).

The primary source for relatively large groundwater supplies in the local area is Pleistocene sand and gravel deposits. However, only 10 existing wells within two miles of the station are believed to tap these deposits; and they withdraw only small quantities, for domestic or stock watering purposes as seen in Table 2.4-22. Essentially all the Pleistocene deposits within two miles of the station are mapped as kame terrace deposits. As seen on Figure 2.4-36, the kame terrace deposits (Qkt) cover nearly one-fourth of the two-mile radius area. In addition, the sand and gravel deposits commonly underlying the Holocene alluvium (Qal) are, in all likelihood, kame terrace deposits.

The major portion of the kame terrace deposits consists of stratified sand and gravel, including varying amounts of silt, grading with cobbles and boulders particularly in the lower part of the deposit. The overlying portion commonly consists of well-sorted fine to medium sand, or fine sand and silt, which exhibit both simple and complex bedding structure. In general, thicker sequences of the deposits would be expected to occur close to the river. The permeability of the kame terrace deposits can vary considerably areally and with depth.

The ten wells in the local area tapping these deposits range in depth from 20 to 100 feet with a median depth of 22 to 24 feet. The wells are mostly dug wells two to three feet in diameter. The reported static water level ranges from 5 to 75 feet below ground with the median value of 12 feet. Eighty percent of the well owners having wells tapping kame terrace deposits reported the water to be soft and of good quality.

Four other wells within two miles of the site tap unconsolidated deposits other than kame terrace deposits. Two are believed to tap Holocene alluvium, one along Wapwallopen Creek and the other along Walker Run. These have shallow depths (≤18 feet) and have reported static water levels of two feet below the surface. North of the site there are two dug wells apparently completed in the residual soil or the upper highly weathered portion of the underlying Trimmers Rock. These are of shallow depth (<15 feet) with a reported static water level just four feet below the surface.

END HISTORICAL

2.4.13.1.3 Onsite Use of Groundwater

Plant use of groundwater is anticipated during the operation of the plant. Two production wells, TW-1 and TW-2, exist on site and are located about 1,200 feet northeast of the turbine building. They have been used for construction purposes, and have an approximate capacity of 50 gpm and 150 gpm, respectively. During plant operation, these wells fill the clarified water storage tank and the domestic water storage tank and supply seal water for the circulating water pumps and the

service water pumps. Clarified river water may occasionally be used to supply some of these needs.

Two 30 gpm wells exist at the River Water Make Up facility and are utilized for seal water to the River Water Make Up pumps. These wells are located about 200 feet north of the River Water Make Up facility.

START HISTORICAL

2.4.13.2 Sources

2.4.13.2.1 Water Well Inventory

A complete water well inventory in the local area was performed by making a house-to-house survey within two miles of the Susquehanna SES during March 1977. The results of this inventory with the available well data are presented in Table 2.4-22. The locations of these wells are given in Figure 2.4-37. Wherever springs were utilized for water supply they were tabulated separately. The pertinent information on the springs used locally is given in Table 2.4-23 and their locations are shown in Figure 2.4-38. A summary discussion of the information given in these two tables is provided in Subsection 4.13.1.2. Estimates of present withdrawal rates from each well or spring were calculated on the daily per-person or per-animal consumption rate shown at the bottom of Tables 2.4-22 and 2.4-23, based primarily on Reference 2.4-64.

A total of 185 water wells and 33 developed springs were inventoried in the two-mile radius area. The vast majority of the wells are used for domestic or stock-watering purposes. Nineteen of the wells are used, at least in part, for commercial purposes; seven are currently unused, and one is used as standby for public supply purposes by the Citizens Water Company of Wapwallopen. The largest estimated average withdrawal from a single well in the area is about 2,700 gpd. With one exception, the developed springs in the local area provide supplies of water only for domestic and stock use. At Wapwallopen, the Citizens Water Company withdraws an average of 8,200 gpd from a spring believed to issue from the Trimmers Rock Formation.

In the region, an inventory of major wells (with the exception of public-supply wells) located between 2 and 10 miles from the Susquehanna SES was performed. A major well was defined as one with a reported tested yield of 15 gpm or more. In addition, an inventory of all public supply wells located between 2 and 20 miles from the station was carried out. The source for both these inventories were a published report (Ref. 2.4-63) and unpublished records and computer printouts from Bureaus of the Pennsylvania Department of Environmental Resources (Ref. 2.4-65 through 2.4-69).

The results of the major-well inventory are presented in Table 2.4-24 and include well location, owner, use, total depth, probable aquifer tapped, reported well yield, specific capacity and static water level. The locations of these wells are shown on Figure 2.4-39. A total of 77 major wells has been enumerated. Reported well yields range up to 550 gpm, and the median value of those wells for which yields are reported is 20 gpm. With the exception of three industrial wells located near Nanticoke, the remaining wells are used exclusively for domestic or stock-watering purposes.

The results of the inventory of public-supply wells are provided in Table 2.4-25, and their locations are shown in Figure 2.4-40. A total of 213 public-supply wells was enumerated over the 20-mile radius area. The area has a large number of small water-supply companies or municipal departments, and because of the relatively low yield of many wells completed in rock, a considerable number of wells is required. As shown on Figure 2.4-40, the majority of these wells are concentrated either in the vicinity of the Wyoming Valley, northeast of the station, or in the southeastern quadrant, in the Freeland-Hazleton-Mahanoy City area.

2.4.13.2.2 Groundwater Withdrawal

The estimated average groundwater withdrawal rate during 1976 from all wells and springs within a two-mile radius of the site is 56,000 gpd, which is equivalent to only 38.9 gpm. Table 2.4-26 shows the estimated withdrawals from wells and from springs, as well as from individual geologic units within this local area for that year. Approximately 52 percent of the withdrawals is from the Mahantango Formation. Spring withdrawal amounts to about 25 percent of total groundwater use in the area. The values in Table 2.4-26 were obtained by summing up the appropriate figures in the column for "estimated present average withdrawal" in Tables 2.4-22 and 2.4-23.

The estimated projections of groundwater use through the year 2020 in the two-mile radius area are given in Table 2.4-27. It is estimated that by the year 2000, local groundwater withdrawal will amount to about 64,000 gpd. The projections are based on the population projections given in Tables 2.1-7 through 2.1-16.

Estimates of regional groundwater withdrawals are based on records and computer printouts of the Pennsylvania Department of Environmental Resources (Ref. 2.4-70) a personal communication with a water department (Ref. 2.4-71) and the U.S. Census publication for 1970 (Ref. 2.4-72). Tables 2.4-28 and 2.4-29 summarize the information and calculations on which we based the estimate of the groundwater withdrawal rate for 1975 within 20 miles of the station. As shown in Table 2.4-29, the estimated average withdrawal in 1975 from all geologic units by water departments or companies and by industries was 6.3 mgd and that from private domestic wells and springs was 5.2 mgd. Thus, the estimated average withdrawal rate in 1975 was 11.5 mgd for the 20-mile area.

Table 2.4-27 gives the estimated projections of groundwater use in the region through the year 2020. The projections are based on population projections as found in Tables 2.1-15 through 2.1-16. The estimated average groundwater withdrawal rate within 20 miles of the station for the year 2000 is 12.1 mgd.

2.4.13.2.3 Aquifer Characteristics and Groundwater Conditions at the Site

2.4.13.2.3.1 Data Sources

Previous Investigations

As defined herein, the site or site area refers to property owned by PP&L at the Susquehanna SES. A number of borings, observation wells and test or production have been drilled on the PP&L property at the Susquehanna SES as a part of previous investigations. These have been drilled at different times since 1965 under the supervision of different engineering contractors. Data from a total of 328 borings or wells on the property have been used in evaluating the hydrogeologic conditions on site. Their locations, along with those for test pits, are shown in Figures 2.4-41 and 2.4-42.

The data utilized from these borings and wells include boring log data, with lithologic and structural notations, and the results of water pressure tests performed at several borings. In addition, pumping tests of the overburden materials were carried out in the two production wells located on the north side of the property, TW-1 and TW-2. Laboratory tests, including grain-size, dry density and permeability tests, have been performed on a number of soil samples obtained from the borings on site. Water-level data have been obtained in borings in the process of their being drilled, and at frequent intervals in observation wells constructed on the property.

The data from these previous investigations have been obtained from published documents, reports submitted to PP&L, and unpublished records (Ref. 2.4-73 through 2.4-85).

Investigations Performed for this Report

Some of the observation wells constructed during previous investigations were found to be usable for this investigation. They were each confirmed to be in hydraulic continuity with the geologic unit(s) they are open to. This was done by pouring in a slug of water of known volume (two to five gallons) and measuring the rate of recovery of the water level. A total of 11 observation wells (Nos. 2, 8, 11, 19, 109, 124, 1111, 1113, 1114, B-1 and CPW) were found to be in satisfactory condition and have been used for groundwater level monitoring since early November, 1976. The location of these observation wells is shown on Figures 2.4-32 and 2.4-43. Details of their construction are given in Table 2.4-30.

Six new observation wells were constructed in the summer of 1977 as a part of this investigation (Nos. 1200A, 1201, 1204, 1208, 1209A, and 1210). Four of these are overburden wells and tap the overburden and the upper two to three feet of bedrock. The remaining two (1201 and 1209A) are bedrock wells and tap the zone between 4 and 34 feet below the top of bedrock. The location of the new observation wells is along a narrow band running east from the plant area to the river as indicated in Figure 2.4-32. Details of the manner of their construction are given in Table 2.4-30. Figures 2.4-44 through 2.4-49 provide the boring log information and schematic well construction details for each well.

Water pressure tests (packer tests) were performed in both of the bedrock observation wells (Nos. 1201 and 1209A). Pumping tests were conducted for overburden wells 1204 and 1210. The purpose of the water pressure tests (packer tests) and pumping tests was to provide estimates of

the horizontal hydraulic conductivity of the upper bedrock and the overburden soils, respectively, along the groundwater path from the plant to the river.

2.4.13.2.3.2 Groundwater Parameters and Movement at the Site

On the PP&L property at the Susquehanna SES, the saturated portion of the overburden serves as an aquifer. The underlying fractured portion of the Upper Mahantango siltstone also contains groundwater, but its generally lower porosity (storage capacity) and permeability make it of only secondary importance as a local aquifer.

The overburden consists primarily of kame terrace Pleistocene deposits. East of Route 11 on the flood plain, these deposits are covered with up to 10 to 20 feet of Holocene alluvial material consisting of silty fine sand or fine sandy silt.

The Pleistocene kame terrace deposits are poorly to moderately well-graded stratified deposits consisting dominantly of sand and gravel, with variable amounts of clay, silt, cobbles and boulders. In portions of the area, the upper layers tend to consist of well-sorted fine to medium sand, or fine sand and silt, exhibiting both simple and complex bedding structures. The lower layers are generally more coarse-grained and more well graded, with cobbles and boulders occurring most abundantly near the top of bedrock. Elsewhere, these deposits consist of alternating layers of: (1) relatively poorly-graded sand and gravel; and (2) a well-graded mixture of clay, silt, sand, gravel, cobbles and boulders.

The lenses and layers of poorly graded (uniform) sand or sandy gravel are most important from a groundwater point of view, as they are high in permeability. Thus, the bulk of the groundwater flow will tend to flow through these layers where they are continuous for some distance. Boring log information indicates that a moderately to highly permeable zone exists within the lower 20 feet of overburden, over considerable distances within the station area.

An isopach map of the Susquehanna SES area, which indicates the approximate thickness of overburden across the site, is given in Figure 2.4-41. The thickness of overburden on site ranges from 0 to 125 feet. By comparison to Figure 2.4-42, which gives the approximate top-of-bedrock contours for the site area, it is seen that the greatest thickness of overburden occurs in the two east-west oriented buried bedrock valleys, which occur on the northern side of the site. One of these valleys (called the "major bedrock valley") appears to extend all the way from the west side of the property to the river, between Pennsylvania coordinates, N341,500 and N342,500. A prominent kame terrace with a thickness of up to 70 feet occurs along the southern flank of this valley, between the plant and Route 11, as seen on Figure 2.4-41. The other important east-west bedrock valley is located about 1500 feet further north and extends from the location of Route 11 to the river. Figure 2.4-42 also shows a secondary bedrock valley extending from the plant in a northeast direction until it joins the major bedrock valley.

Apart from the above described features, the overburden thickness in the site area west of Route 11 is 20 feet or less. On the flood plain, the overburden thickness is seen to range from less than 20 feet up to 125 feet, with the average probably in the range of 50 to 80 feet, clearly higher than that in the upland area. To illustrate the nature of the topography and the thickness of the overburden over the site area, Figure 2.4-33 shows a geologic cross-section extending eastward

from the northern part of the turbine building to the Susquehanna River. The location of the cross-section along a groundwater flow path is shown on Figure 2.4-33.

Of greater importance than the overall overburden thickness is the height of the groundwater level above the top of bedrock. Where groundwater is unconfined, this corresponds to the saturated thickness which varies from season to season and from year to year depending on the quantity of water recharging the groundwater. Where groundwater is confined, as on the flood plain, the saturated thickness of the aquifer generally remains constant, while the height of the groundwater level fluctuates. The height of groundwater in the overburden at the site ranges from 0 to about 90 feet, and the values at various borings or wells and at different times are given in Tables 2.4-31 and 2.4-32.

On the flood plain, the height of the groundwater level above bedrock in the overburden generally ranges from 30 to 90 feet, with the greatest thickness found along the river and within the two major bedrock valleys, as shown by wells B-1 and CPW in Table 2.4-32. On the uplands, the height of the water level above bedrock ranges from less than zero to about 65 feet. The greater thicknesses are always found in the center of the bedrock valleys; and generally, the greater the distance from the axis of the valleys, the less the thickness. For example, as shown in Table 2.4-32, in early November, 1976, the height of the water level above bedrock at observation well 109 located in the center of the major bedrock valley was 65 feet, while at nearby observation well 124, located on the north flank of that valley, the thickness was only 20 feet. The saturated thickness of unconfined Pleistocene deposits near the center of the major bedrock valley, about 500 feet west of U.S. Route 11 at Well 1208, was found to be 8.8 feet in August 1977. Forty feet to the east at Well 1210, it was only 3.9 feet.

Static water levels measured in 1972 indicate that in the vicinity of the reactor area, turbine building and the cooling towers, the height of the groundwater level above bedrock ranges from 0 to 18 feet. This is shown in Table 2.4-31 for the relevant borings: 116, 202, 205, 206, 209, 211, 215, 301, 312, 317, 319 and 444. With the exception of the June 1972 reading at boring 319, the groundwater levels in this area were only 0 to 8 feet above the top of rock.

At one observation well, No. 1114 located in the spray pond area, groundwater levels in the period 1974 to 1977 were in the bedrock. Tables 2.4-31 and 2.4-32 indicate that static water levels in this well ranged between one to five feet below top of rock.

The approximate height of the groundwater level in the Pleistocene deposits along the assumed groundwater flow path from the northern part of the plant to the river is shown on Figure 2.4-33.

The underlying bedrock at the site consists primarily of the Mahantango siltstone, while the Trimmers Rock sandstone borders the Mahantango on the north, approximately along coordinate N343,500. The logs of borings penetrating up to 250 feet of the Mahantango siltstone were examined. Broken or severely fractured zones commonly occur at the bedrock and alternate between massive and moderately fractured zones. No uniform pattern was observed with respect to the occurrence of fractures at depth. There does, however, appear to be a tendency for the fractures or joints at depth to be filled in with calcite, pyrite or quartz crystals. This is also true of the many brecciated zones occurring in the rock cores. However, open or partially open joints and fractures do occur at depths greater than 20 feet below the top of rock. The joint planes or cleavage planes examined were nearly always in the range of 30^o to 60^o from the horizontal, and generally opposing or nearly perpendicular to the bedding planes.

Water pressure tests (packer tests) in the 300-series, 900-series and 1200-series borings on site indicate a clear tendency for the effective rock permeability to decrease with depth within the upper 50 feet of bedrock (Ref. 2.4-75 and 2.5-84). This is shown in Tables 2.4-33 and 2.4-34.

Water level data from the overburden and bedrock wells of the 1200-series indicate this upper relatively permeable bedrock zone is not everywhere in direct hydraulic connection with the overlying Pleistocene deposits. Water levels measured in bedrock Well 1201 have been about 11 feet higher than in overburden Well 1200A, 6.4 feet away, as shown in Table 2.4-30. With the exception of the uppermost fractured bedrock zone of one-to-three-foot thickness (tapped by the overburden wells in the 1200-series), it appears that groundwater filling the underlying bedrock fractures forms in places over the site area a hydraulic system essentially separate from that of the overburden.

Many of the static water level readings presented in Tables 2.4-31 and 2.4-32 were made in borings or observation wells in hydraulic connection with both the bedrock and the overlying overburden. Thus, assuming that water levels in the upper bedrock do not generally coincide with those for the overburden, the water levels in such cases represent composite levels probably dominated by the overburden. This would clearly be the case for borings 7 through 209 and 215 through 319 as given in Table 2.4-31 and for observation wells 8, 109 and 124 in Table 2.4-32.
The fluctuation of the groundwater table at the Susquehanna SES is indicated in Tables 2.4-31 and 2.4-32. From July 1974 to July 1975, water levels in observation wells 1111, 1113 and 1114 fluctuated within a range of five feet, while from November 1976 to late April 1977, they fluctuated within 0.9 to 7.5 feet. For wells 1111 and 1113, groundwater levels were one to eight feet higher between November 1976 and May 1977 than for the same period in 1974-75. At well 1114, on the other hand, groundwater levels were three feet lower in 1976-77 than in 1974-75. This reversal of groundwater level trends in the same general area may possibly be ascribed to the effect of large amount of earth-moving work performed over this period in close proximity to the spray pond area. The construction of the railroad embankment, settlement of fill material and excavations in the immediate vicinity could have a profound affect on the elevation and gradient of the local water table.

For the other seven observation wells for which full records are available, groundwater levels onsite fluctuated within a range of 5.5 to 11.2 feet for the period from early November 1976 until mid-September 1977. The exception to this was Well 109 which experienced a water level decline of 22.2 feet between April 14th and September 20th because of its proximity to the production wells TW-1 and TW-2.

Figure 2.4-50 shows the approximate groundwater contours onsite recorded in June 1971. These contours should be considered largely a composite of overburden and upper bedrock water-level contours, with the overburden levels exerting primary influence. Figures 2.4-32 and 2.4-43 show the groundwater contours in September 1977 and April 1977, respectively, over the portion of the site area for which wells were available. As Wells 1201 and 1209A are clearly bedrock wells, their water level data were not included in the contours shown in Figure 2.4-32. Wells 1111, 1114 and 1115 were destroyed in May 1977 in the process of constructing the spray pond. Hence, data for that area were not available for contouring in Figure 2.4-32. None of the 1200-series wells were contoured in Figure 2.4-43 as they were constructed after April 1977.

The direction of groundwater flow away from the plant area, as shown in Figures 2.4-32, 2.4-43, 2.4-50 and Dwg. FF62005, Sh. 1 is generally toward the northeast, and hence, eastward to the

river. There seems to be a clear tendency for the groundwater flow paths to follow the major and minor buried bedrock valleys shown in Figure 2.4-42. A portion of the groundwater flowing eastward discharges as springs along the stream running eastward toward Route 11, approximately along Pennsylvania coordinate N342,100. However, it appears that most of the groundwater discharges ultimately to the Susquehanna River.

Consistent with the topographic relief, the average groundwater gradient is quite high between the plant area and observation Well 1210, located 550 feet west of U.S. Route 11. The average hydraulic gradient over this reach of the groundwater path is about 0.068, based on groundwater levels taken in the fall of 1970 and in September 1977. The average gradient in the flood plain from Well 1210 to the river is estimated to be only 0.0073 based on September 1977 water level readings.

The slope of the piezometric surface toward the east in the upper bedrock between Wells 1201 and l209A had a magnitude of 0.084 in September 1977.

A summary of the aquifer tests and permeability tests carried out in previous investigations in the overburden materials and the upper bedrock on the site is presented in Table 2.4-33. The results of tests performed for this investigation are summarized in Table 2.4-33. It is seen that the estimated horizontal hydraulic conductivity of the Pleistocene kame terrace deposits varies widely, from 0.022 to 200 feet/day, while estimates for the vertical hydraulic conductivity range from 2.3 to 63 feet/day. Horizontal hydraulic conductivity values obtained from packer tests for the upper 20 to 30 feet of bedrock range from 0 to 2.5 feet/day. Table 2.4-33 indicates that the upper 20 feet of bedrock commonly is significantly more permeable than are intervals lower than 20 feet, presumably because of the greater frequency of open joints in the uppermost bedrock zone. The values given for borings 305, 1201 and 1209A shown in Table 2.4-34 support this finding.

Two of the drawdown curves used to obtain estimates of horizontal hydraulic conductivity from the pumping tests of Wells 1204 and 1210 are presented in Figure 2.4-51. Well 1204 was pumped at a constant rate of 25.8 gpm for six hours. The range of values shown in Table 2.4-34 for the pumping test of Well 1204 derive from analysis of the time drawdown curves and the recovery curves for both the pumping Well (No. 1204) and the observation Well (No. 11). Well 1210 was pumped for nearly six hours at a constant rate of 1.1 gpm with no drawdown observable in Well 1208, 40 feet away. Transmissivity and horizontal hydraulic conductivity in the vicinity of Well 1210 were estimated from the recovery curve of the pumped well as shown in Figure 2.4-51. Calculations are shown on the figure.

Slug tests were performed in Wells 1208 and 1210 by quickly introducing 5 gallons of water into the well and measuring the subsequent sudden rise and gradual decline of the water level with time. The results in both cases were analyzed using the formula for a well point filter in a uniform soil given by Lambe and Whitman (Ref. 2.4-93).

To estimate groundwater movement in the Pleistocene deposits from the plant toward the river, it was necessary to divide the groundwater flow path into segments because of the deposits' wide range in horizontal hydraulic conductivity (K_h). Based on the results of pumping tests summarized in Tables 2.4-33 and 2.4-34, values for K_h were selected for each segment. It is reasonable to assign a value of 8 ft/day for K_h for the deposits west of Well 1210. The segment from Well 1210 eastward to Well 1204 is assigned an average K_h of 22 ft/day. And, eastward of well 1204 to the river, the average K_h may be taken as approximately 120 ft/day.

It is difficult to estimate movement of groundwater in the upper bedrock because of uncertainty about the areal extent of the open fractures tested by packer tests in the borings. Based on the results of packer tests summarized in Table 2.4-34, the average horizontal hydraulic conductivity of the upper bedrock over at least the western portion of the groundwater path shown in Dwg. FF62005, Sh. 1 would probably be less than 0.50 ft/day.

As discussed earlier, over much of the site area where there is a significant saturated thickness of Pleistocene deposits, there are overlying layers of lower permeability, consisting commonly of sandy silt or even clayey silt. These layers serve to confine the water in the aquifer materials, at least locally. Indeed, the results of the pumping tests of these deposits at wells TW-1 and TW-2, indicated a storage coefficient ranging from 1×10^{-4} to 4×10^{-4} which implies the aquifer at that location is confined (Ref. 2.4-80).

Similarly, on the flood plain, analysis of the pumping test of Well 1204 indicated the aquifer to be confined with a storage coefficient of 7.0×10^{-5} to 1.5×10^{-4}. But in the vicinity of Well 1210, just west of U.S. Route 11, analysis of slug tests based on the method of Cooper, Bredehoeft and Papadopulos (Ref. 2.4-94) yielded a storage coefficient of about 0.10, indicating the groundwater there is unconfined.

The total porosity of the Pleistocene deposits was estimated from laboratory values for dry density obtained on relatively undisturbed soil samples. The equation used was:

$$n = 1 - \rho_B / \rho_s$$

where:

n is the porosity

ρ_B is the bulk density (dry density) in g/cm^3, and

ρ_s is the particle density assumed to be 2.65 g/cm^3.

Dry density values were obtained for 29 samples taken in the depth range 22 to 75 feet from 16 different borings onsite. The dry density values ranged from 1.57 to 2.31 g/cm^3 and the median value was 1.72 g/cm^3. The corresponding range of total porosity values was 0.13 to 0.41 with a median of 0.35.

The effective porosity (n_e) of the Pleistocene deposits was estimated by applying a factor of 0.90 to the total porosity values. This factor was derived from the results of column studies performed on sandy and loamy soils in which non-reactive tracers were used to estimate the actual fraction of the total porosity that was effective in transporting an aqueous solution through the column (Ref. 2.4-86). The column studies indicated a ratio of effective to total porosity ranging from 0.87 to 0.96. The value of 0.90 was selected as an average value for the saturated Pleistocene deposits at the site. Applying this factor to the foregoing total porosity values, we obtain an estimated effective porosity range of 0.12 to 0.37 with a median of 0.32.

Regarding the Mahantango siltsone, it is difficult, if not impossible, to accurately determine values for the total and effective fracture porosity that are representative of the upper bedrock over a distance of hundreds of feet at the site. Examination of the boring logs from this and previous

investigations does, however, provide a basis for making a rough estimate of fracture porosity. In general, the spacing between natural joints and fractures in the Manhantango at the site is in the range of 2 inches to 2 feet or more. Assuming that the average width of opening of a fracture is 0.05 inches, the range of fracture porosity would be about 0.002 to 0.025. In general, the higher porosity figure would apply particularly to the upper foot or two of rock.

The velocity (μ) of groundwater movement in the Pleistocene deposits can be estimated from Darcy's law:

$$\mu = \frac{K_h i}{n_e}$$

where

- K_h *is the horizontal hydraulic conductivity*
- i *is the hydraulic gradient, and*
- n_e *is the effective porosity*

Over the reach of the groundwater flow path from the plant to observation Well 1210, the estimated average gradient is 0.068. Based on the value of 8 feet/day for K_h, and the median value of 0.32 for n_e, an average velocity of 1.7 feet/day was obtained. Between observation Well 1210 and the river, the average hydraulic gradient is 0.0073 and the estimated horizontal hydraulic conductivity is 70 feet/day. The estimated average velocity of groundwater flow for this reach is then 1.6 feet/day.

Preliminary evaluation of the hydraulic gradient of groundwater in the upper bedrock based on bedrock Wells 1201 and l209A, indicates the gradient is close to that for the overburden for the same reaches of groundwater flow. Thus, comparison of groundwater velocities in the two domains can be made on the basis of the ratio K_h/n_e. Derived from field test data, K_h for the upper bedrock is conservatively taken as 0.50 feet/day, while effective porosity is assumed to be 0.02. Thus, K_h/n_e = 25 feet/day. For the overburden, in the reach of the groundwater path from the plant to observation well 1210, K_h/n_e = 8.0/0.32 = 25 feet/day, while over the reach from Well 1210 to the river, K_h/n_e = 70/0.32 = 219 feet/day. Thus, groundwater velocities in the upper bedrock are expected to be approximately the same as those in the overburden over the portion of the flow path from the plant to the western edge of the flood plain. But beneath the flood plain, velocities in the overburden are estimated to be nearly 10 times greater than those in the upper bedrock.

There is virtually no possibility for the groundwater gradient of the overburden or bedrock in the upland area to be reversed. The slope of the underlying bedrock surface is so steep and it controls the groundwater flow so completely, that no condition could conceivably develop that would alter the direction of flow. On the flood plain, the gradient toward the river could be reversed for short periods when the river is in flood stage; and this reversal would no doubt occur only over a few hundred feet west of the river. An example of such a temporary and localized reversal is shown on Figure 2.4-32 (on September 20, 1977, the water level was 1.5 feet higher at Well B-1 than at Well 2). No long-term flow reversal of this type is likely unless an impoundment of the river were effected downstream. In such a case, the entire flood plain area would probably be inundated, and groundwater from the upland area would discharge directly into the reservoir.

Pumpage from regional wells or wells in the vicinity of the site is unlikely to have any effect on groundwater levels or quality in the station area. Pumpage in the Mahantango Formation will probably not affect groundwater levels in the plant area because of the generally low yields of such

wells, and consequently, the limited area of influence of such pumpage. As discussed previously, the primary aquifer on site is the saturated Pleistocene deposits located within the major and secondary bedrock valleys. These materials are recharged almost entirely within the PP&L property, and they drain directly toward the river. Thus, they are largely isolated from the effects of the pumpage offsite. Local pumpage in the Pleistocene deposits is low.

As shown in Figure 2.4-37 and in Table 2.4-22, existing wells in the flood plain within a two-mile radius of the plant are used for domestic or small commercial requirements. The estimated current groundwater withdrawal within a two-mile radius is extremely low -- 56,000 gpd. There are no plans for large increases in the level of groundwater pumpage in the future near the property boundaries.

END HISTORICAL

2.4.13.3 Accidents Effects

This Subsection describes the potential effect on groundwater quality of an accidental release of liquid radwaste at the Susquehanna SES.

2.4.13.3.1 Postulated Accident and Potential Flow Paths

The postulated accident to be analyzed is a rupture of one of the two Reactor Water Clean-Up (RWCU) Phase Separator Tanks, which are located in the Radwaste Building at the far northwest corner of the building. The bottoms of the tanks rest on a reinforced concrete slab at elevation 646 feet, approximately 30 feet below the original land surface. The tanks are each ten feet high and approximately 11 feet in diameter, with a total capacity of 7,400 gallons and an assumed fluid volume of 5,920 gallons (80% of tank volume). The two tanks are used to collect backwash sludge from the fuel pool and RWCU demineralizer systems. The tanks are alternated at 12-month intervals, each tank being in the sludge-collection mode for 12 months, and then at rest for 12 months to allow radioactive decay of isotopes with short half lives. Table 2.4-35 provides the expected content of those radionuclides which are a potential concern from a safety and environmental point of view, and which will be evaluated in this section; Mn-54, Fe-55, Co-60, Sr-90, I-131, Cs-137 and Pu-239.

The bottoms of the RWCU Phase Separator tanks are located approximately 14 feet below the top of the original bedrock surface. Boring log information indicates that at this location the upper 15 feet of bedrock is moderately fractured siltstone with some slickensides. It grades to massive below this level. As shown in Table 2.4-34, packer tests performed in a nearby boring (No. 305) reveal that the upper 12 feet of bedrock is nearly ten times as permeable as the underlying 40-foot interval (Ref. 2.4-75). Overlying the bedrock, before the excavation took place, was approximately 18 feet of Pleistocene deposits, consisting, at the bottom, of sandy gravel with cobbles and boulders. The position of the water table in this location was approximately at the bedrock surface plus or minus two feet.

A complete and instantaneous rupture of one of the RWCU Phase Separator tanks, the bottom slab and the adjacent wall of the Radwaste Building is postulated. The liquid contents of the tank would

seep out into the zone of coarse rock fill surrounding the Radwaste Building and thence into the upper 10 to 15 feet of fractured bedrock.

The postulated groundwater flow paths (Flowpaths 1 and 2) taken by the groundwater contaminated by the slug of radioactive solution are shown in Dwg. FF62005, Sh. 1. Flowpath 1 is initially toward the north and then follows the east-west valley toward the east to the Susquehanna River. Flowpath 2 parallels Flowpath 1 on the south, but then merges with Flowpath 1 at a point in the stream in the north valley just east of the railroad tracks. A hydrogeologic cross-section along Flowpath 2 to the discharge point at the river is presented in Figure 2.4-33. The selection of the two flow paths was based in part on the groundwater contours shown in Figure 2.4-50 and in part on the top-of-bedrock contours shown in Figure 2.4-42. No wells, other than those owned by PP&L on its property, occur anywhere near the two flow paths.

The possibility of the slug of contaminated liquid following a third flow path to the closest offsite private well was considered. The well is located about 2,300 feet southeast of the RWCU Phase Separator Tanks. However, such a flow path is quite unlikely as the top-of-bedrock contours shown on Fig. 2.4-42 indicate that the pathway would be at least cross-gradient and possibly against the gradient of the top-of-rock surface. It is reasonable to assume that the bottom surface of the highly fractured rock zone making up the top 10 to 15 feet of the bedrock closely reflects the top-of-rock surface shown on Figure 2.4-42. Flow from the RWCU Phase Separator Tanks is far more likely to take the easier course along Flowpath 1, or possibly Flowpath 2.

To reflect the differing hydrogeologic properties along different portions of the two flow paths, they were divided into segments:

FLOWPATH 1

Segment No.	Description
1	RWCU tank north to buried valley
2	Along buried valley to Well TW-2
3	Well TW-2 to stream just east of RR tracks
4	From point in stream to Lake Took-A-While
5	From Lake Took-A-While to River

FLOWPATH 2

Segment No.	Description
1	RWCU tank east to north stream just east of RR tracks
2	From point in stream to Lake Took-A-While
3	From Lake Took-A-While to River

In Flowpath 1, the contaminated slug would migrate northward through Segment 1 in the fractures of the upper 10 to 15 feet of bedrock until it reached the east-west oriented buried valley aquifer located about 800 feet north of the Radwaste Building. At this point it would enter the Pleistocene deposits of the aquifer, and would follow Segment 2 eastward to Well TW-2. From Well TW-2 it would migrate over Segments 3 and 4 eastward through shallower Pleistocene deposits constrained within a narrow valley to Lake-Took-A-While. The last

segment of the flow path to the river would be through the deeper and more permeable Pleistocene deposits beneath the flood plain.

In Flowpath 2, the slug of contaminated water would flow eastward through Segment 1 in the fractures of the upper bedrock. In the vicinity of Boring 348, the bedrock surface dips sharply toward the northeast as shown on Figures 2.4-33 and 2.4-42. At this point, beneath the stream in the north valley just east of the railroad tracks, the slug would emerge from the bedrock into the shallow Pleistocene deposits. The remaining two segments (Segments 2 and 3) of Flowpath 2 are identical to Segments 4 and 5 of Flowpath 1.

2.4.13.3.2 Description of the Models Used

Both a contaminant transport analytical model and a flow and transport numerical model were used in the analysis. The analytical model used was SLUG3D, which was previously certified and utilized in the initial version of this section of the SSES FSAR. The flow and transport numerical model used was the combination of the public-domain codes MODFLOW and MT3D96, as implemented in version 2.2 of the ground-water modeling system, Visual Modflow (Refs. 2.4-86a through 2.4-86c).

SLUG3D, simulating the movement of dissolved solutes in groundwater, was used to predict the likely migration of the radionuclides over the entire lengths of Flowpaths 1 and 2 assuming no effect of the pumping of the station's main supply well TW-2. The combination of the MODFLOW and MT3D96 model was used to simulate the effects of the continuous pumping of Well TW-2 on the fate of the radionuclides passing through the buried-valley aquifer in Flowpath 1. The domain of this finite-difference model was the entire buried-valley aquifer lying north of the plant, which is 3,100 feet long (east to west) with an average width of approximately 500 feet.

The factors affecting solute movement that were incorporated into both models include: the natural downgradient movement of the groundwater, hydrodynamic dispersion of the solutes due to the range of pore-water velocities in the formations, adsorption of cations on the clay minerals present in the Pleistocene deposits, and the decay of radioisotopes with time. One of the assumptions of the SLUG3D model is that solutes can disperse freely in all directions, the extent of dispersion limited only by the magnitude of the velocities and the dispersivity assigned for each dimension. The dispersion results in dilution by mixing in three-dimensional space with native groundwater, assumed to be initially free of the particular isotopes. Recharge of an unconfined aquifer by rainfall increases the saturated thickness of the aquifer and, thus, can provide increased dilution capability. This process was directly incorporated into the numerical simulation in Visual Modflow.

The equation utilized in SLUG3D was derived by the integration over the volume of the slug, of the equation for the instantaneous introduction of a slug having an infinitesimally small volume (Ref. 2.4-87a through Ref. 2.4-87e):

$$C = \frac{mR_f}{n(4\pi D'_X t)^{1/2}\,(4\pi D'_Y t)^{1/2}\,(4\pi D'_Z t)^{1/2}} \exp - \{\frac{(x - u'_X t)^2}{4D'_X t} + \frac{y^2}{4D'_Y t} + \frac{z^2 +}{4D'_Z t} + \lambda_i t\} \qquad (1)$$

for the case where $u_y = u_z = 0$, where,
point of interest (cm)

y	=	Distance horizontally and normal to flow from the centerline of the flowpath (cm)
z	=	Distance vertically and normal to flow from the centerline of the flowpath (cm)
λ_i	=	Decay coefficient = .693/ $T^{1/2}$, where $T^{1/2}$ is the radionuclide half-life in seconds (Sec^{-1})
t	=	Time since introduction of slug of liquid (Sec)
$u\rho_x$	=	The average velocity of the radionuclide in the x direction (cm/sec)
$u\rho_x$	=	$(R_f)(u_x)$, where:
u_x	=	see page velocity in the x direction, (cm/sec)
R_f	=	the reduction factor due to absorption or cation exchange (dimensionless)

$$1+(\rho_\beta / n)K_d \qquad \text{(Ref. 2.4-88)}$$

where,

ρ_β	=	bulk density of the aquifer (gm/ml),
K_d	=	Distribution coefficient of the radiouclide on the aquifer material (ml/gm)
	=	(Q!/c)(E), where
Q!	=	concentration of native cations absorbed on the exchange complex of the aquifer materials (milli-equivalents/g) or (meq/g)
c	=	total concentration of native cations in the groundwater at equilibrium (meq/ml)
E	=	equilibrium exchange constant for radionuclide cation displacing native cations on the exchange complex
$D\rho_x$	=	reduced dispersion coefficient in the x direction
	=	D_xR_f (Ref. 2.4-89)
$D\rho_y$	=	reduced dispersion coefficient in the y direction
	=	D_yR_f, and

$D\rho_z$ = reduced dispersion coefficient in the z direction
= D_zR_f,

where,

$D\rho_x$ $D\rho_y$, and $D\rho_z$ are the dispersion coefficients in the x, y, and z directions, respectively, and

D_x = $(I_L)(U_x)$

D_y = $(I_T)(u_y)$

D_z = $(I_v)(u_z)$, where I_L, I_T and I_V are, respectively, the dispersivities in feet in the longitudinal (x) direction), in the horizontal transverse (y) direction and in the vertical (z) transverse direction.

To obtain an expression for the concentration of radionuclides introduced into the groundwater as a finite prismatic volume, at any point (x, y, z) down gradient of the slug origin, equation (1) is integrated with respect to x, y, and z, over the limits $-x_o/2$ to $x_o/2$, $-y_o/2$ to $y_o/2$ and $-z_o/2$ to $+z_o/2$, respectively. Here x_o, y_o and z_o are the dimensions of the slug in the groundwater along the respective axes at time t_o=0, and x, y, and z are measured from the center of the prismatic volume of the slug. The resulting expression is given as Equation (2):

$$C = \frac{C_o R_f}{8}\left\{ erf\left(\frac{x + x_o/2 - u'_x t}{\sqrt{4D'_x t}}\right) - erf\left(\frac{x - x_o/2 - u'_x t}{\sqrt{4D'_x t}}\right)\right\}$$

$$\bullet \left\{ erf\left(\frac{y + y_o/2}{\sqrt{4D'_y t}}\right) - erf\left(\frac{y - y_o/2}{\sqrt{4D'_y t}}\right)\right\}$$

$$\bullet \left\{ erf\left(\frac{z + z_o/2}{\sqrt{4D'_z t}}\right) - erf\left(\frac{z - z_o/2}{\sqrt{4D'_z t}}\right)\right\}\left\{ \exp(-\lambda_i t)\right\}$$

Where: C_o = the initial concentration in the interstitial liquid in the slug
= $m/nx_oy_oz_o$

This equation was derived for the case of a slug introduced instantaneously into a saturated porous medium, where the slug has a finite volume at t = 0. The inclusion of the factor, exp $(-\lambda_i t)$, implies that radionuclide decay is accounted for in the calculated concentration (c). SLUG3D was used to

calculate the values of concentration at the particular points of interest over the range of time during which the peak occurs.

2.4.13.3.3 Selection of Parameters for SLUG3D Simulations

Conservative parameter values were selected from the range of values determined by field and laboratory tests and from a review of the literature. A summary of the values selected and used in the analysis is given in Table 2.4-36. A description of the parameter-value selection follows.

A sensitivity analysis was previously performed to evaluate what parameter value in each case would yield the highest computed concentration. For all ranges of isotope half life, the results were the same wherever the cation exchange capacity was assumed to be zero. Highest concentration values resulted as the initial slug length approached twice the width, as the total and effective porosities and the dispersion coefficients decreased, and as the flux rate increased. Flux rate is defined as the product of horizontal hydraulic conductivity (K_h) and hydraulic gradient (i). With the cation exchange capacity equal to 0.016 meq/ml, highest concentration values resulted as the initial length of the slug approached twice the width as effective porosity (n_e) decreased as the dispersion coefficients decreased as cation exchange capacity (Q) decreased as the exchange constant (E) decreased as total porosity (n) increased as flux rate increased and as the cation concentration increased.

a. Distance to Discharge Points (x)

As described in Subsection 2.4.13.3.1, the postulated flow paths are shown on Dwg. FF62005, Sh. 1. The calculated distances follow:

Flowpath 1

Flow Path Segment	Description	Distance (x)(feet)
1	RWCU tank north to buried valley	805
1a	RWCU tank north to buried valley model edge	680
2	Along buried valley to Well TW-2	725
3	Well TW-2 to stream just east of RR tracks	860
4	From point in stream to Lake Took-A-While	1,420
5	From Lake Took-A-While to River	1,720

Flowpath 2

Flow Path Segment	Description	Distance (x)(feet)
1	RWCU tank east to stream just east of RR tracks	1,865
2	From point in stream to Lake Took-A-While	1,420
3	Lake Took-A-While to River	1,720

b. Horizontal Hydraulic Conductivity (K_h)

As described in Subsection 2.4.13.2.3, different values of hydraulic conductivity characterize the different segments of the flow paths. Based on the pumping tests and packer tests performed in borings or wells located on or close to the flow path, the following conservative assignment of values has been made:

Flowpath 1

Flow Path Segment	Geologic Unit	Horizontal Hydraulic Conductivity (K_h)
1	Upper 15 ft of bedrock	0.5 ft/day
2	Lower Pleistocene deposits	18.0 ft/day
3	Lower Pleistocene deposits	8.0 ft/day
4	Lower Pleistocene deposits	20.0 ft/day
5	Lower Pleistocene deposits	60.0 ft/day

Flowpath 2

Flow Path Segment	Geologic Unit	Horizontal Hydraulic Conductivity (K_h)
1	Upper 15 ft of bedrock	0.5 ft/day
2	Lower Pleistocene deposits	20.0 ft/day
3	Lower Pleistocene deposits	60.0 ft/day

c. Hydraulic Gradients (i)

Hydraulic gradients were estimated based on the groundwater contours in October 1985 and on a river-level elevation of 491 feet, the latter serving as the groundwater elevation at the discharge point on the Susquehanna River. The elevation of the water table in the vicinity of the RWCU Phase Separator tank was taken as elevation 662 feet, based on water level readings taken in Boring 305 and Boring 116.

The magnitude of water-level elevations at the intermediate points (edge of buried-valley aquifer, Well TW-2, downgradient edge of buried-valley aquifer, and Lake Took-A-While) were taken from the groundwater level data for October 1985 shown on Dwg. FF62005, Sh. 1. The calculated gradients follow:

FLOWPATH 1

Flow Path Segment	Hydraulic Gradient (I) (Dimensionless)
1	0.060
2	0.024
3	0.042
4	0.0388
5	0.0081

FLOWPATH 2

Flow Path Segment	Hydraulic Gradient (I) (Dimensionless)
1	0.055
2	0.0388
3	0.0081

d. Total Porosity (n)

As discussed in Subsection 2.4.13.2.3, based on 29 formation samples obtained from onsite borings, the values of total porosity of the more permeable Pleistocene deposits range between 0.13 and 0.41 with a median of 0.35. Using data from samples taken from borings in the vicinity of the groundwater flow paths, an average value of 0.30 has been selected for the porosity of the lower saturated Pleistocene deposits for the two flow paths. For the first segment of both Flowpaths 1 and 2 through the upper bedrock, a relatively high horizontal hydraulic conductivity (0.50 feet/day) has been estimated for the upper bedrock. Because of this, a relatively high fracture porosity of 0.02 seems justified for the upper bedrock, particularly as the upper 11 feet of bedrock at Boring 305 (located close to the Radwaste Building) was described as severely fractured.

e. Effective Porosity (n_e)

As described in Subsection 2.4.13.2.3, the effective porosity for the Pleistocene deposits was obtained by multiplying the total porosity by a factor of 0.90. This factor is an approximate ratio of effective to total porosity for such materials, based on column studies using non-reactive tracers (Ref. 2.4-86d). Thus, for the Pleistocene deposits, effective porosity is estimated as 0.30 x 0.90, or 0.27. For segment 1 of the flow path, the effective porosity of the fractured bedrock is taken as equal to the estimated total fracture porosity.

f. Dispersivities (α_L, α_T, α_v)

The dispersivities in the longitudinal (x) direction, in the horizontal transverse (y) direction, and in the vertical (z) transverse direction are denoted, respectively, by α_L, α_T, and α_v. Table 4.1.2.2 of Reference 2.4-90 shows values for longitudinal and transverse dispersivities determined at different sites with different rock or sediment materials. Based on this information, the following conservative values for dispersivity have been estimated for the site:

	α_L (ft)	α_T (ft)	α_v (ft)
Upland Flow Paths	10.0	0.5	0.001
Flood Plain	30.0	2.0	0.05

g. Size and Dimensions of the Slug

The volume of liquid involved in the accident is 5,920 gallons or 791.34 cubic feet. It was assumed that the slug would have a thickness of 8 feet in the rock immediately, or very soon after, the rupture occurred. Then, assuming the slug would initially be square

in plan view, and taking 0.02 as the upper bedrock total porosity, the initial x_0 (or y_0) dimension of the slug in the ground would be {791.34 cu. ft./[(8 ft.)(0.02)]}$^{1/2}$ or, 70.3 feet.

h. Radionuclides

Seven radionuclides were selected for analysis--Mn-54, Fe-55, Co-60, Sr-90, I-131, Cs-137 and Pu-239. These are believed to be the most significant from the safety and public health point of view and those with half lives on the order of days or years. Table 2.4-35 indicates the radionuclides studied, their half lives and their presumed initial concentrations at the time of the accident.

i. Distribution Coefficients (k_d)

The distribution coefficient (k_d) defines the equilibrium ratio of the mass of a cationic species adsorbed on the exchange complex of geologic materials to that in the interstitial solution. The concentration of native cations adsorbed on the formation material (Q'), concentration of cations in the interstitial fluid (c), and the equilibrium exchange constant (E) for each species are the parameters uniquely involved in the process of adsorption through cation exchange and are related to the distribution coefficient by:

$$k_d = (Q'/c)(E)$$

Considering that the concentration of native cations adsorbed on the formation material would be large compared to that of radionuclide cations of concern here, for purposes of this analysis we have approximated Q' (concentration of adsorbed native cations on the formation) by Q, the total cation exchange capacity. Thus, we have

$$k_d = (Q/c)(E)$$

It is recognized that this model of cation exchange will not strictly imitate the actual cation exchange process that would follow the postulated accident which would simultaneously involve many radionuclide cations from the RWCU Phase Separator tank. However, to minimize the complexity of the adsorption model it was decided to use this equation to model the process assuming a simple binary system of calcium-radionuclide cation and using the native groundwater cation concentration at the site. For conservatism, calcium was chosen as the native cation in solution in the groundwater rather than sodium, as calcium can displace strontium or cesium much more readily than can sodium.

Cation exchange capacity determinations were performed previously on ten soil samples obtained from the lower part of the Pleistocene deposits, in borings, located on or very near the groundwater flow paths. Values ranged from 0.006 to 0.05 milliequivalents/gram (meq/g).

For Segment 1 of both Flowpaths 1 and 2, the cation exchange capacity for the fractured bedrock was assumed to be zero. For the remainder of the two flow paths, occurring in the Pleistocene deposits, estimated Q values ranged from 0.016 to 0.026 meq/g, based

on the values obtained for the borings located along the flow paths. An average value for Q for migration through these deposits of 0.021 meq/g was adopted.

Water samples were obtained from observation wells onsite in three different seasons during 1977. Total cation concentration for overburden wells in the vicinity of the flow paths ranged from 7.4×10^{-4} meq/ml to 3.7×10^{-3} meq/ml. A value for C is unnecessary for the analysis of flow in Segment 1 of both Flowpaths 1 and 2, as the cation exchange capacity there is assumed to be zero. It was concluded that the average cation concentration in the groundwater in the Pleistocene deposits along the flow paths ranges from 2.1×10^{-3} to 3.7×10^{-3} meq/ml. An average value for C of 3.0×10^{-3} meq/ml was selected for each segment of the flow paths passing through these deposits.

Considering the case of Strontium-90 first, a simple binary system of Ca-Sr was assumed. Accordingly, an equilibrium exchange constant (E_{Sr}) was estimated from the literature for this condition. An E value of 1.0 was obtained for Strontium-89 and Strontium-90 from Reference 2.4-95a. Thus, the k^d value for Sr-90 in the Pleistocene deposits was calculated to be:

$$(0.021 \text{ meq/g})\ (1.0)\ /\ (0.003 \text{ meq/ml}) = 7 \text{ ml/g}$$

Reference 2.4-95b indicates that the k_d for cesium on less than 100-µ fraction of sediments from the Savannah River Plant ranged from 18.3 to 130 ml/g. For this analysis, a k_d of 18 ml/g for Cs-137 was selected, which was the lowest of this range.

For Cobalt-60, the estimate for the k_d was based on Table 1.11 of Reference 2.4-96a. In the table, the ratio [k_d(Co-60)]/[k_d(Cs-137)] for adsorption at pH 6.0 on Clinch River Sediment in Tennessee ranged from 0.56 to 0.81. Applying the lowest ratio (0.56) to the estimated k_d for Cs-137 of 18 ml/g, we obtain 10.0 ml/g for Co-60.

For Manganese-54, Appendix B of Reference 2.4-96b indicates that the ratio of Mn-54 adsorbed to large colloids (pre-filter) to that of Co-60 ranged from 0.87 to 2.04. For this analysis, the lowest ratio value, 0.87, was applied to the k_d value estimated for Co-60 of 10 ml/g, to obtain an estimated k_d of 8.7 ml/g for Mn-54.

For Iron-55, Reference 2.4-96c indicates typical k_d values for radioisotopes in desert soils. Relatively high k_d values are given for Strontium, Cesium and Cobalt: 20, 200, and 75 ml/g, respectively. The value given for Iron is 150 ml/g. Using the principal of proportionality based on the values selected in the foregoing paragraphs, the estimated k_d's for Fe-55 range from 13.5 to 52.5 ml/g. For this analysis, the lowest of these values, or 13.5 ml/g is selected for Fe-55.

No k_d values were estimated for Iodine-131 or Plutonium-239. I-131 has a half life of only 8.04 days and its concentration would be expected to decline below the maximum permissible levels in a period less than the travel time to possible receptors. Because the concentration of Pu-239 in the RWCU Phase Separator tanks, as shown in Table 2.4-35, is only about ten times the limits for unrestricted areas, natural attenuation through dispersion in the course of the migration would be expected to reduce its concentration to acceptable levels without assuming adsorption due to cation exchange.

j. Calculation of Mean Parameter Values to Each Receptor

Because the SLUG3D program provides an analytical solution for the case where the groundwater parameters are assumed constant over the entire flow path, it was necessary to calculate a weighted average for each parameter based on the values from each flow-path segment. The parameter values for each segment of the flow-path were weighted on the basis of the segment length or the time for water to travel over the segment, whichever was most appropriate. Time of travel was the weighting factor in determining the mean horizontal hydraulic conductivity (K_h) and the mean effective porosity (n_e), while the segment length was the weighting factor to determine the mean gradient, the mean dispersivities and the mean k_d values.

2.4.13.3.4 Numerical Model Simulation of Buried-Valley Aquifer

The combination of the MODFLOW and MT3D96 model was used to simulate the effects of the continuous pumping of Well TW–2 on the fate of the radionuclides passing through the buried-valley aquifer along Flowpath 1. The domain of this finite-difference model was the entire buried-valley aquifer lying north of the plant, which is 3,100 feet long (east to west) with an average width of approximately 500 feet.

The aquifer was modeled as a single-layer water-table aquifer, and a finite-difference grid 3,100 feet east to west and 1,000 feet north to south was established. The grid consisted of 36 rows and 13 columns with cell widths ranging from 50 to 100 feet. The finer grid was set up for the area around Well TW-2 located on the eastern side of the grid domain. A series of cells on the northern and southern sides of the grid were set to be inactive so that the boundaries of the aquifer could be approximated as closely as possible. Constant-head boundaries were established at the eastern and western edges of the model. These head values were based on interpolated head values from the October 1985 water-table contour map (Dwg. FF62005, Sh. 1).

The flow model, using MODFLOW, was calibrated under steady-state conditions on the basis of the October 1985 static water-level data and it was verified, or the calibration task was completed, by transient simulations of a 7.11-day pumping test of Well TW-2 performed in December 1992. In the process of calibration, recharge and hydraulic conductivity values were adjusted in several different zones making up the model until satisfactory matching to the measured water levels resulted. Hydraulic conductivity values in the final calibrated model ranged from 4 to 50 ft/day, with the zone in the immediate vicinity of Well TW-2 having a K_h of 18 ft/day. Recharge across the final calibrated model ranged from 0.0003 to 0.0085 ft/day. For the transient calibration, the specific yield, representing the primary storage coefficient, was 0.20.

The calibrated flow model was run in the steady-state mode with Well TW-2 pumping at a series of different rates to determine the maximum long-term pumping rate possible. The maximum pumping rate was found to be 33 gallons per minute (gpm). It was decided to perform the simulations in which MT3D was to be coupled by having TW-2 pump continuously at a conservative rate of 31 gpm (average daily pump rate).

The MT3D component in Visual Modflow was run in conjunction with the steady-state flow model by specifying a number of transport steps. A constant-concentration boundary cell serving as the contaminant source was established at cell (17,8), which was located approximately 680 feet north of the RWCU Phase Separator tanks and along Flowpath 1. This was the edge of the modeled buried-valley aquifer.

A simulation with SLUG3D was employed to evaluate the transport of the slug of contaminated fluid from the tank to cell (17,8) of the numerical model. The duration of the appearance of the contaminants at that point lasted, for all intents and purposes, no longer than 500 days. Based on the results of the SLUG3D simulation, an average concentration for the 500 days for each of the seven radionuclides of concern was computed taking into consideration the rise and decline of concentration with time and the decline of concentration with distance from the center of the slug. These mean concentrations averaged over time and space are given in Table 2.4-37, which presents the parameter values used in the numerical model. The assumed values for dispersivities and k_d's, consistent with the values given in Section 2.4.13.3.3, are also included in Table 2.4-37.

Simulations with the MODFLOW/MT3D model were performed for all the radionuclides of concern except for I-131, which was omitted because the SLUG3D simulation showed that its peak concentration at cell (17,8) was well below the effluent concentration limits for unrestricted use. MT3D was run primarily using the upstream finite-difference option, although in most cases parallel runs were made using the Method of Characteristics (MOC) option as well. The finite-difference method was preferred because it was more stable, it had much lower mass balance errors, and it gave peak concentration values comparable to the MOC method.

An observation point was established in the cell where Well TW-2 was located and in the cell at the far downstream boundary of the model. Concentrations were obtained over time at these observation points, and the transport simulations were run in each case well past the time of the peak concentration for each radionuclide. As shown in Table 2.4-39, for those isotopes for which adsorption was included in the model (Mn-54, Fe-55, Co-60, Sr-90, and Cs-137) the computed time to peak at Well TW-2 ranged from 3,100 to 19,140 days and that for the most downgradient cell 4,050 to 92,110 days. Plutonium-239, for which no adsorption was assumed, was computed to peak at 708 days at Well TW-2 and at 1,740 days at the farthest downgradient cell.

2.4.13.3.5 Discussion of Results of Analysis

The results of the groundwater transport simulations are presented in Tables 2.4-38 and 2.4-39. Table 2.4-38 presents the results of the SLUG3D simulations for the two probable flow paths. Of the radionuclides evaluated the peak concentrations for Sr-90 and Pu-239 are predicted to exceed the 10 CFR 20 Appendix B effluent concentrations values at the entry point to the Susquehanna River via either flow path. The peak concentrations for the other isotopes evaluated are predicted to be well within the regulatory limits prior to discharging into the river.

Table 2.4-39 provides the results of the simulations with the numerical model of the buried-valley aquifer based on MODFLOW and MT3D. All of these simulations include steady state pumping of Well TW-2 at 31 gpm, which was not simulated in the SLUG3D runs. The results of this analysis indicate that most of the contaminated water can be expected to be captured by

the onsite TW-2 Well and that of the radionuclides studied, the longer half-life isotopes, Co-60, Sr-90, Cs-137, and Pu-239 would exceed their respective effluent concentration limits. However, at the down-gradient boundary of the aquifer, all isotopes are found to be below their effluent concentration limits for unrestricted or public use.

Since Well TW-2 is on site, access to the use of this water can be restricted if post accident well-water monitoring determines that the radioactive content exceeds the 10CFR 20, Appendix B, Table 1 occupational does limits. As indicated in Subsection 2.4.12.3 the nearest potable water system that utilizes the waters of the Susquehanna River is at Danville, approximately 31 miles downstream of the plant. Dilution due to mixing of the contaminated groundwater with the river water is expected to be in excess of a factor of 650 at Danville. Thus at the nearest potable water system the concentration of all significant radionuclides will be significantly below the effluent concentrations limits for unrestricted public use.

It is believed that flow of groundwater from the RWCU Phase Separator Tanks is more likely to occur along Flowpath 1 rather than Flowpath 2. The bedrock contours shown on Fig. 2.4-42 lend credence to this belief. Thus, if the most likely scenario is for the slug of contaminated fluid to migrate along Flowpath 1 into and through the buried-valley aquifer, then the influence of the constant pumping of Well TW-2 is seen to have a profound effect on the concentrations of radionuclides migrating downgradient of the aquifer through the sediment-filled narrow valley toward the Susquehanna River. Plant records indicate that Well TW-2, sometimes in combination with nearby Well TW-1, withdraws an average of 30,000 to 45,000 gallons a day from the buried-valley aquifer, or an average of 21 to 31 gpm. Should this constant withdrawal be maintained during the months and years following the postulated accident, it should significantly enhance the attenuation of the radionuclides when they appear at the Susquehanna River.

2.4.13.4 Design Bases for Subsurface Hydrostatic Loadings

Plant safety-related structures were designed assuming subsurface hydrostatic loadings caused by a groundwater table at an elevation of 665 ft. This is higher than the expected maximum water table because the groundwater configuration in the region is primarily controlled by topography. At the site the natural topography has been modified by plant excavations. The modified topography in the vicinity of safety-related structures restricts the maximum elevation of the water table to approximately 660 ft. Groundwater levels are further reduced due to a decrease of the effective recharge area by placement of plant structures. These structures and paved other areas intercept rainfall and divert it, reducing infiltration in that area.

At the spray pond, a liner has been designed to restrict the seepage rate from the pond to limit buildup of a groundwater mound in the glacial materials underlying the pond. The pond has been designed for a maximum groundwater elevation of 665 ft. Detailed description of design criteria for control of groundwater levels and seepage at the spray pond, and the stability of the pond, are found in Subsection 2.5.5.

2.4.14 TECHNICAL SPECIFICATION AND EMERGENCY OPERATION REQUIREMENTS

The possibility of adverse hydrologically-related events at the plant site is precluded due to the configuration of the plant site topography. Consequently, there are no emergency protective

measures designed to minimize the water associated impact of adverse hydrologically-related events on safety-related facilities. In addition, there is no need for technical specifications for plant shutdown required by accidents resulting from these events. Further discussion may be found in Subsections 2.4.1.1 and 2.4.2.2.

The ultimate heat sink, as described in Subsections 2.4.8 and 9.2.7, has been designed with appropriate consideration to adverse hydrologically-related events.

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Table 2.4-1

HISTORICAL INFORMATION

EXISTING AND PROPOSED DAMS LOCATED IN THE SUSQUEHANNA RIVER BASIN

Security-Related Information Table Withheld Under 10 CFR 2.390

Table 2.4-2

HISTORICAL INFORMATION

MINOR UPSTREAM DAMS AND RESERVOIRS

Security-Related Information
Table Withheld Under 10 CFR 2.390

Table 2.4-3

HISTORICAL INFORMATION

WATER USERS

Security-Related Information

Table Withheld Under 10 CFR 2.390

TABLE 2.4-4

HISTORIC FLOODS IN THE VICINITY OF THE SUSQUEHANNA STEAM ELECTRIC STATION

Flood Date	Danville[a] Elevation[c] (ft-msl)	Danville[a] Discharge (cfs)	Susquehanna SES Elevation[d] (ft-msl)	Susquehanna SES Discharge (cfs)	Wilkes-Barre[b] Elevation[e] (ft-msl)	Wilkes-Barre[b] Discharge (cfs)
June 24, 1972	463.6	363000	516.6	349000	552.7	345000
March 9, 1904[f]	462.0	-	-	-	-	-
Sept. 27, 1975	458.8	257000	510	252000	547.2	251000
March 20, 1936	458.6	250000	510	236000	545.2	232000

(a) 31 Miles Downstream of Susquehanna SES
(b) 22 Miles Upstream of Susquehanna SES
(c) Danville Flood Stage 451.3 Feet, MSL
(d) Susquehanna SES Flood Stage 670 Feet, MSL
(e) Wilkes-Barre Flood Stage 534.1 Feet, MSL
(f) High Stage Due to Ice Jam Flooding

TABLE 2.4-5

ALL-SEASON 24-HOUR PROBABLE MAXIMUM PRECIPITATION

Time (Hour)		Precipitation Increment (Inches)
0 - 9		2.52
9.5		1.24
10.0		1.24
10.5		1.49
11.0		1.74
11.5		1.74
12.0		2.73
12.5		6.70
13.0		1.98
13.5		1.74
14.0		1.49
14.5		1.49
15.0		1.24
15 - 24		2.38
	Total	29.72

TABLE 2.4-6

PROBABLE MAXIMUM PRECIPITATION FOR DURATIONS LESS THAN 30 MINUTES

Duration (Minutes)	Accumulated PMP (Inches)
5	2.48
10	3.82
15	4.82
30	6.70

Table 2.4-7
PEAK RUNOFF RATES FROM AND MAXIMUM PONDING DEPTHS ON ROOFS OF SAFETY RELATED STRUCTURES FOR LOCAL ALL-SEASON PMP

Security-Related Information
Table Withheld Under 10 CFR 2.390

TABLE 2.4-8

ADOPTED 4-HOUR UNIT HYDROGRAPHS AND CHARACTERISTICS

STATION NO.	DRAINAGE AREA (sq. mi.)	R*	Tc*	Cp*	tp**
101	351	48.31	27.99	.423	27.30
102	164	14.79	11.83	.471	11.49
103	118	11.46	16.02	.630	13.42
104	43	2.70	2.00	.423	3.46
105	108	16.14	12.81	.500	13.07
106	102	9.60	8.60	.579	8.00
107	98	5.71	5.62	.499	5.59
108	196	18.46	19.85	.579	18.60
109	94	4.67	7.04	.578	6.22
110	79	5.96	4.04	.426	4.53
111	121	12.00	7.00	.430	9.00
112	28	10.03	2.00	.402	5.30
113	116	7.36	6.03	.479	6.34
114	622	28.30	22.39	.483	20.34
1261	40	7.38	5.95	.477	6.29
115	264	16.04	7.06	.334	7.81
116	58	11.04	2.03	.374	5.76
117	84	11.78	2.03	.364	5.87
118	192	14.78	7.23	.352	7.79
119	36.4	11.31	4.52	.376	6.03
120	27.8	9.91	2.00	.389	5.57
121	231.8	16.69	7.70	.354	8.37
122	255	15.09	16.06	.579	14.48
123	184	24.67	12.17	.357	12.00
124	95	13.60	7.66	.382	7.92
125	64	9.30	3.94	.396	5.44
126	118	7.46	5.99	.470	6.33
127	70	5.32	4.86	.486	4.75
128	113	13.00	7.50	.485	7.11
129	186	11.82	10.10	.405	4.70
130	151	12.10	10.50	.405	8.00
131	370	20.05	14.81	.477	14.19
132	30	4.52	6.15	.539	5.52
133	56	4.48	6.18	.541	5.53
134	96	4.79	6.19	.539	5.68
135	160	6.35	7.22	5.260	6.74
136	114	4.94	5.04	.554	5.43
137	72	3.66	5.82	.510	4.73
138	402	5.59	10.51	.601	8.03
139	298	8.47	6.91	.457	6.95
140	70	6.74	5.08	.460	5.49

TABLE 2.4-8 (Continued)

STATION NO.	DRAINAGE AREA (sq. mi.)	R*	Tc*	Cp*	tp**
141	72	8.85	5.97	.441	6.55
142	66	19.00	5.50	.276	7.05
143	254	35.00	9.06	.258	10.89
144	75	19.20	4.34	.270	6.55
145	77	20.90	2.00	.260	6.52
146	44	9.64	4.93	.410	6.05
147	127	12.61	4.83	.359	6.35
148	80	18.39	9.71	.405	10.42
149	67	10.92	6.40	.396	6.99
150	143	13.00	7.00	.531	6.50
151	227	10.60	7.16	.403	7.26
152	193	16.00	8.40	.738	9.00
153	144	9.60	10.00	.480	9.60
154	150	6.18	5.33	.479	5.51
155	215	8.11	7.68	.469	7.19
156	101	6.48	5.27	.472	5.56
157	223	15.96	13.50	.475	12.06
158	159	14.47	13.87	.505	12.11
159	114	14.00	7.00	.556	6.50
160	114	12.00	8.00	.738	7.70
161	383	2.50	28.00	.860	13.70
163	37	5.44	5.63	.502	5.49
165	41	5.11	5.30	.481	5.02
168	97	6.89	6.43	.498	6.47
171	157	6.94	7.43	.507	6.94
172	116	12.80	8.10	.738	7.90
173	206	11.04	9.93	.505	9.28

*Clark coefficients

**Snyder coefficients

Table 2.4-9

SUSQUEHANNA RIVER BASIN ROUTING COEFFICIENTS

Security-Related Information
Table Withheld Under 10 CFR 2.390

TABLE 2.4-10

MANNING "N" VALUES

COMPUTED FROM 1936 FLOOD

Section	Elevation	River Mile	Reach Length	"n" Channel	"n" Overbank	
1	493.2	157.2	10,050	0.041	0.100	Berwick Bridge
2	497.1	159.0	6,900	0.035	0.100	
3	500.0	160.31	4,400	0.037	0.100	
4	502.0	161.15	8,970	0.032	0.100	
5	506.0	162.85	3,960	0.027	0.100	
6	507.6	163.6	5,808	0.046	0.100	
7	511.0	164.7	7,650	0.052	0.100	
8	514.3	166.15				
Site		165.64				

TABLE 2.4-11

SUSQUEHANNA RIVER FREEZE OVER AT HARRISBURG (1870-1955)

Number Days Frozen	Number of Freeze Overs
1-14	36
15-30	33
31-60	20
61-90	8
91+	1

Month	Number of Years River Frozen
Nov.	2
Dec.	36
Jan.	54
Feb.	49
Mar.	22
Apr.	1

TABLE 2.4-12

ICE JAM FLOODING

Danville (Flood Stage = 20 ft.)

DATE	STAGE	ELEVATION
Jan. 25, 1904	26.2	457.5
Feb. 10, 1904	24.6	455.9
Mar. 9, 1904	30.7	462.0

Wilkes Barre (Flood Stage = 22 ft.)

DATE	STAGE	ELEVATION
Mar. 11, 1893	28.7	540.8
Mar. 3, 1895	27.0	539.1
Jan. 16, 1898	21.8	533.9
Jan. 7, 1899	25.0	537.1
Feb. 9, 1900	17.8	529.9
Mar. 12, 1901	21.5	533.6
Jan. 23, 1902	18.2	530.3

TABLE 2.4-13

SUMMARY OF FLOOD ROUTING STUDIES - SPRAY POND

Case	Condition	Maximum Water Elevation (ft)	Outflow Through Conduit @ ESSW (cfs)	Outflow Over Emergency Spillway* (cfs)	Total Outflow from the Pond* (cfs)
1	Normal operation (10,000 gpm blowdown into pond)	679.0	13.8	0	13.8
2	10,000 gpm blowdown into pond + 50% PMF + PMF 72 hr later	682.3	41	150	191
3	10,000 gpm blowdown into pond + PMF + 50% PMF 72 hr later	682.3	41	150	191
4	10,000 gpm blowdown into pond + 1/2 PMF + 1/2 PMF 72 hr later and failure of discharge conduit at ESSW	681.8	0	105	105
5	10,000 gpm blowdown into pond + 25-yr flood and failure of discharge conduit at ESSWP	681.4	0	65	65
6	10,000 gpm blowdown into pond + 10-yr flood	679.6	33	0	33

* Spillway discharge and total discharge shown to nearest 5 cfs, except for those under normal operating conditions.

TABLE 2.4-14

RESULTS OF WIND WAVE COMPUTATION

Wind speed (mph)	40	65	80
Static water level (ft, msl)	682.3	681.8	679.6
Wave height (ft)			
Significant	0.7	1.3	1.7
1%	1.2	2.2	2.9
Runup elevation from 1% wave (ft, msl)			
at ESSW pumphouse	684.3	684.8	683.3
at side of spray pond	683.8	684.6	683.4

TABLE 2.4-15

MAXIMUM LOADINGS RESULTING FROM WIND-WAVE ACTIVITIES

Structure	Wind Speed (mph)	Static Water Level (El. ft)	1% Wave Height (ft)	Maximum Resultant Forces (kips)	Maximum Moment at Base of Structure (ft-kips)
Pipe Supports	40	682.3	1.2	0.2	2.4
	65	681.8	2.2	0.3	3.0
	80	679.6	2.9	0.5	4.2
ESSW Pumphouse	40	682.3	1.2	33	218
	65	681.8	2.2	62	679
	80	679.6	2.9	76	642

TABLE 2.4-16

MAXIMUM HYDRODYNAMIC LOADING RESULTING FROM EARTHQUAKE

Structure	Earthquake Type	Direction	Maximum Resultant Forces (kips)	Maximum Moment at Base of the Structure (ft-kips)
Pipe Support	SSE	E-W & N-S	0.6	5.0
	OBE	E-W & N-S	0.3	2.4
ESSW Pumphouse	SSE	N-S	263.3	2250
		E-W	140.6	1910
	OBE	N-S	145.7	1271
		E-W	77.5	1062

Table 2.4-18

ESW COOLING DUTY ON SIMULTANEOUS LOSS OF ALL AUXILIARY POWER TO BOTH UNITS

Security-Related Information
Table Withheld Under 10 CFR 2.390

TABLE 2.4-21

REGIONAL HYDROGEOLOGIC SECTION
(within 20-mile radius of Susquehanna SES)

Era	Period	Epoch	Group or Formation	Lithologic Character	Estimated Range of Thickness in Region (ft)	Groundwater Yield Characteristics
Cenozoic	Quanternary	Pleistocene	Stratified glacial drift	Primarily outwash sediments or kame terraces consisting of sand and gravel deposits, with occasional layers of clay.	0 to 300	Yield per well ranges from 6 to 1300 gallons per minute (gpm), with a median yield of 100 gpm. Where sufficient saturated thickness occurs, properly constructed wells should yield more than 250 gpm.
Paleozoic	Pennsylvanian	-	Llewellyn Formation	Sandstone, conglomerate, shale, fire clay, slate and numerous anthracite coal beds.	700 to 2200	Yield per well ranges from 2 to 80 gpm, with a median yield of 10 gpm. Highly acidic water is common because of proximity to coal mining operations.
		-	Pottsville Formation	Generally a hard quartzose unit consisting of gray conglomerate and white, gray or brownish sandstone.	150 to 850	Yield per well ranges from 5 to 160 gpm, with a median yield exceeding 50 gpm.
	Mississippian	Middle Mississippian	Mauch Chunk Formation	Red, green, yellow or brown shale, with some sandstones.	200 to 2000	Yield per well ranges from 4 to 375 gpm, with a median yield of 22 gpm, based on 101 wells for which data were available.
		Lower Mississippian	Pocono Formation	Hard massive gray sandstone and conglomerate, including some shale layers.	600 to more than 1000	Yield per well ranges from 3 to 133 gpm, but the median yield for 8 wells for which data were available is only 6 gpm.

TABLE 2.4-21

REGIONAL HYDROGEOLOGIC SECTION
(within 20-mile radius of Susquehanna SES)

Era	Period	Epoch	Group or Formation	Lithologic Character	Estimated Range of Thickness in Region (ft)	Groundwater Yield Characteristics
Paleozoic	Devonian	Upper Devonian	Catskill Formation	Red to brownish shales, red and gray crossbedded sandstone, and gray to green sandstone tongues.	1000 to 3000	Yield per well ranges from 2 to 325 gpm, and the median yield is 12 gpm.
			Marine beds consisting essentially of Trimmers Rock Formation	Hard gray to greenish-gray massive to flaggy sandstone, containing little shale.	1500 to 3000	Yield per well ranges from 1 to 15 gpm, with a median yield of 5 gpm.
		Middle Devonian	Mahantango Formation	Bluish-gray to brownish sandy shale with interbedded sandstones, and locally thin limestone.	~1100	Yield per well ranges from 2 to 21 gpm in nine wells for which data were available.
			Marcellus Formation	Black, gray or dark blue fissile shale.	~400	
		Middle to Lower Devonian	Onondaga Formation	Non-cherty limestone overlying a gray calcareous shale.	~150	No well yield data are available. Not believed to be high yielding in the region.
	Silurian	Silurian	Keyser Formation	Alternating beds of sandy limestone and calcareous sandstone, some conglomeritic sandstone and a bed of soft shaly limestone.	?	Yield per well ranges from 16 to 250 gpm, with three out of the four wells for which data were available yielding 125 gpm, or more.
			Tonoloway Formation	Platy, laminated and argillaceous limestones, with thick beds occurring locally at the top.	100 to 150	
			Wills Creek Formation	Alternating limestone, limy shales and fissile shales.	~300	
Paleozoic	Silurian	Middle Silurian	Bloomsburg Formation	Dark-red sandy shale, with a few thin layers of bright-green shale, and a few beds of red sandstone.	~800	Yield per well ranges from 5 to 20 gpm. One well is reported to give a "large", although unmeasured supply.
			Clinton Formation	Units of the Clinton Formation in the region consist to a large extent of hard ferriferous red sandstone and yellowish green and olive-green shale.	600 to 700	No wells tapping units of the Clinton Formation are recorded, as they form a high ridge in the outcrop area.

Table 2.4-22

HISTORICAL INFORMATION

WATER WELL DATA WITHIN TWO MILES OF THE STATION

Security-Related Information
Table Withheld Under 10 CFR 2.390

Table 2.4-23

HISTORICAL INFORMATION

SPRING DATA WITHIN TWO MILES OF THE STATION

Security-Related Information
Table Withheld Under 10 CFR 2.390

Table 2.4-24

HISTORICAL INFORMATION

DATA FOR MAJOR WATER WELLS[1], OTHER THAN PUBLIC WELLS, LOCATED BETWEEN 2 AND 10 MILES FROM THE STATION

[1]Major water wells are defined as those with a reported yield of 15 gpm or more.

Security-Related Information Table Withheld Under 10 CFR 2.390

Table 2.4-25

HISTORICAL INFORMATION

DATA FOR PUBLIC SUPPLY WELLS LOCATED BETWEEN 2 AND 20 MILES FROM THE STATION

Security-Related Information
Table Withheld Under 10 CFR 2.390

TABLE 2.4-26

ESTIMATED GROUNDWATER WITHDRAWAL IN 1976
WITHIN TWO MILES OF THE STATION

Geological Unit	Rate, in gpd		
	From Wells	From Springs	Total
Mahantango Formation	27,800	1,060	28,860
Trimmers Rock Formation	10,790	11,310	22,100
Catskill Formation	0	300	300
Pleistoncene Sand & Gravel	2,070	1,550	3,620
Recent Alluvium	210	0	210
Residual Soil	300	0	300
TOTALS	41,170	14,220	55,390

Note: gpd - gallons per day

Metric Conversion Factor: 1 gallon = 3.785 liters

TABLE 2.4-27

PROJECTIONS OF FUTURE GROUNDWATER WITHDRAWAL
WITHIN 2 AND 20 MILES OF THE STATION

Area	Year				
	1980	1990	2000	2010	2020
For area within 2-mile radius of station, in mgd	0.060	0.063	0.064	0.066	0.066
For area within 20-mile radius of station, in mgd	11.7	12.1	12.1	11.5	10.9

Notes:

Groundwater Withdrawal Projections were based on population projections, as found in Tables 2.1-7 through 2.1-16.

mgd - million gallons per day

Metric Conversion Factor: 1 gallon = 3.785 liters

HISTORICAL INFORMATION

TABLE 2.4-28

MAJOR GROUNDWATER WITHDRAWAL, AND POPULATION SERVED BY WATER SUPPLY COMPANIES WITHIN 20-MILE AREA

GROUNDWATER USER OR WATER SUPPLY COMPANY	APPROXIMATE POPULATION SERVED WITHIN 20-MILE RADIUS IN 1970*	ESTIMATED AVERAGE GROUNDWATER WITHDRAWAL IN 1975*(GPD)
Keystone Water Company (Berwick Water Company)	16,982	2,900,000+
Bloomsburg Water Co.	14,768	0
Benton Water Co.	1,022	60,300
Catawissa Municipal Authority	1,701	175,000
Orangeville Municipal Water Company	431	19,400
Pennsylvania Gas and Water Company	159,705	0
Dallas Water Company	4,292	390,000
Freeland Municipal Water Authority	6,102	316,400
Hazleton City Authority:		
1) Derringer Division	209	27,668+
2) Lattimer Division	378	62,767+
3) Ebervale Division	610	116,644+
4) Tomhicken Division	101	6,748+
5) Delano Division	518	0+
6) Buck Mountain Division	969	0+
7) Hazleton Division	42,501	423,296+
Williams and Son Water Co.	75	0
Conyngham Water Company	1,556	120,600
Citizens Water Company	200	8,200+
Mocanaqua Water Company	1,151	0
Indian Springs Water Company	150	9,500
Beaver Brook Water Co.	232	10,000
Garbush Water Company	16	700
John Fielding	137	12,000
Shavertown Water Co.	1,212	268,300
Midway Manor Water Co.	263	30,000
Trucksville Water Co.	553	0
Shaverstown-Kingston Township Water Co.	158	22,000
Hillcrest Water Co.	53	9,600
Meadowcrest Water Co.	369	72,000
William A. Still, Estate Water Company	106	12,000
Oakhill Water Supply Co.	444	50,000

HISTORICAL INFORMATION

HISTORICAL INFORMATION

TABLE 2.4-28

MAJOR GROUNDWATER WITHDRAWAL, AND POPULATION SERVED BY WATER SUPPLY COMPANIES WITHIN 20-MILE AREA

GROUNDWATER USER OR WATER SUPPLY COMPANY	APPROXIMATE POPULATION SERVED WITHIN 20-MILE RADIUS IN 1970*	ESTIMATED AVERAGE GROUNDWATER WITHDRAWAL IN 1975*(GPD)
Village Water Company	44	2,800
Shickshinny Water Co.	1,832	0
Warden Place Water Co.	~30	2,000
Whitebread Water Co.	37	11,200
Harvey's Lake Water Co.	60	5,500
Honey Brook Water Co.	6,133	720,000
Oneida Water Co.	319	23,000
Nuremburg Water Co.	486	33,300
Ringtown Boro Water Co.	909	38,400
Shenandoah Boro Municipal Authority	10,311	0
Keystone Water Company, Frackville Div.	357	0
Mahanoy Township Authority	7,538	4,000
Weatherly Municipal Authority	1,916	146,700
Beaver Meadows Municipal Authority	1,057	0
Wilbar Realty Company:		
1) Forest Park Division	146	10,000
2) Penn Lake Division	36	2,000
White Haven Municipal Authority	1,323	~0
Native Textiles, Dallas, Pa.	0	3,150
White Haven State School	--	22,460
Pennsylvania Institution For Def. Delinquents	--	167,650
TOTALS	289,498	6,315,283

* Information taken from Reference 2.4-70

\+ Information obtained from the local water department and from Reference 2.4-70

Note: gpd - gallons per day

Metric Conversion Factor: 1 gallon = 3.785 liters

HISTORICAL INFORMATION

TABLE 2.4-29

**ESTIMATION OF TOTAL GROUNDWATER WITHDRAWAL IN 1975
WITHIN 20-MILE RADIUS OF THE STATION**

1.	Total Estimated 1970 population within 20-mile radius*	352,852
2.	Estimated Population within 20-mile radius served by water companies or municipal water departments in 1970+	289,498
3.	Estimated population using private wells or springs to supply water needs in 1970	63,354
4.	Estimated population using private wells or springs in 1975: Approximate ratio of 1975 to 1970 population in area = 1.022** Therefore, 63,354 x 1.022 =	64,748
5.	Estimated withdrawal from private wells and springs in 1975 (for domestic and livestock use): 64,748 x 80 gpd/person =	5,179,840 gpd
6.	Estimated total withdrawal from public supply and industrial wells and from major springs within 20-mile radius, in 1975+	6,315,283 gpd
7.	Total Estimated Groundwater Withdrawal in region in 1975	11,495,123 gpd

* Based on Reference 2.4-72 (U.S. Bureau of the Census, 1970 Census of Population, Number of Inhabitants Pennsylvania PC(1)-A40, U.S. Govt. Printing Office, Washington, D.C.)

** Based on Tables 2.1-3 and 2.1-5

\+ Source is unpublished records and computer printouts from the Division of Comprehensive Resources and Planning of the Pennsylvania Department of Environmental Resources (Reference 2.4-70).

Note: gpd - gallons per day

Metric Conversion Factor: 1 gallon = 3.785 liters

TABLE 2.4-30

DETAILS OF THE CONSTRUCTION OF OBSERVATION WELLS AT THE SUSQUEHANNA SES

Observation well no.	Casing, internal diam. & type of material	Approx. depth interval of any screen or slotted casing (ft)	Original depth of boring (ft)	Present plumbable depth (ft)	Depth to top of bedrock (ft)	Static water level in late April '77 (ft below ground)	Probable geologic zone(s) in hydraulic connection with well
2	1.8", PVC	*	160.	54.4	60.	15.9	Lower Overburden (silty sand and gravel)
8	1.6", PVC	+	264.	>200.	75.	9.4	Lowermost overburden and bedrock
11	1.5", PVC	*	80.	46.5	60.	9.0	Lower overburden (coarse sand and silty gravel layers)
19	1.6", PVC	*	80.	42.6	53.	9.2	Lower overburden (coarse sand and gravel)
109	1.7", PVC	~70 - 90	176.	116.3	81.	16.2	Lower overburden and upper bedrock
124	1.6", PVC	~18 - 38	168.	46.	40.	19.5	Lower overburden and bedrock (sandy gravel)
1111	1.8", PVC	~75 - 95	109.	104.	99.	70.5	Lower overburden (sandy gravel with boulders)
1113	1.8", PVC	~60 - 80	97.	85.8	83.	60.6	Lower overburden (gravel, boulders and sand)
1114	1.9", PVC	~40 - 60	74.	64.2	63.	57.6	Lower overburden (boulders, gravel and sand)
B-1	8", steel	84.5 - 89.5	96.	87.5	>96.	5.4	Lower overburden (sand with fine gravel)
CPW	12", steel	42 - 57	82.	54.5	80.	4.5	Lower overburden (gravel and sand)
1200A	3", PVC	20 - 32	32.	32.	30.	31.2[1]	Lower overburden and upper two feet of bedrock
1201	4", steel	+	69.	69.	34.	19.5[1]	Upper 35 feet of bedrock
1204	4", PVC	42 - 54	54.8	54.	52.5	13.5[1]	Lower overburden and upper two feet of bedrock
1208	3", PVC	26 - 38	38.	38.	36.	26.6[1]	Lower overburden and upper two feet of bedrock
1209A	4", steel	+	60.	60.	26.	15.9[1]	Upper 34 feet of bedrock
1210	4", PVC	26 - 38	39.5	38.	35.5	31.2[1]	Lower overburden and upper three feet of bedrock

* Not known

\+ Only blank casing used

1 Measured in September 1977

Metric Conversion Factor: 1 foot = 0.3048 meters

HISTORICAL INFORMATION

TABLE 2.4-31

GROUNDWATER LEVEL DATA TAKEN AT SUSQUEHANNA SES 1972 THROUGH 1975

Boring or Well	Depth (ft)	Probable zone(s) in hydraulic connection with boring or well	Date of measurement	Depth of water level below ground surface (ft)	Elevation of groundwater level (ft above m.s.l.)	Estimated height of groundwater level above top of bedrock (ft)
7	75	Overburden and upper bedrock	6-30-72 8-18-72 7-25-72	8.7 19.0 23.0	497.3 487.0 626.2	42 32 0
104	122	Bedrock	8-18-72	31.1	618.1	0
107	208	Overburden and bedrock	8-16-72 8-18-72	54.5 54.3	608.8 609.0	46 46
111	110	Overburden and bedrock	6-30-72 8-18-72	10.6 14.7	672.8 668.7	23 19
116	95	Overburden and bedrock	7-10-72 8-18-72	15.0 15.8	663.1 662.3	6 5
202	57	Overburden and upper bedrock	7-10-72 8-11-72	24.8 26.0	640.2 639.0	6 5
205	46	Overburden and upper bedrock	7-10-72 8-16-72	18.7 20.5	646.3 644.5	8 7
206	116	Overburden and bedrock	7-18-72 8-16-72	19.5 21.1	645.5 643.9	7 6
209	30	Overburden and upper bedrock	7-18-72 8-18-72 7-10-72	7.7 9.4 7.2	661.3 659.6 696.8	2 1 0
211	38	Bedrock	8-18-72	9.7	694.3	0
215	44	Overburden and upper bedrock	7-10-72 8-18-72	25.0* 19.0	644.0* 650.0	0* 3
301	42	Overburden and upper bedrock	6-30-72 7-18-72 6-30-72	13.7 15.2 18.2	666.3 664.8 659.8	2 1 0
305	70	Bedrock	8-1-72	25.7	652.3	0
312	62	Overburden and upper bedrock	6-30-72 8-18-72	5.2 9.9	698.8 694.1	4 0
317	50	Overburden and upper bedrock	6-30-72 8-01-72	19.3 23.7	696.7 692.3	5 0
319	60	Overburden and upper bedrock	6-30-72 8-18-72	3.7 15.0	688.3 677.0	18 7
410	73	Overburden and upper four feet of bedrock	7-18-72 8-18-72	57.0 65.3	626.1 617.8	12 4
411	70	Overburden and upper three feet of bedrock	7-18-72 8-01-72	58.5 60.4	629.4 627.5	9 7
413	67	Overburden and upper four feet of bedrock	7-18-72 8-18-72 7-21-72	48.4 48.7 25.5	640.5 640.2 672.8	15 14 33
415	59	Overburden	8-18-72	36.0	662.3	23
422	24	Overburden and upper four feet of bedrock	7-26-72	7.2	509.7	17

HISTORICAL INFORMATION

TABLE 2.4-31

GROUNDWATER LEVEL DATA TAKEN AT SUSQUEHANNA SES 1972 THROUGH 1975

Boring or Well	Depth (ft)	Probable zone(s) in hydraulic connection with boring or well	Date of measurement	Depth of water level below ground surface (ft)	Elevation of groundwater level (ft above m.s.l.)	Estimated height of groundwater level above top of bedrock (ft)
444	15	Overburden and upper four feet of bedrock	8-08-72	4.2	719.7	7
605	25	Overburden and upper six feet of bedrock	8-08-72	6.0	689.1	13
606	10	Bedrock	8-08-72	4.2	691.2	0
1002	18	Overburden	4-08-74	3.3	505.7	>15
1006	24	Overburden	4-08-74	6.1	504.9	>18
1008	22	Overburden	4-08-74	10.8	501.2	>11
1010	15	Overburden	4-08-74	12.8	501.7	>2
1111	109	Overburden	7-31-74 11-07-74 1-17-75 5-15-75	74.7 78.7 77.7 73.7	610.7 608.7 609.7 613.7	22 20 21 25
1113	97	Overburden	7-31-74 11-08-74 1-17-75 5-15-75	69.0 73.0 70.0 70.0	633.1 629.1 632.1 632.1	14 10 13 13
1114	74	Overburden and upper 11 feet of bedrock	7-31-74 11-14-74 1-10-75 5-15-75	64.0 65.0 64.0 64.0	647.7 646.7 647.7 647.7	-1 -2 -1 -1

* Reading in doubt

Metric Conversion Factor: 1 foot = 0.3048 meters

HISTORICAL INFORMATION

TABLE 2.4-32

GROUNDWATER LEVEL DATA TAKEN AT SUSQUEHANNA SES 1976 THROUGH 1977

Observation Well	Zone(s) Tapped By Well	Date of Measurement	Depth of Water Level Below Ground Surface (ft)	Elevation of Groundwater Level (ft above m.s.l.)	Height of Groundwater Level Above Top of Bedrock (ft)
2	Lower Overburden	11-09-76	17.1	495.3	43
		01-04-76	21.2	491.2	39
		04-14-77	16.2	496.2	44
		07-26-77	23.0	489.4	37
		08-16-77	22.9	489.5	37
		09-20-77	22.0	490.3	38
4	Lower Overburden and Bedrock	11-10-76	10.4	495.7	65
		01-05-76	13.5	492.6	62
		04-14-77	8.5	497.5	67
		07-26-77	14.9	491.1	60
		08-16-77	14.9	491.1	60
		09-20-77	13.5	492.6	62
11	Lower Overburden	04-14-77	7.8	500.6	52
		07-26-77	12.9	495.5	47
		08-16-77	12.9	495.5	47
		09-20-77	12.7	495.6	47
19	Lower Overburden	11-10-76	12.0	493.1	41
		01-05-76	15.1	490.0	38
		04-14-77	11.2	493.9	42
		07-26-77	16.7	488.4	36
		08-16-77	16.4	488.8	37
		09-20-77	13.1	492.1	40
109	Lower Overburden and Upper Bedrock	11-09-76	16.3	593.0	65
		01-04-77	21.8*	587.5	59*
		04-14-77	15.5	593.9	65
		07-26-77	26.3*	583.0	55*
		08-16-77	30.1*	583.0	51*
		09-20-77	37.6**	579.2	43**
				571.7	
124	Lower Overburden and Bedrock	11-08-76	19.7	523.7	20
		01-04-77	22.7	520.7	17
		04-14-77	17.5	625.9	23
		07-26-77	27.5	615.9	13
		08-16-77	28.4	615.0	12
		09-20-77	28.7	614.7	11
1111	Lower Overburden	11-08-76	71.2	616.9	29
		01-04-77	16.0	612.1	24
		04-14-77	68.5	619.5	31
		04-27-77	70.5	617.6	29
1113	Lower Overburden	11-09-76	60.2	634.1	15
		01-04-77	61.4	634.9	14
		04-14-77	59.8	634.5	15
		04-27-77	60.6	633.7	15
1114	Lower Overburden	11-09-76	57.2	644.7	-4
		01-04-77	58.1	643.8	-5
		04-14-77	57.3	644.6	-4
		04-27-77	57.6	644.3	-4

* Pump in well TW-2 (170 feet from observation well 109) was on during measurement.

** Pumps in wells TW-1 (31 feet from observation well 109) and TW-2 were on during measurement.

HISTORICAL INFORMATION

TABLE 2.4-33

SUMMARY OF PERMEABILITY TESTS OF OVERBURDEN AND UPPER BEDROCK AT THE SUSQUEHANNA SES PERFORMED DURING PREVIOUS INVESTIGATIONS

Type of Test	Location of test or sample collection	Geologic material tested	Value(s) of hydraulic conductivity obtained (ft/day)		Reference
			Horizontal (Kh)	Vertical (Kv)	
Pumping Tests	Wells TW-1 & TW-2	Lower 40 feet of Kame terrace deposits	3.3 to 15.0	--	25
	Well C	Lower 43 feet of Kame terrace deposits	200.**	--	21
	Well CPW	37 feet of permeable materials within Kame terrace deposits	194.**	--	22
Falling-head Laboratory Permeability Test	Approximately 1500 ft. northeast of plant center	Upper silty soils	--	0.028	20
Open-End	Prospective retention pond areas	Kame terrace deposits	5.7 (Tests performed in 29 borings	13. to 63. (Tests performed in 29 borings)	27
Tests	Spray pond area (borings 1111, 1112, 1115, 1122, 1123, 1124, & 1125)	Kame terrace deposits	0.022 to greater than 11.8	--	29
in	Spray pond area (borings 1113 & 1114)	Kame terrace deposits and underlying few feet of siltstone	1.0 to 3.8	--	29
Borings+	Spray pond area (boring 1117)	Mahantango siltstone in interval 12-22' below top of rock	2.5	--	29
Packer Tests	Near railway bridge over Rt. 11 (borings 929-935, and 937-940)	Mahantango siltstone and black shale, upper 50 feet of rock	0.013 to 0.76 (Median of 41 tested intervals = 0.22)	--	29
in Borings+	Reactor area and prospective retention pond areas	Mahantango siltstone upper 20 feet	0.85	--	27
		Mahantango siltstone below 20 feet	1.0×10^{-6}	--	27

** Based on specific capacity data, assuming wells were 85 percent efficient.
+ Performed in accordance with designations E-18 and E-19 of the U.S. Bureau of Reclamation's Earth's Manual.
Metric Conversion Factor: 1 foot = 0.3048 meters.

TABLE 2.4-34

SUMMARY OF PERMEABILITY TESTS OF OVERBURDEN AND UPPER BEDROCK AT THE SUSQUEHANNA SES PERFORMED FOR THIS INVESTIGATION

Type of Test	Location of Test or Soil Sample Collection	Geological Material Tested	Average Thickness of Saturated Zone Tested (ft)	Depth of Interval Below Top of Bedrock (ft)	Values of Hydraulic Conductivity Obtained (ft/day)	
					Horizontal (Kh)	Vertical (Kv)
Constant-head Laboratory Permeability Test	Boring 1200A at 27-foot depth	Kame terrace deposits	-	-	-	2.3
Slug Tests	Well 1208	Saturated Kame terrace deposits and upper 2 to 3 feet of bedrock	11.5	-	1.8	-
	Well 1210	Saturated Kame terrace deposits and upper 2 to 3 feet of bedrock	6.8	-	6.6	-
6-Hour Pumping Tests	Well 1210	Saturated Kame terrace deposits and upper 2 to 3 feet of bedrock	6.8	-	7.8	-
	Well 1204	Saturated Kame terrace deposits and upper 2 to 3 feet of bedrock	19.3*	-	21.7 to 29.2	-
Packer Tests	Boring 305+	Mahantango siltstone bedrock	-	7 to 12 12 to 17 17 to 52	0.41 0.048 0.0061	- - -
	Well 1201	Mahantango siltstone bedrock	-	6.7 to 15 15 to 25.3 25 to 35.3	0.063 0.0021 0	- - -
	Well 1209A	Mahantango siltstone bedrock	-	5.7 to 14 14 to 24 24 to 34	0.0012 0.028 0	- - -

* Average thickness for the confined aquifer between wells 1204 and 11.

\+ Only the analysis was performed for this investigation.

Metric Conversion Factor: 1 foot = 0.3048 meters

TABLE 2.4-35

RADIONUCLIDE CONTENT OF THE TANK POSTULATED TO RUPTURE
- REACTOR WATER CLEANUP (RWCU) PHASE SEPARATOR TANKS -
SUSQUEHANNA SES

RADIONUCLIDE OF CONCERN FOR ACCIDENT ANALYSIS	HALF LIFE	HALF LIFE (DAYS)	TOTAL CURIES IN TANK (Ci)	CONCENTRATION ON BASIS OF 80% OF TOTAL TANK VOLUME OF 7400 GAL (µCi/ml)
Mn-54	312 d	312	38.6	1.72E+00
Fe-55	2.7 y	986.18	719.0	3.21E+01
Co-60	5.272 y	1,925.60	307.0	1.37E+01
Sr-90	29 y	10,592.25	5.68	2.53E-01
I-131	8.04 d	8.04	46.0	2.05E+00
Cs-137	30.17 y	11,019.59	16.2	7.23E-01
Pu-239	2.411E+4 y	8.8062E+06	0.0015	6.87E-05

NOTE: Liquid volume of 5,920 gallons x 3.785 liters/gal x 1000 ml/liter = 2.24072E+7 ml

Table 2.4-36

GROUNDWATER PARAMETER VALUES USED FOR SLUG3D SIMULATIONS ACCIDENT ANALYSIS FOR SUSQUEHANNA SES

Flow Path Segment	Description	Geologic Unit in Which Flow Occurs	Travel Distance (ft)	Horizontal Hydraulic Conductivity (ft/day)	Hydraulic Gradient	Total Porosity	Effective Porosity	Dispersivities (ft)			Kd values (ml/g)				
								aL	aT	aV	Mn-54	Fe-55	Co-60	Sr-90	Cs-137
FLOWPATH 1															
1	RWCU Tank North To Buried Valley	Upper 15 ft of Bedrock	805	0.5	0.0600	0.02	0.02	10.0	0.5	0.001	0.0	0.0	0.0	0.0	0.0
2	Along Buried Valley To Well TW-2	Lower Pleistocene Deposits	725	18.0	0.0240	0.30	0.27	10.0	0.5	0.001	8.7	13.5	10.0	7.0	18.0
3	TW-2 To Stream Just East Of RR Tracks	Lower Pleistocene Deposits	860	8.0	0.0420	0.30	0.27	10.0	0.5	0.001	8.7	13.5	10.0	7.0	18.0
4	From Point In Stream To Lake Took-A-While	Lower Pleistocene Deposits	1420	20.0	0.0388	0.30	0.27	10.0	0.5	0.001	8.7	13.5	10.0	7.0	18.0
5	From Lake Took-A- While To River	Lower Pleistocene Deposits	1720	60.0	0.0081	0.30	0.27	30.0	2.0	0.050	8.7	13.5	10.0	7.0	18.0
FLOWPATH 2															
1	RWCU Tank East To North Stream Just East Of RR Tracks	Upper 15 ft of Bedrock	1865	0.5	0.0550	0.02	0.02	10.0	0.5	0.001	0.0	0.0	0.0	0.0	0.0
2	From Point In Stream To Lake Took-A-While	Lower Pleistocene Deposits	1420	20.0	0.0388	0.30	0.27	10.0	0.5	0.001	8.7	13.5	10.0	7.0	18.0
3	From Lake Took-A- While To River	Lower Pleistocene Deposits	1720	60.0	0.0081	0.30	0.27	30.0	2.0	0.050	8.7	13.5	10.0	7.0	18.0

Table 2.4-37

RANGE OF PARAMETER VALUES USED IN CALIBRATED NUMERICAL MODEL OF BURIED VALLEY AQUIFER NORTHERN SIDE OF SUSQUEHANNA SES

PARAMETER	UNITS	VALUE
FLOW MODEL (MODFLOW)		
HEAD IN CONSTANT-HEAD CELLS		
Upgradient Boundary	ft (msl)	648.0 - 649.0
Downgradient Boundary	ft (msl)	562.0 - 564.0
HOR. HYDRAULIC CONDUCTIVITY	ft/day	4.0 - 50.0
RECHARGE	ft/day	0.0003 - 0.0085
SPECIFIC STORAGE	1/ft	1.15E-05
SPECIFIC YIELD	-	0.20
TOTAL POROSITY	-	0.30
EFFECTIVE POROSITY	-	0.27
TRANSPORT MODEL (MT3D)		
INITIAL CONCENTRATIONS		
At cell (17,8)		
Mn-54	µCi/ml	0.0765
Fe-55	µCi/ml	2.290
Co-60	µCi/ml	1.080
Sr-90	µCi/ml	0.0219
Cs-137	µCi/ml	0.0625
Pu-239	µCi/ml	6.01E-06
elsewhere	µCi/ml	0
CONSTANT CONCENTRATION CELL		
At cell (17,8) first 500 days		
Mn-54	µCi/ml	0.0765
Fe-55	µCi/ml	2.290
Co-60	µCi/ml	1.080
Sr-90	µCi/ml	0.0219
Cs-137	µCi/ml	0.0625
Pu-239	µCi/ml	6.01E-06
At cell (17,8) after 500 days	µCi/ml	0.0
AQUIFER BULK DENSITY	(lb/cu.ft.)	115.75
TRANSPORT MODEL (MT3D) - cont		

Table 2.4-37

RANGE OF PARAMETER VALUES USED IN CALIBRATED NUMERICAL MODEL OF BURIED VALLEY AQUIFER NORTHERN SIDE OF SUSQUEHANNA SES

PARAMETER	UNITS	VALUE
DISPERSIVITIES		
I_L	ft	10.0
I_T	ft	0.5
I_V	ft	0.001
DISTRIBUTION COEFFICENTS (kd)		
Mn-54	ml/g	8.7
Fe-55	ml/g	13.5
Co-60	ml/g	10.0
Sr-90	ml/g	7.0
Cs-137	ml/g	18.0
Pu-239	ml/g	0.0
DECAY CONSTANTS		
Mn-54	1/day	2.2212E-03
Fe-55	1/day	7.0272E-04
Co-60	1/day	3.5989E-04
Sr-90	1/day	6.5425E-05
Cs-137	1/day	6.2888E-05

Table 2.4-38

ESTIMATED PEAK CONCENTRATION OF RADIONUCLIDES IN GROUNDWATER
RESULTING FROM POSTULATED RUPTURE OF RWCU PHASE SEPARATOR TANK
- RESULTS OF SIMULATIONS WITH SLUG3D MODEL -
SUSQUEHANNA SES

Radionuclide	At Well TW-2		At Biology Lab Well		At Susquehanna River		10 CFR 20 Appendix B Table 2 Effluent Concentration Limit (µCi/ml)
	Time of Peak Since Accident (days)	Peak Concen. (µCi/ml)	Time of Peak Since Accident (days)	Peak Concen. (µCi/ml)	Time of Peak Since Accident (days)	Peak Concen. (µCi/ml)	
FLOWPATH 1							
Mn-54*	10,350	7.70E-15	-	< 1.0E-30	-	< 1.0E-30	3.0E-05
Fe-55*	18,660	4.27E-08	-	< 1.0E-30	79,100	< 1.0E-30	1.0E-04
Co-60*	15,850	1.38E-04	-	< 1.0E-30	73,870	9.16E-16	3.0E-06
Sr-90*	12,010	5.34E-04	-	< 1.0E-30	59,720	5.60E-07	5.0E-07
I-131	360	5.83E-18	-	< 1.0E-30	-	< 1.0E-30	1.0E-06
Cs-137*	28,240	2.08E-04	-	< 1.0E-30	141,770	3.34E-09	1.0E-06
Pu-239	560	6.75E-06	-	< 1.0E-30	1,570	2.86E-07	2.0E-08
FLOWPATH 2							
Mn-54*	np	np	-	< 1.0E-30	22,030	1.54E-29	3.0E-05
Fe-55*	np	np	-	< 1.0E-30	39,480	1.69E-16	1.0E-04
Co-60*	np	np	-	< 1.0E-30	33,760	6.56E-09	3.0E-06
Sr-90*	np	np	-	< 1.0E-30	25,400	7.49E-06	5.0E-07
I-131	np	np	-	< 1.0E-30	630	3.33E-30	1.0E-06
Cs-137*	np	np	-	< 1.0E-30	59,900	8.55E-07	1.0E-06
Pu-239	np	np	-	< 1.0E-30	900	3.00E-07	2.0E-08

* Adsorption through cation exchange on Pleistocene deposits included in simulation
\+ 10CFR20 Appendix B (2007)
np not on flow path

Table 2.4-39

ESTIMATED PEAK CONCENTRATION OF RADIONUCLIDES IN GROUNDWATER IN BURIED VALLEY AQUIFER RESULTING FROM POSTULATED RUPTURE OF RWCU PHASE SEPARATOR TANK - RESULTS OF SIMULATIONS WITH SLUG3D MODEL - SUSQUEHANNA SES

Radionuclide	At Entry Point to Aquifer Cell (17,8)		At Well TW-2 Cell (27,6)		At Downgradient Boundary Cell (36,1)		10 CFR 20 Appendix B Table 2 Effluent Concentration Limit (µCi/ml)
	Time of Peak Since Accident** (days)	Peak Concen.** (µCi/ml)	Time of Peak Since Accident (days)	Peak Concen. (µCi/ml)	Time of Peak Since Accident (days)	Peak Concen. (µCi/ml)	
Mn-54*	281	1.78E-01	3,100	3.33E-09	4,050	1.25E-27	3.0E-05
Fe-55*	300	5.10E+00	7,020	3.97E-06	11,180	2.09E-20	1.0E-04
Co-60*	300	2.41E+00	8,140	1.03E-04	28,210	7.24E-13	3.0E-06
Sr-90*	300	4.87E-02	8,540	5.78E-05	46,140	3.59E-08	5.0E-07
I-131	190	1.49E-09	ns	ns	ns	ns	1.0E-06
Cs-137*	300	1.39E-01	19,140	3.14E-05	92,110	8.70E-10	1.0E-06
Pu-239	300	1.34E-05	708	4.97E-07	1,740	1.20E-08	2.0E-08

* Adsorption through cation exchange on Pleistocene deposits included in simulation
** Computed from SLUG3D simulations of migration from ruptured tank to buried-valley aquifer
\+ 10CFR20 Appendix B (2007)
ns Simulation not run because of low I-131 concentration at entry point to aquifer

NOTE: These simulations were performed with Well TW-2 pumping continuously at 31 gpm

Security-Related Information
Figure Withheld Under 10 CFR 2.390

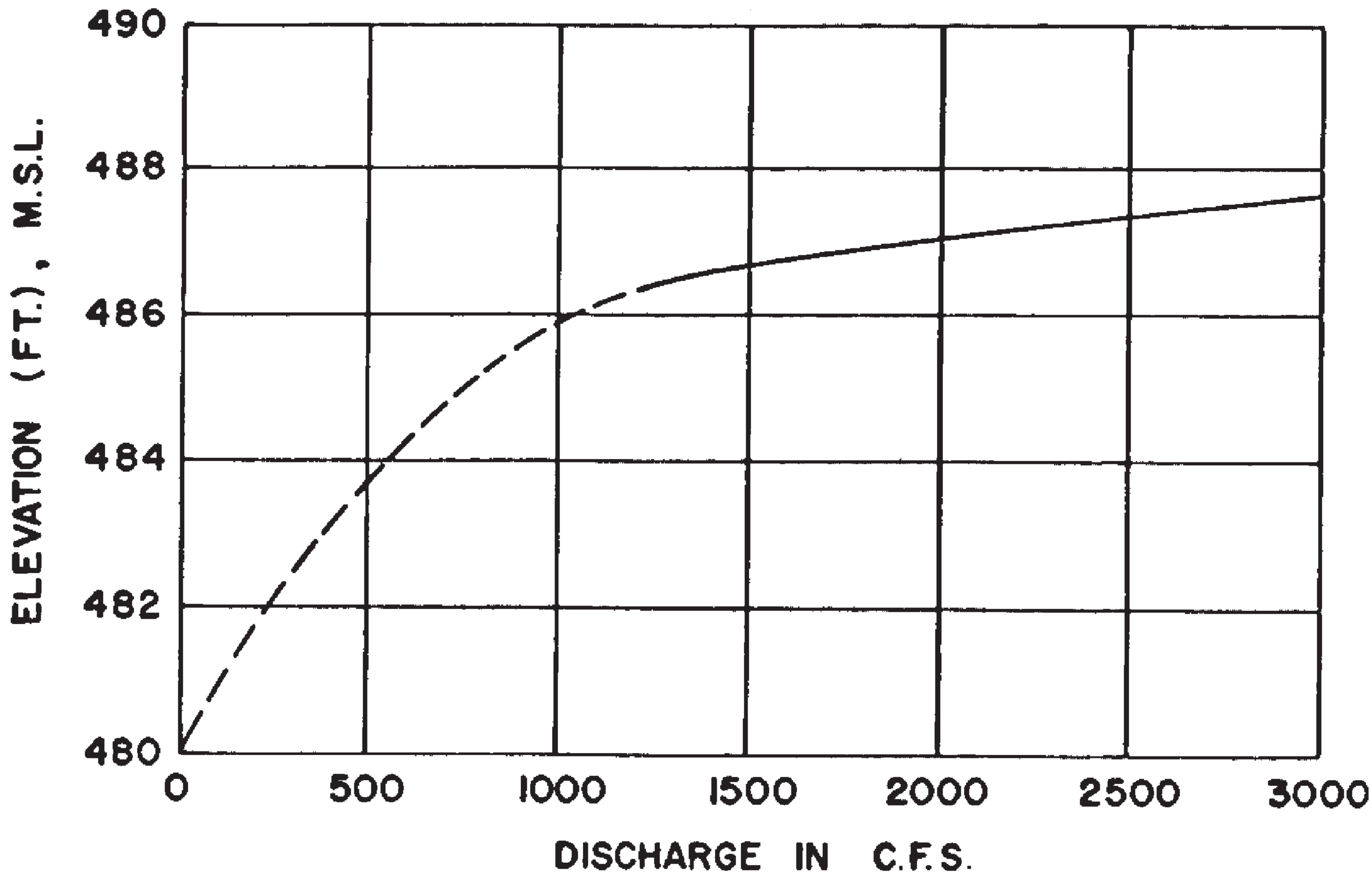
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

PLANT COMPLETE
SHOWING STORM DRAIN
PIPE LAYOUT

FIGURE 2.4-3

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
SUSQUEHANNA RIVER BASIN FIGURE 2.4-4



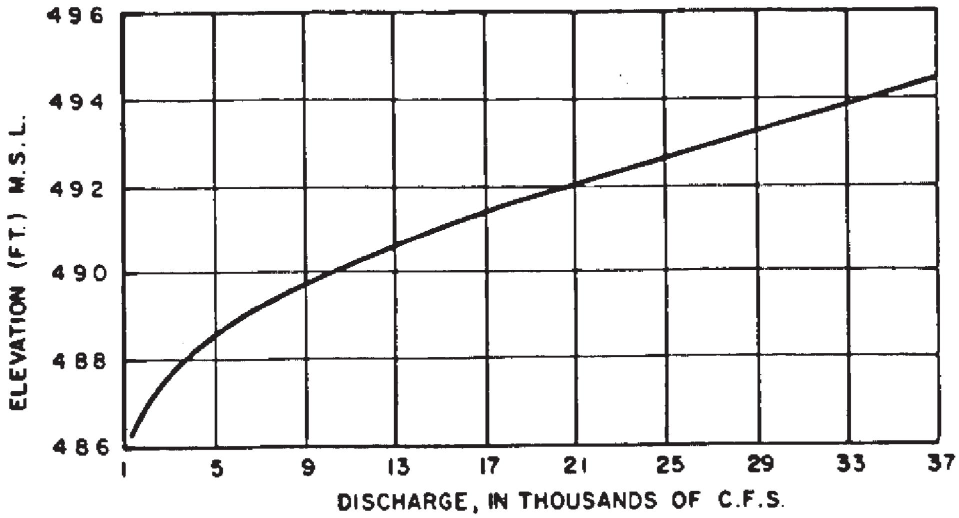
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

STAGE DISCHARGE CURVE
AT LOW FLOWS

FIGURE 2.4-5, Rev 47

AutoCAD: Figure Fsar 2_4_5.dwg



NOTES: ELEVATIONS MEASURED BY GAGE AT SUSQUEHANNA SITE

FLOWS MEASURED AT WILKES-BARRE AND DANVILLE. FLOW AT SITE OBTAINED BY INTERPOLATION ON BASIS OF DRAINAGE AREA.

DRAINAGE AREAS :

WILKES-BARRE 9960 SQ MILES (25795 SQ KM)

SUSQUEHANNA SITE 10200 SQ MI (26416 SQ KM)

DANVILLE 11220 SQ MI (29058 SQ KM)

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SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

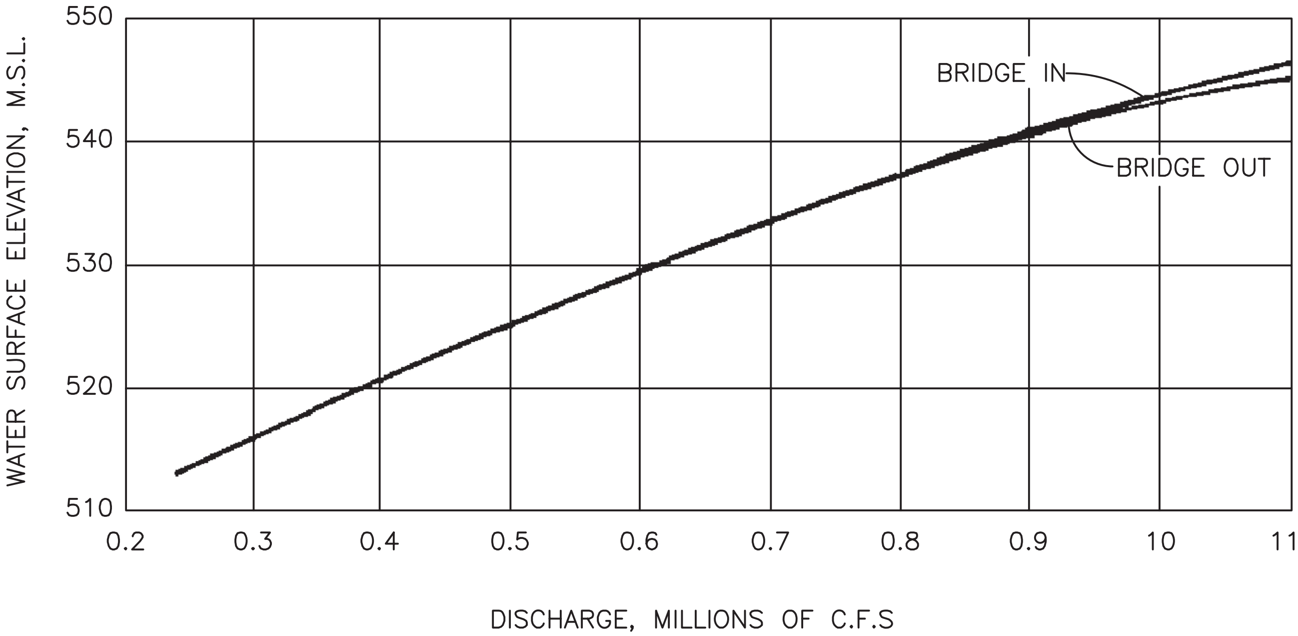
STAGE DISCHARGE CURVE,
DISCHARGE RANGE
1000 - 37000 CFS

FIGURE 2.4-6, Rev 47

AutoCAD: Figure Fsar 2_4_6.dwg

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
WATER USERS ON THE SUSQUEHANNA RIVER FIGURE 2.4-7



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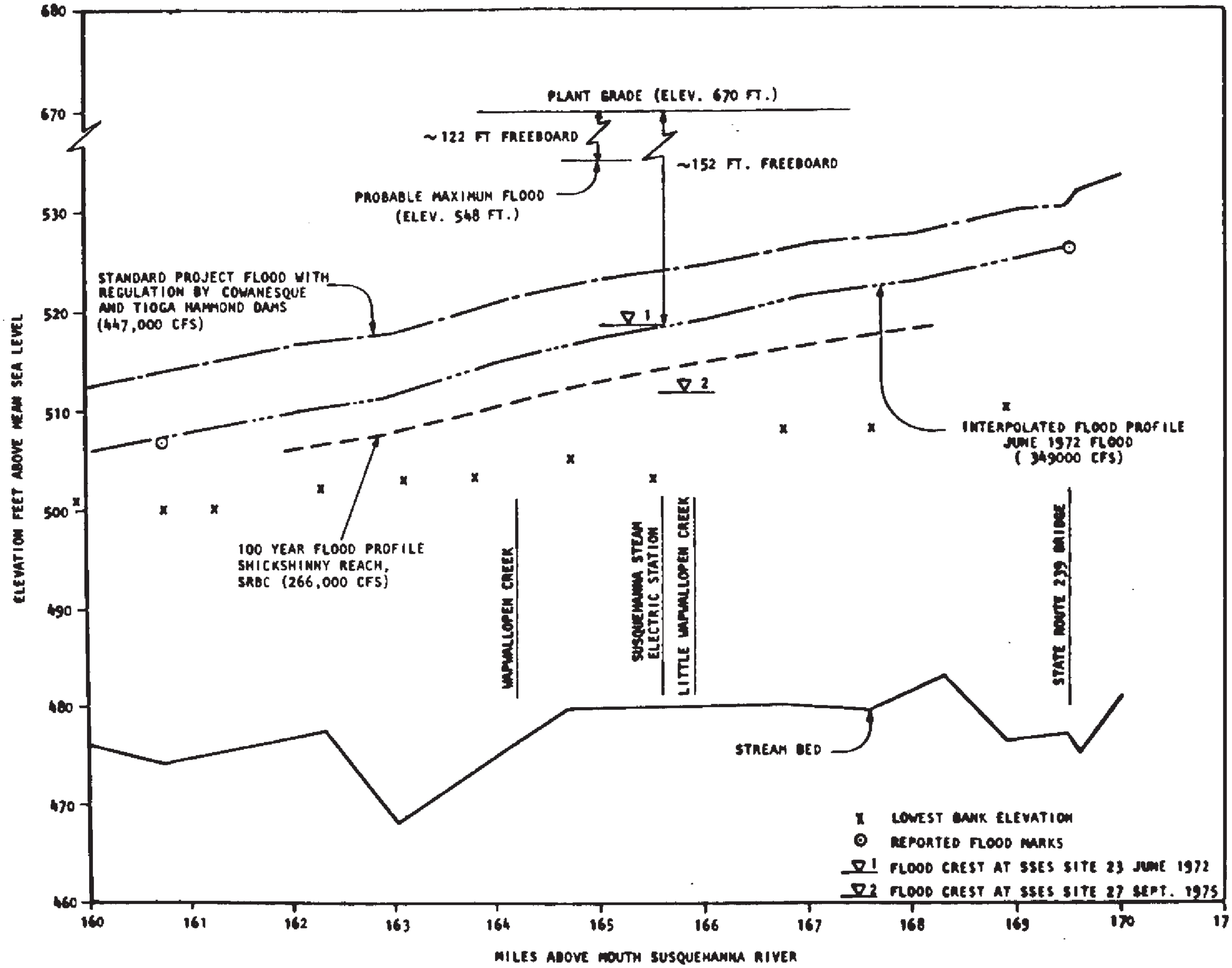
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

STAGE DISCHARGE CURVES
AT PLANT SITE

FIGURE 2.4-8, Rev 47

AutoCAD: Figure Fsar 2_4_8.dwg



DATA SOURCE:
BALTIMORE DISTRICT CORPS OF ENGINEERS 1974.

DATA SOURCE FOR THE 100 YEAR FLOOD PROFILE:
FEDERAL INSURANCE ADMINISTRATION, TYPE 15,
FLOOD INSURANCE STUDY. OBTAINED FROM
SUSQUEHANNA RIVER BASIN COMMISSION (SRBC)

FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

FLOOD PROFILES ON THE
SUSQUEHANNA RIVER
AT THE SITE

FIGURE 2.4-9, Rev 47

AutoCAD: Figure Fsar 2_4_9.dwg

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
GENERAL SITE DRAINAGE PLAN FIGURE 2.4-10

Security-Related Information
Figure Withheld Under 10 CFR 2.390

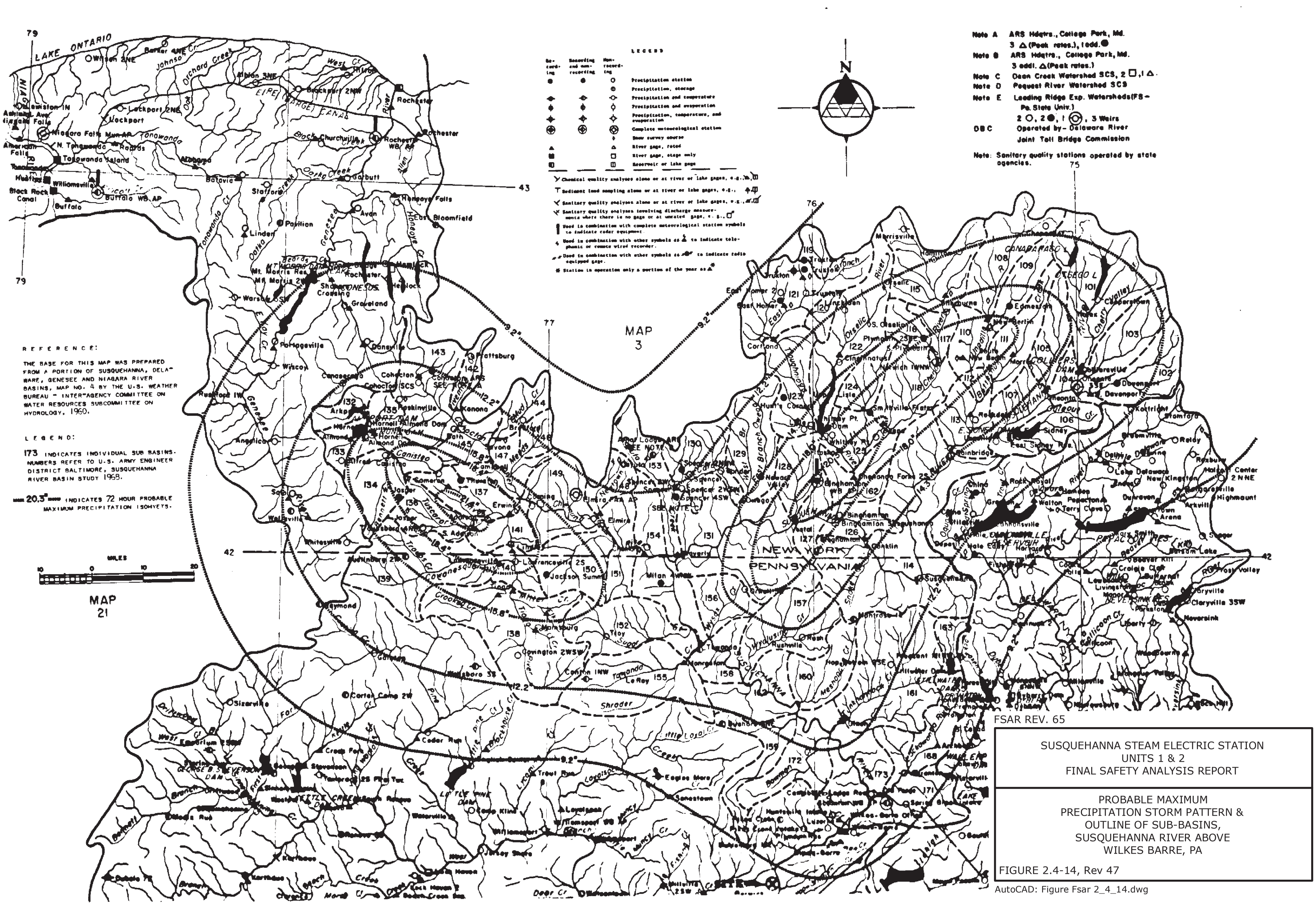
SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
SITE DRAINAGE LOCATIONS A & B FIGURE 2.4-11

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
SITE DRAINAGE LOCATIONS C & D FIGURE 2.4-12

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
SITE DRAINAGE LOCATIONS E & F FIGURE 2.4-13

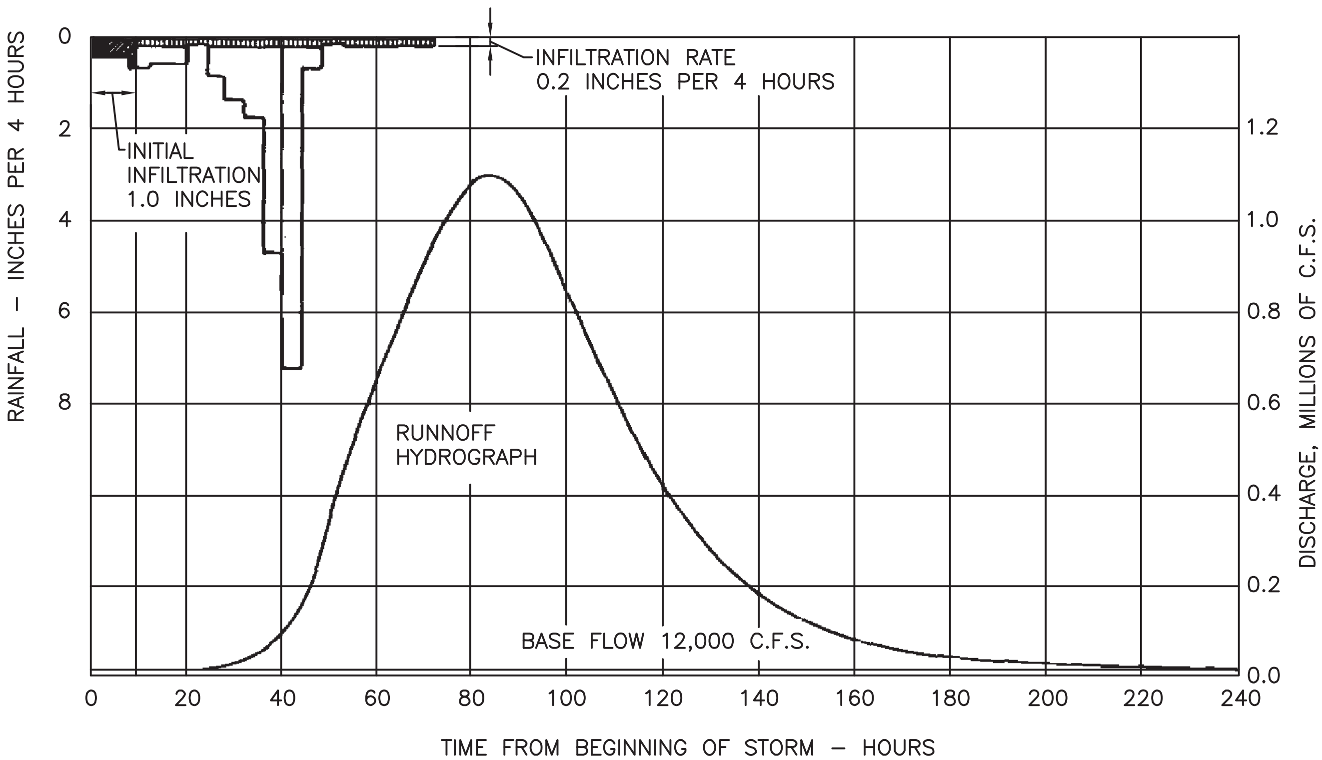


SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

PROBABLE MAXIMUM
PRECIPITATION STORM PATTERN &
OUTLINE OF SUB-BASINS,
SUSQUEHANNA RIVER ABOVE
WILKES BARRE, PA

FIGURE 2.4-14, Rev 47

AutoCAD: Figure Fsar 2_4_14.dwg



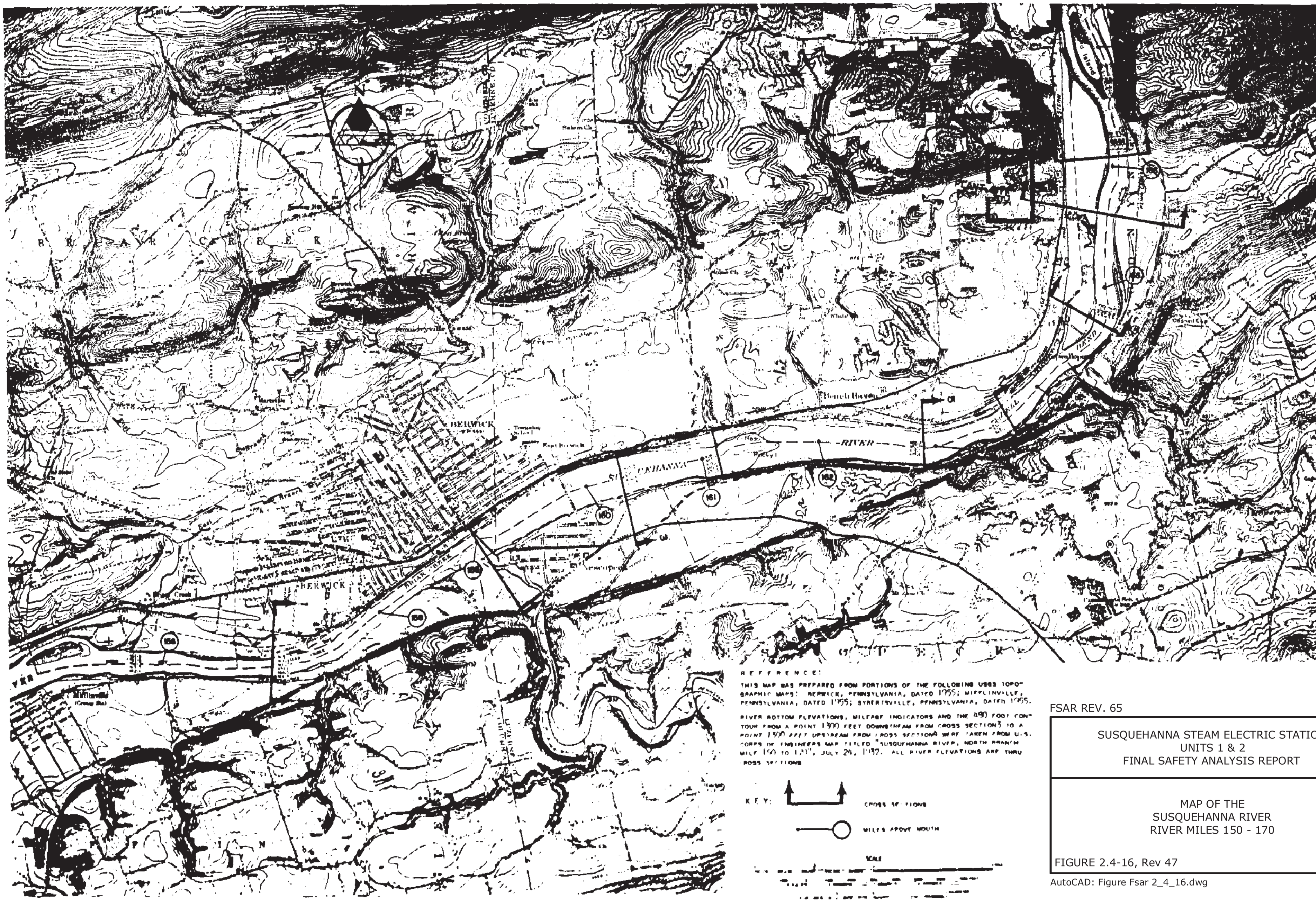
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

PROBABLE MAXIMUM
PRECIPITATION ON SUSQUEHANNA
RIVER BASIN & PROBABLE
MAXIMUM FLOOD HYDROGRAPH
AT WILKES BARRE, PA

FIGURE 2.4-15, Rev 47

AutoCAD: Figure Fsar 2_4_15.dwg



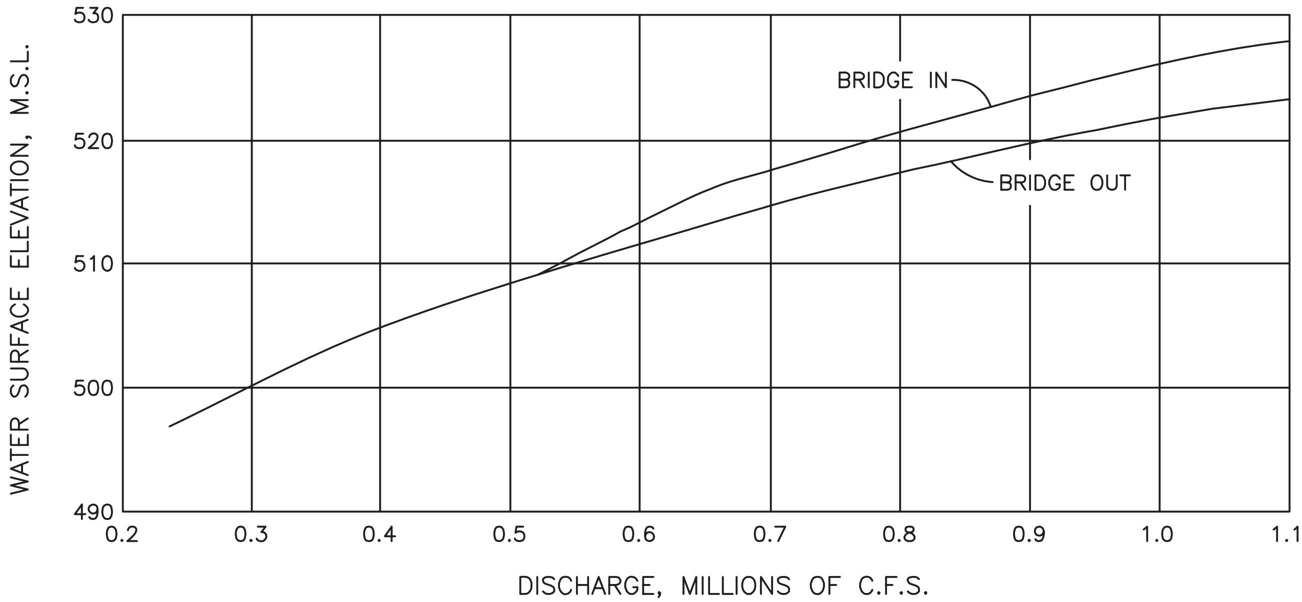
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

MAP OF THE
SUSQUEHANNA RIVER
RIVER MILES 150 - 170

FIGURE 2.4-16, Rev 47

AutoCAD: Figure Fsar 2_4_16.dwg



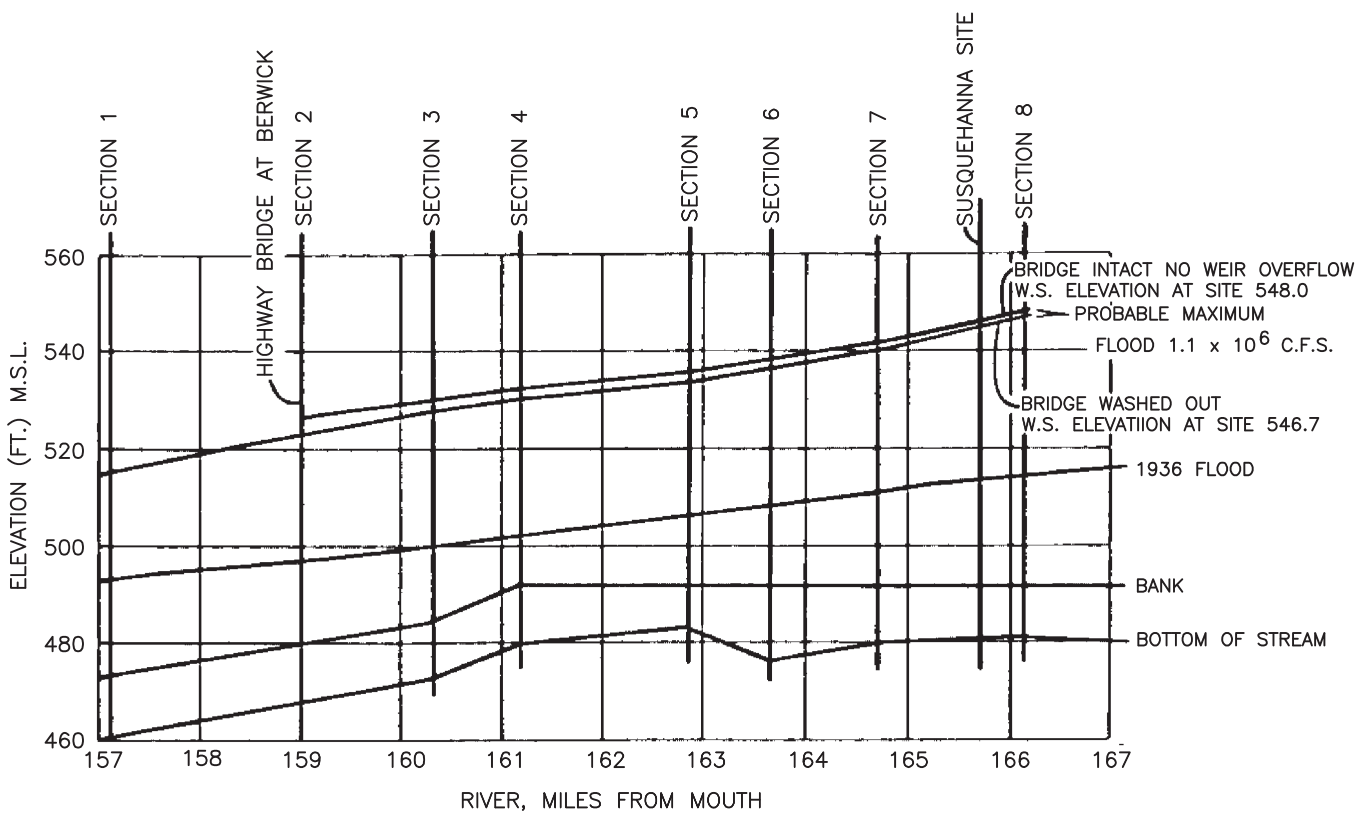
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

STAGE DISCHARGE CURVE AT
SECTION 2 -
BERWICK BRIDGE

FIGURE 2.4-17, Rev 47

AutoCAD: Figure Fsar 2_4_17.dwg



NOTE:

W.S. ELEVATIONS REFLECT
ADJUSTMENT OF +2.3 FT.
FOR WIND–WAVE RUNUP

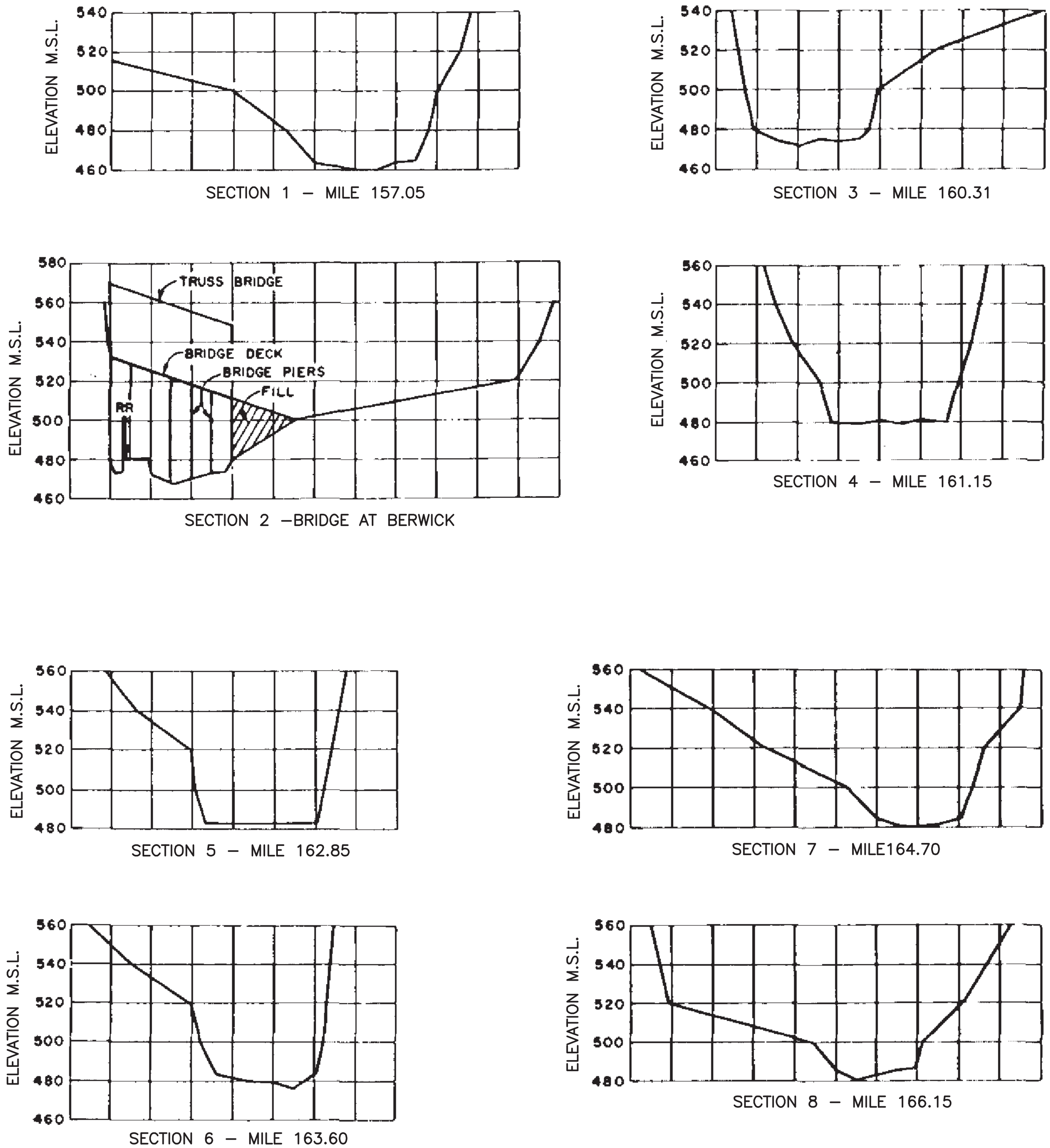
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

RIVER AND WATER SURFACE
PROFILES OF SUSQUEHANNA
RIVER IN VICINITY OF SITE

FIGURE 2.4-18, Rev 47

AutoCAD: Figure Fsar 2_4_18.dwg



CROSS SECTIONS LOOKING UPSTREAM
SCALE – HORIZONTAL– 1"=2000FT.
VERTICAL– 1"= 80FT.

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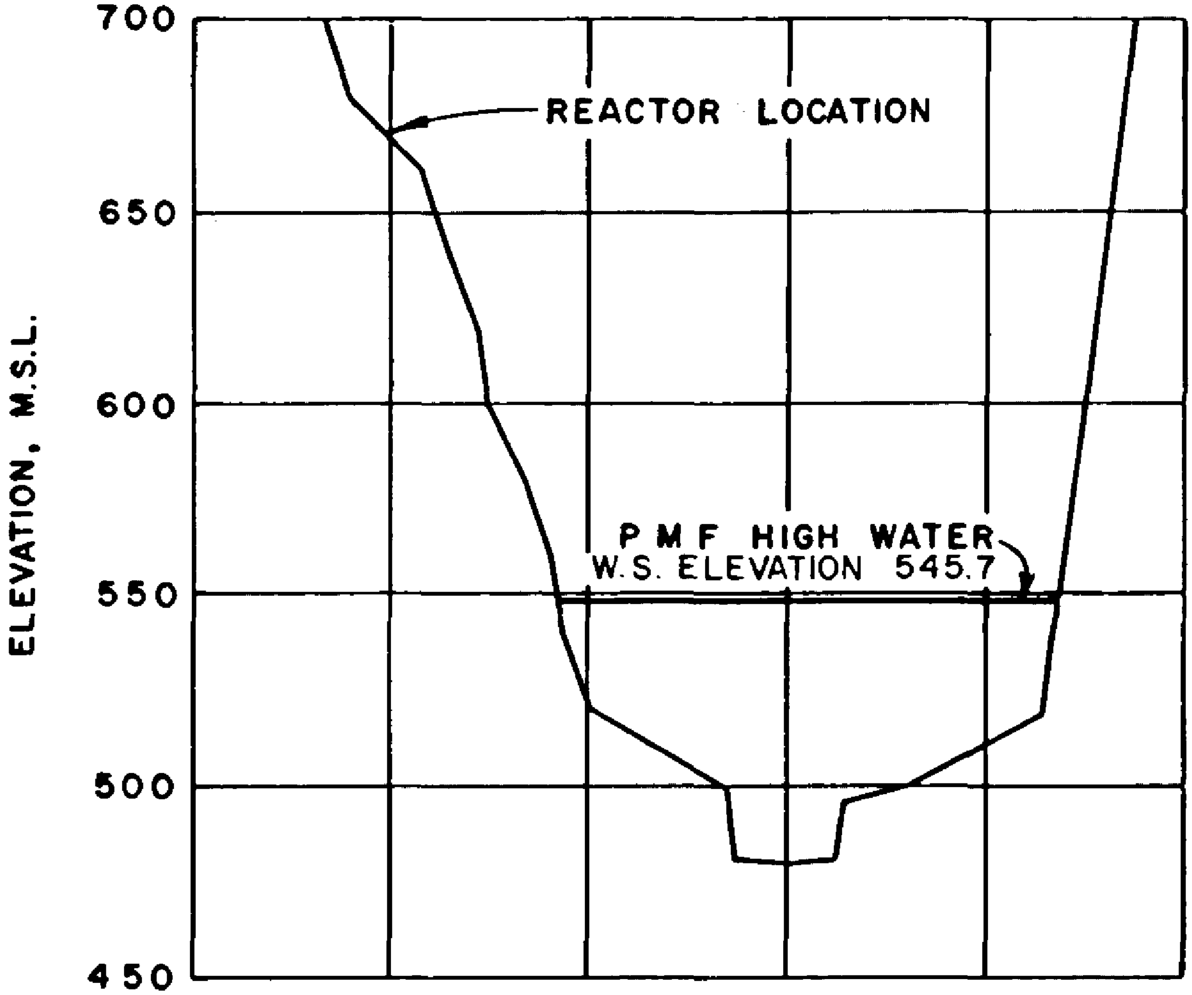
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

CROSS SECTIONS AT THE
EIGHT RIVER SECTIONS
NEAR THE SITE

FIGURE 2.4-19, Rev 47

AutoCAD: Figure Fsar 2_4_19.dwg



MILE 165.64
SECTION AT PLANT SITE

CROSS SECTIONS LOOKING UPSTREAM
SCALE - HORIZONTAL : 1" = 2000 FT.
VERTICAL : 1" = 50 FT.

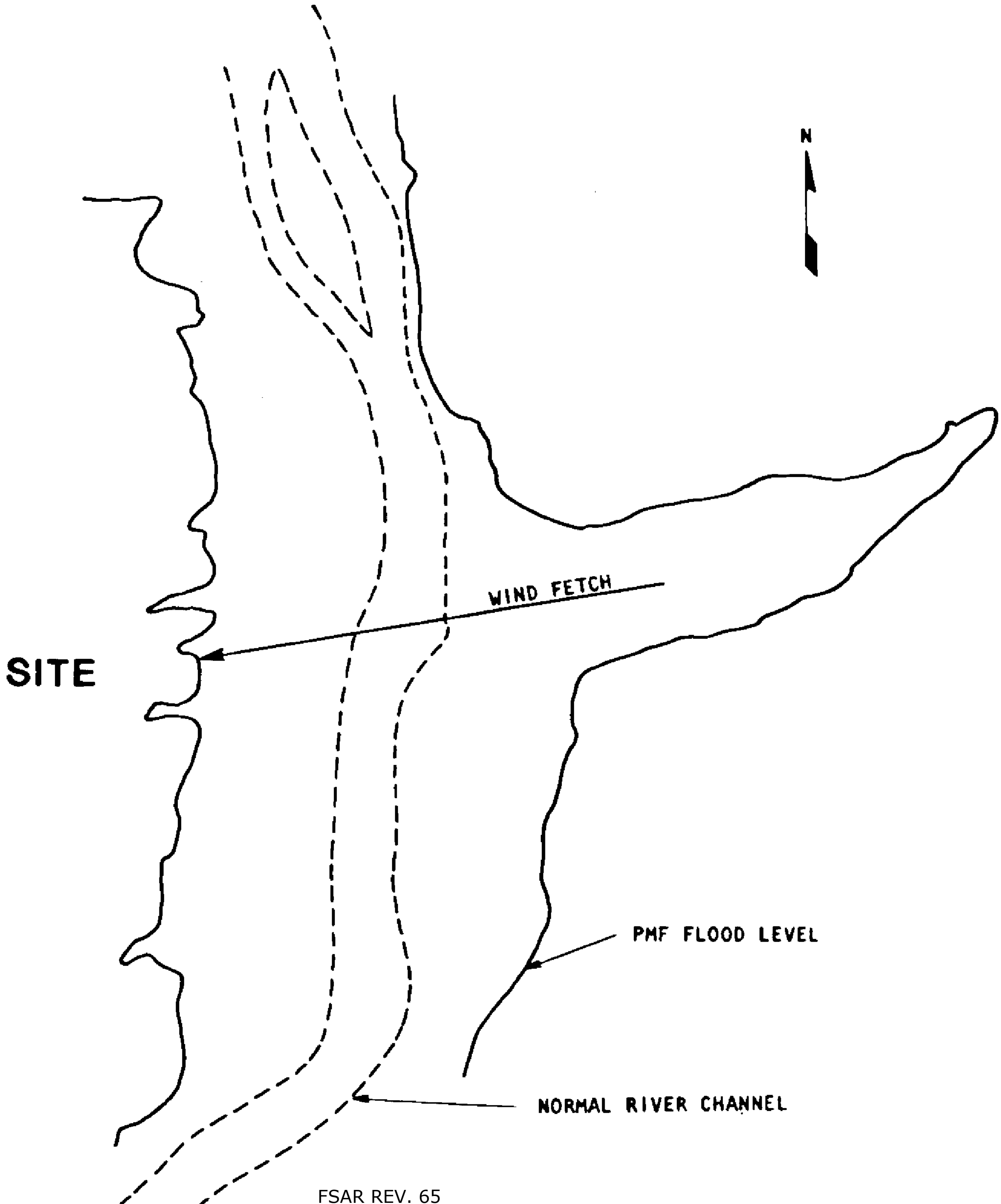
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

CROSS SECTION
OF RIVER AT
THE PLANT SITE

FIGURE 2.4-20, Rev 47

AutoCAD: Figure Fsar 2_4_20.dwg



FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

WIND FETCH,
SUSQUEHANNA RIVER PMF

FIGURE 2.4-21, Rev 47

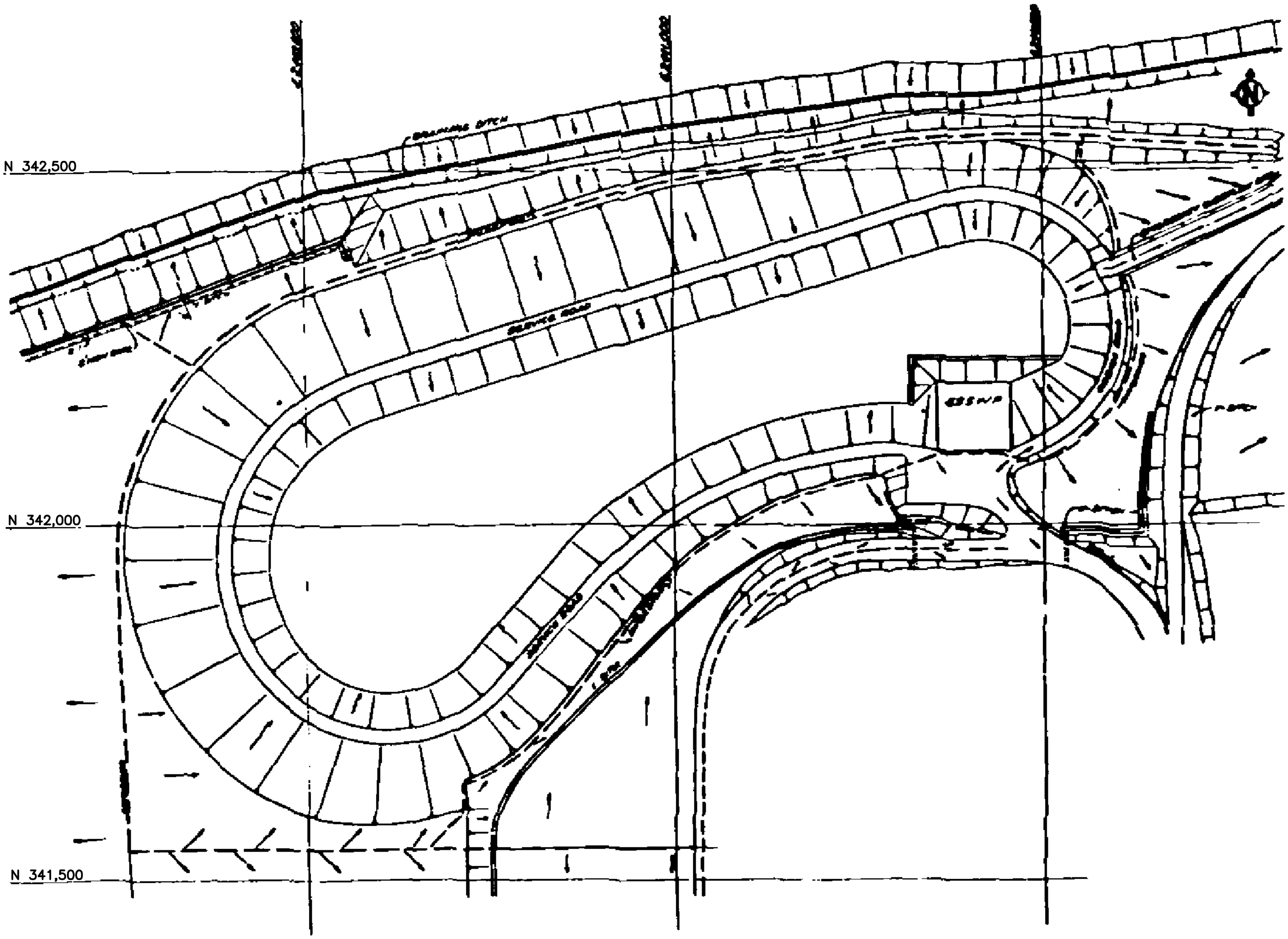
AutoCAD: Figure Fsar 2_4_21.dwg

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

TIOGA DAM & HAMMOND DAM
RESERVOIR
AREAS AND SECTIONS

FIGURE 2.4-22



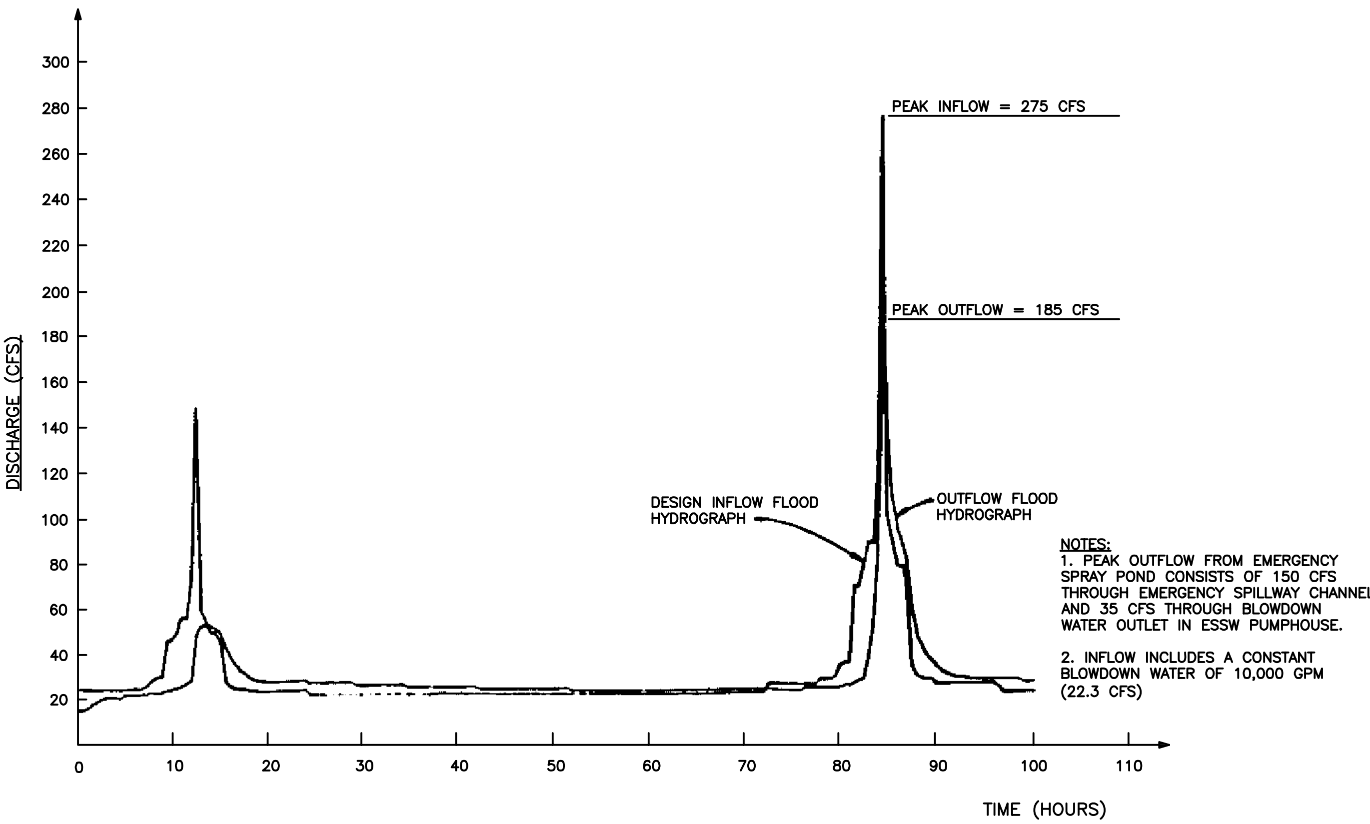
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SURFACE DRAINAGE PATTERNS
AROUND THE SPRAY POND

FIGURE 2.4-23, Rev 47

AutoCAD: Figure Fsar 2_4_23.dwg



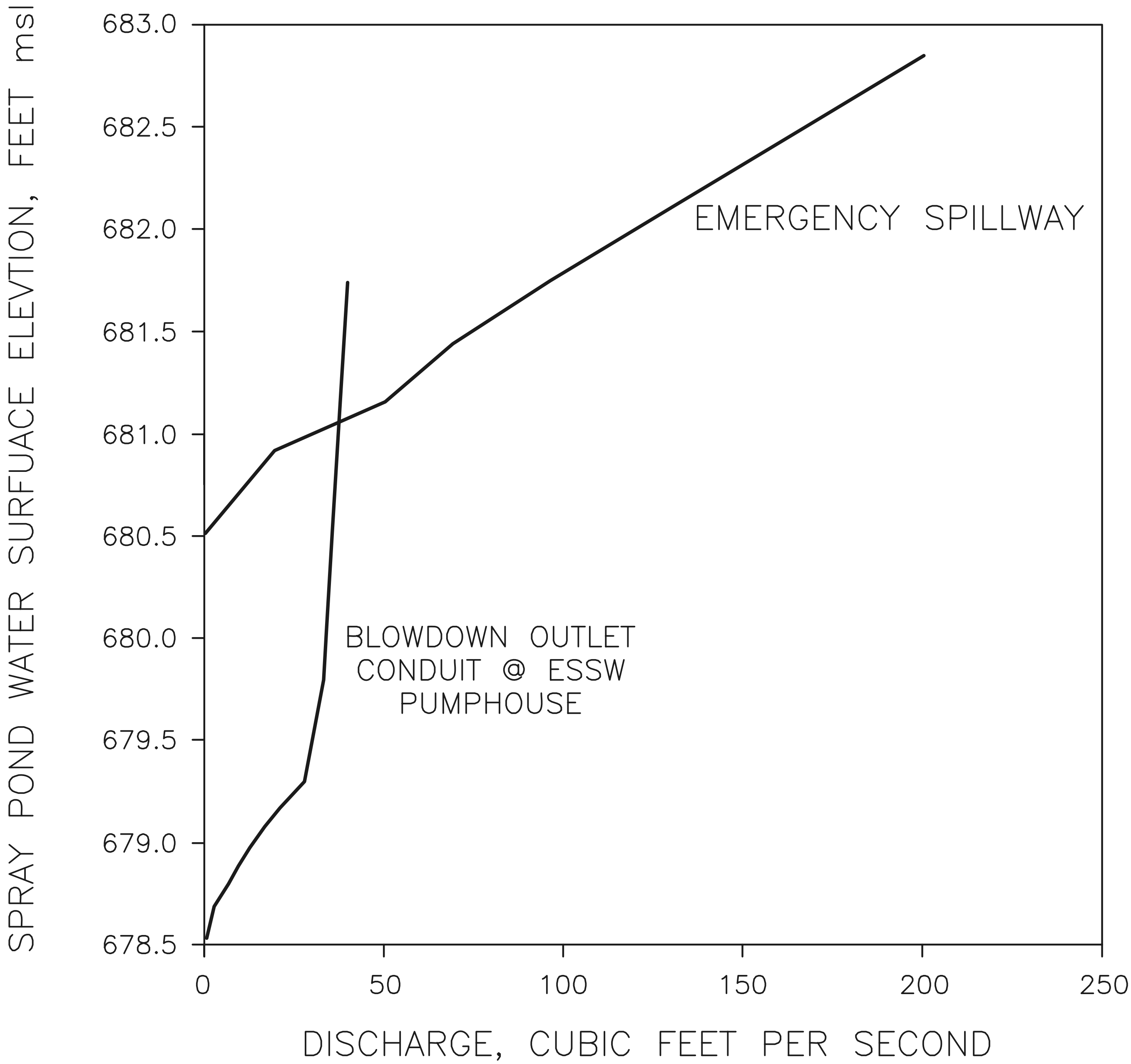
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND INFLOW
AND
OUTFLOW HYDROGRAPH

FIGURE 2.4-24, Rev 47

AutoCAD: Figure Fsar 2_4_24.dwg



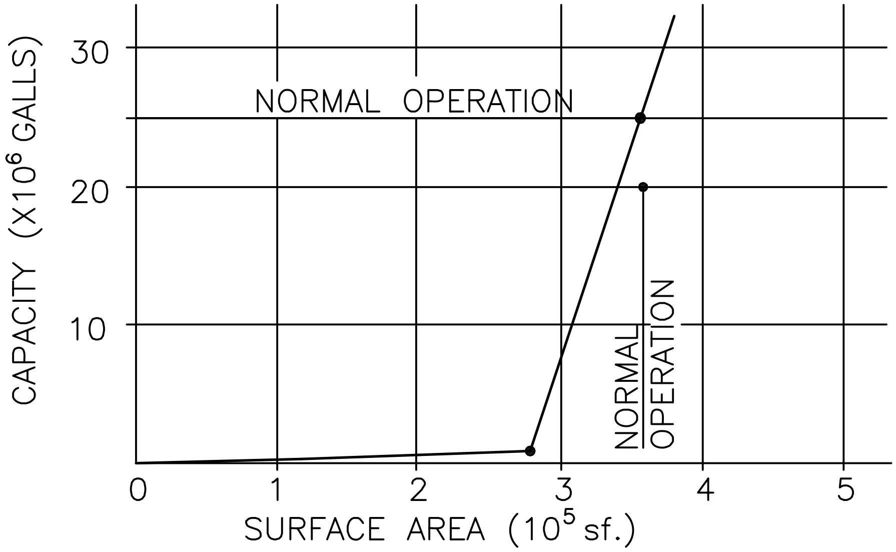
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

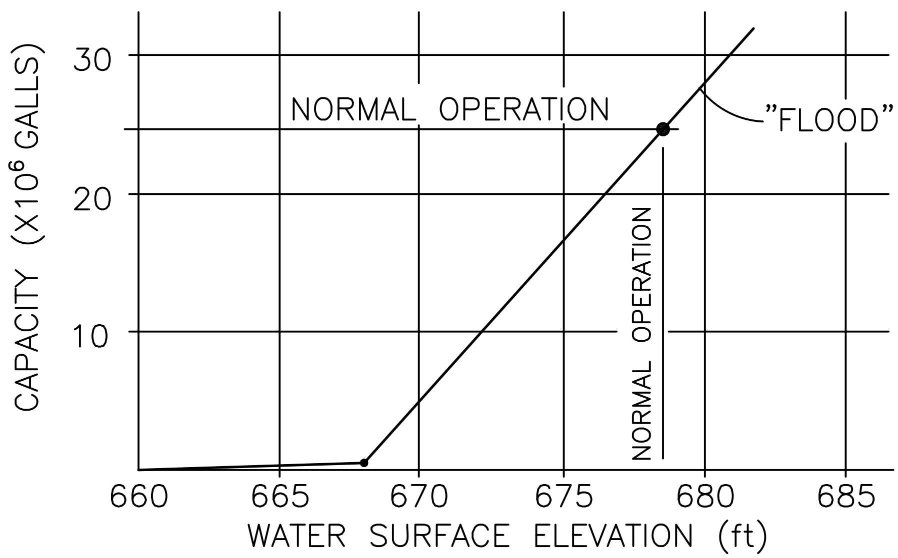
SPRAY POND OUTLET
RATING CURVES

FIGURE 2.4-25, Rev 49

AutoCAD: Figure Fsar 2_4_25.dwg



CAPACITY VS WATER SURFACE AREA CURVE



CAPACITY VS WATER ELEVATION CURVE

FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

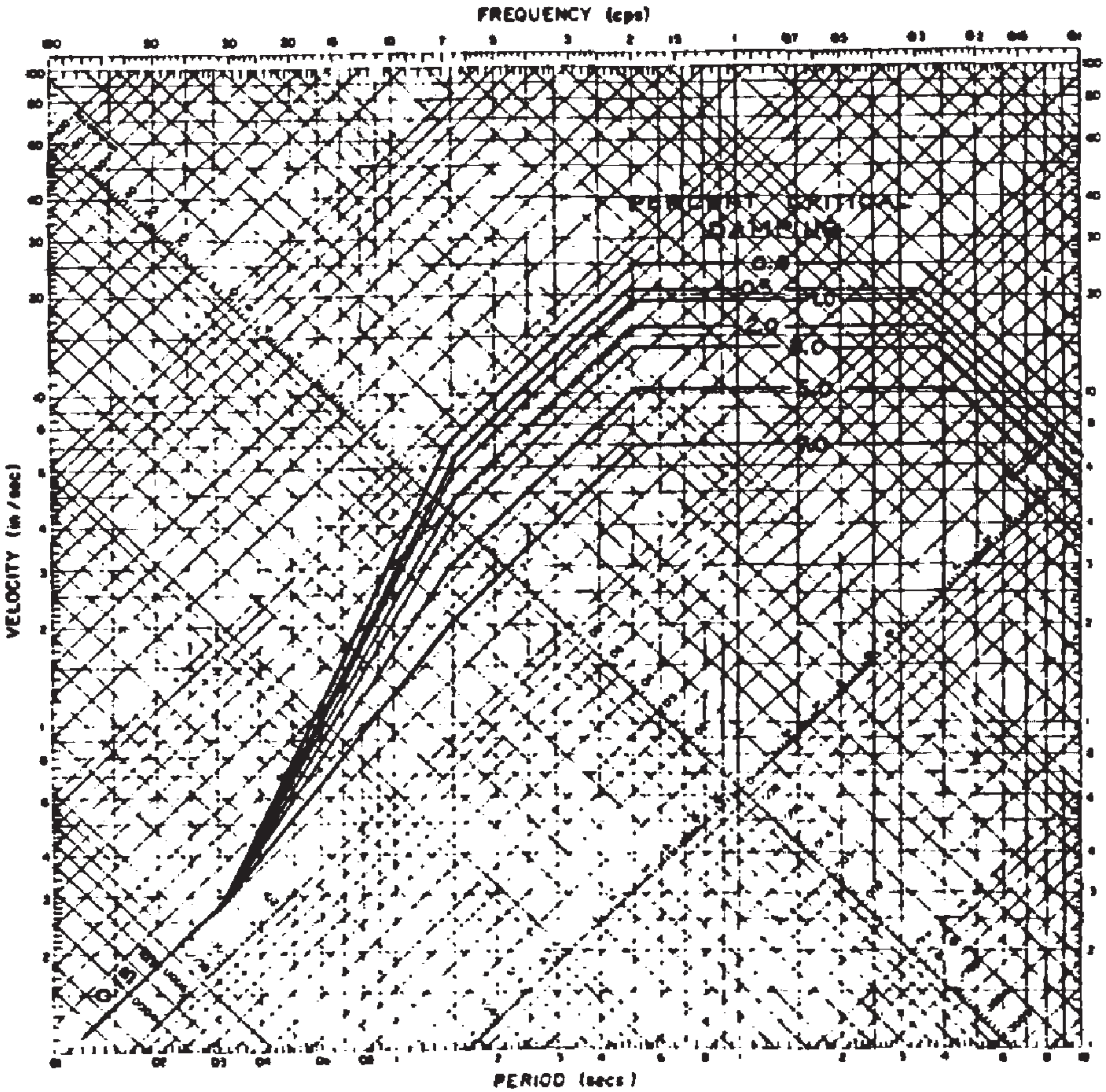
SPRAY POND ELEVATION AREA
STORAGE CAPACITY CURVES

FIGURE 2.4-26, Rev 47

AutoCAD: Figure Fsar 2_4_26.dwg

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
SPRAY POND EMERGENCY SPILLWAY WATER SURFACE PROFILE FIGURE 2.4-27



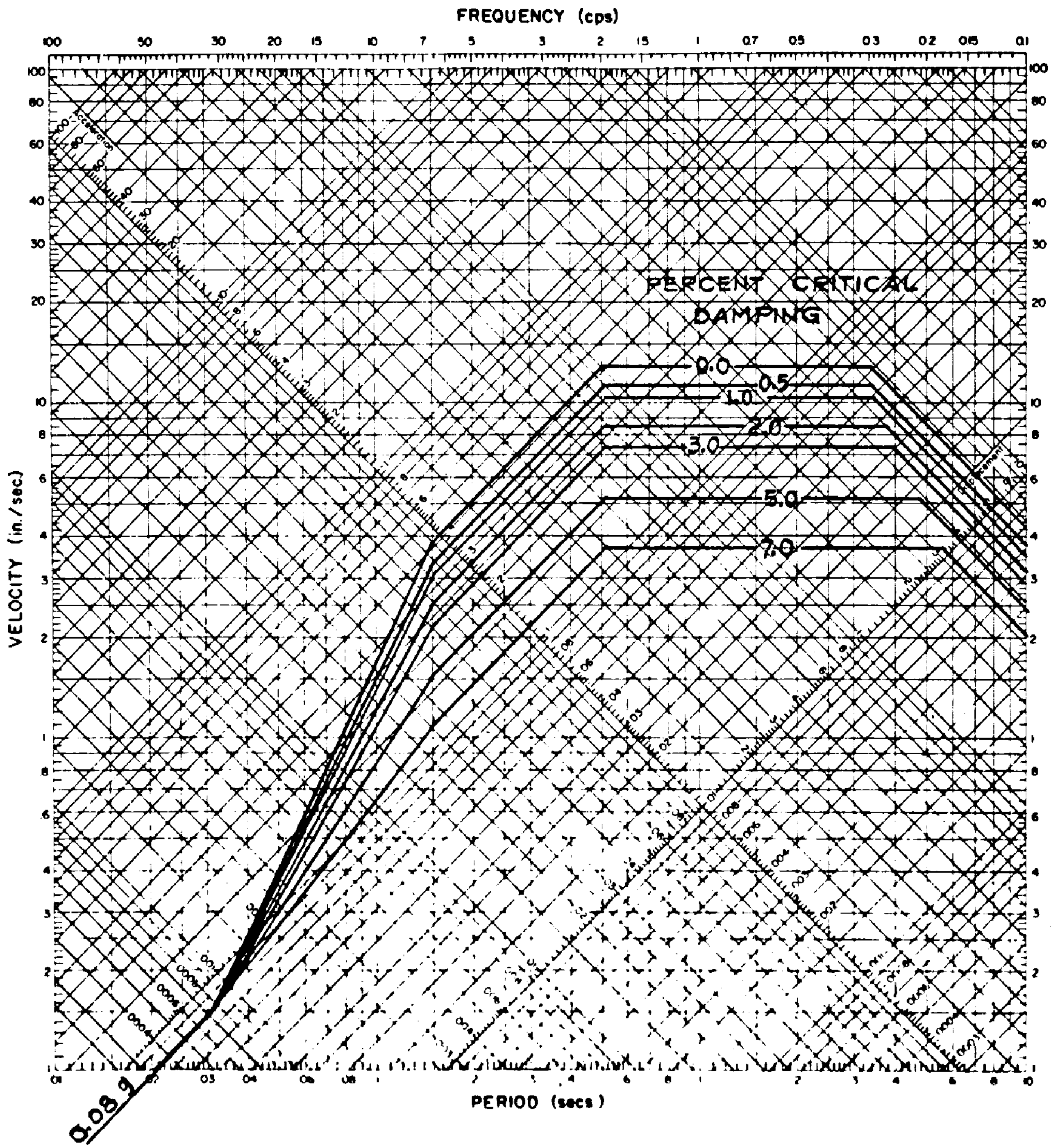
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND
DESIGN SPECTRUM
FOR SSE

FIGURE 2.4-28, Rev 47

AutoCAD: Figure Fsar 2_4_28.dwg



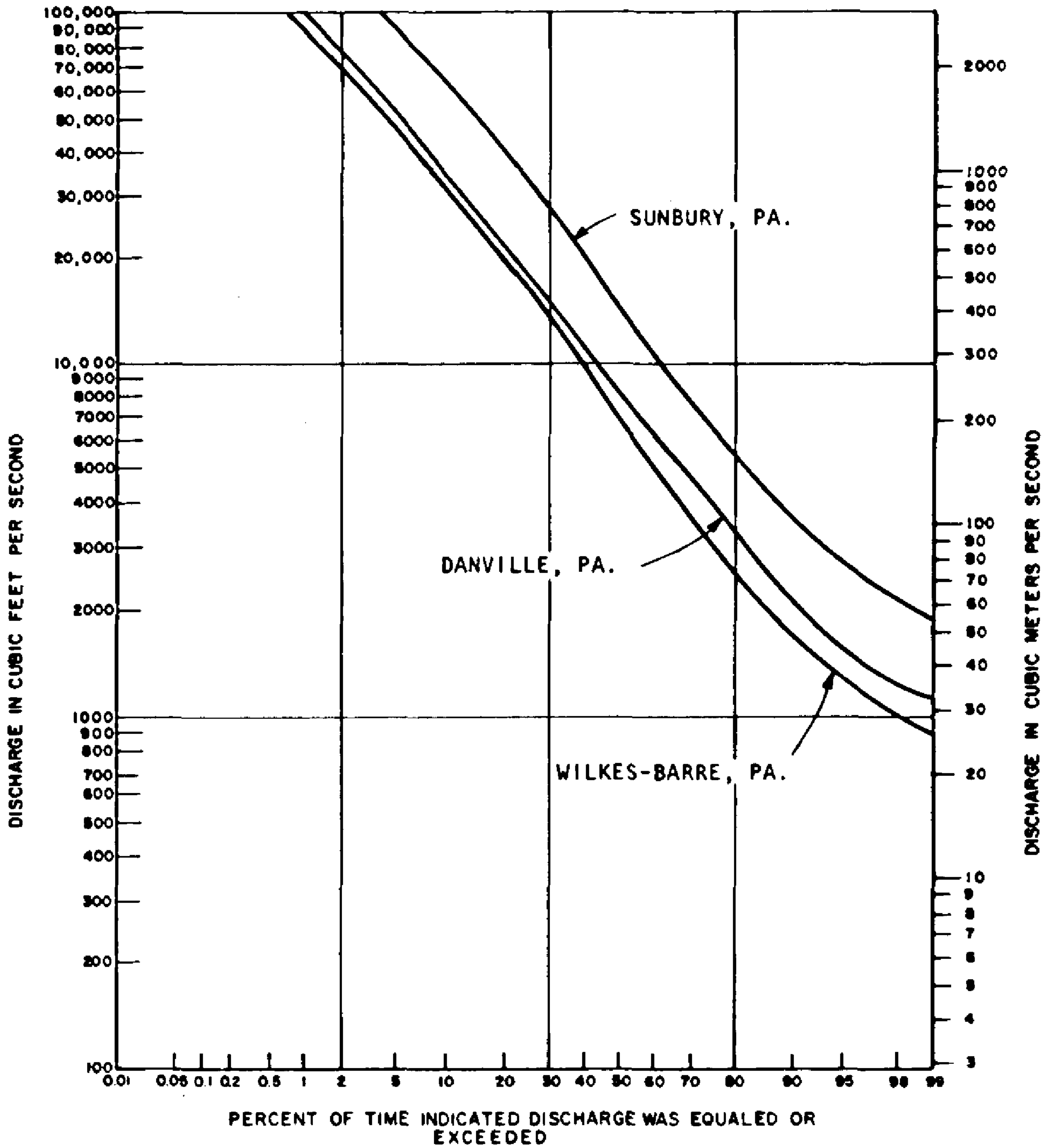
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND
DESIGN SPECTRUM
FOR OBE

FIGURE 2.4-29, Rev 47

AutoCAD: Figure Fsar 2_4_29.dwg



SOURCE : USGS OPEN FILE REPORT 76-247 , 1976

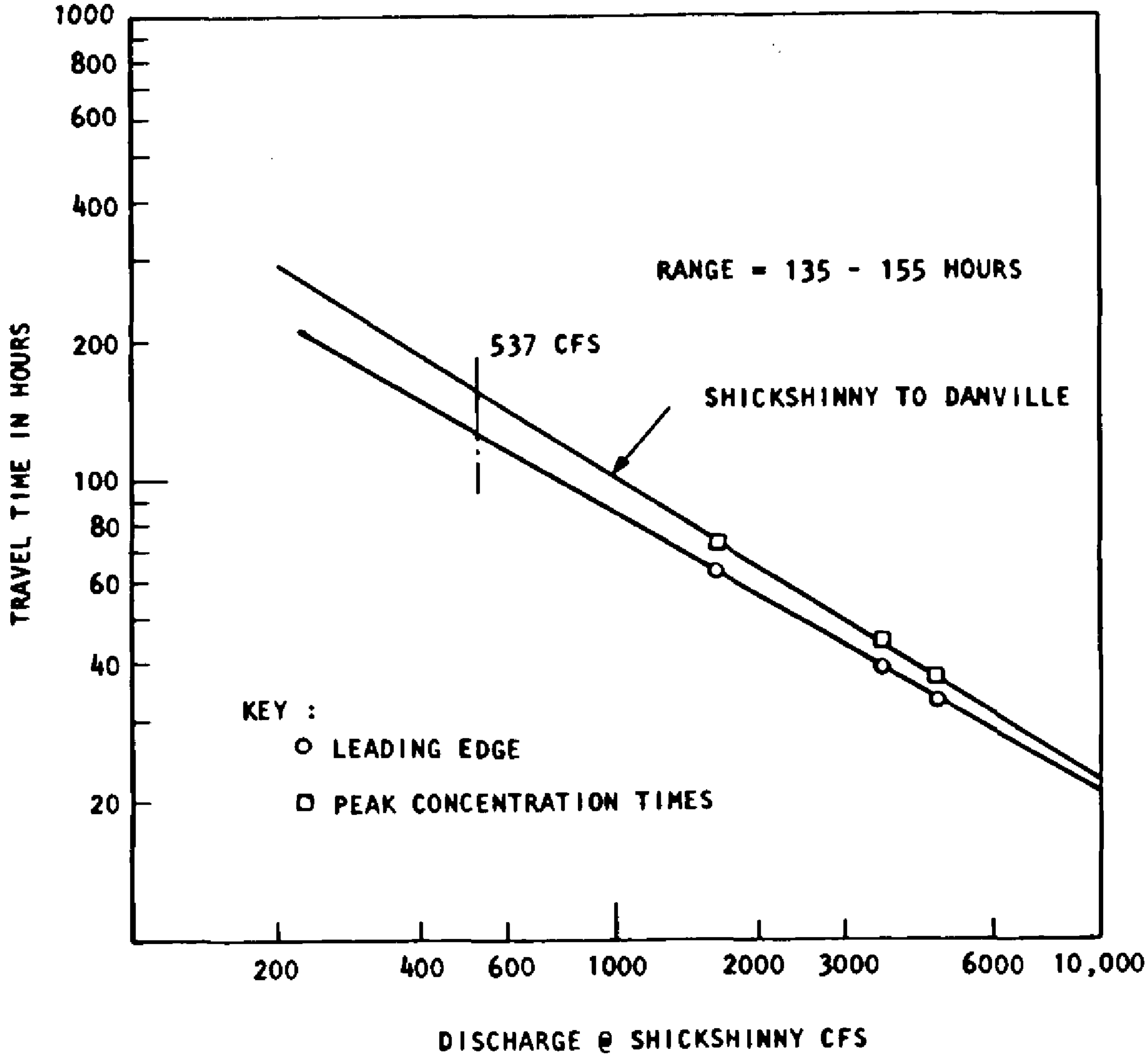
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

DURATION CURVES OF
DAILY DISCHARGE

FIGURE 2.4-30, Rev 47

AutoCAD: Figure Fsar 2_4_30.dwg



SOURCE: USGS, 1976

FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

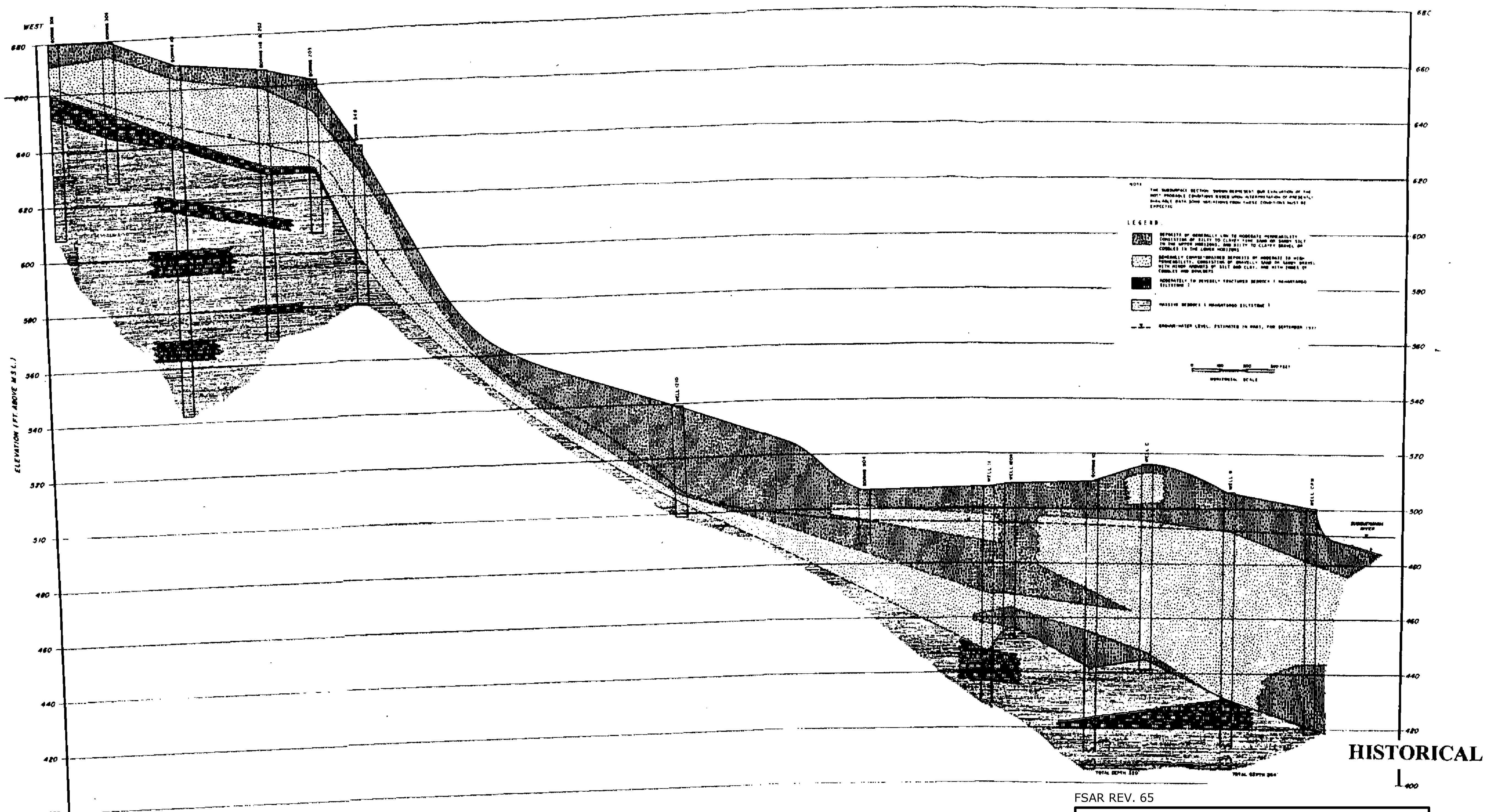
TIME OF TRAVEL
SUSQUEHANNA RIVER
SHICKSHINNY TO DANVILLE

FIGURE 2.4-31, Rev 47

AutoCAD: Figure Fsar 2_4_31.dwg

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
MAP AT SUSQUEHANNA SES SHOWING GROUNDWATER CONTOURS IN SEPTEMBER 1977 FIGURE 2.4-32



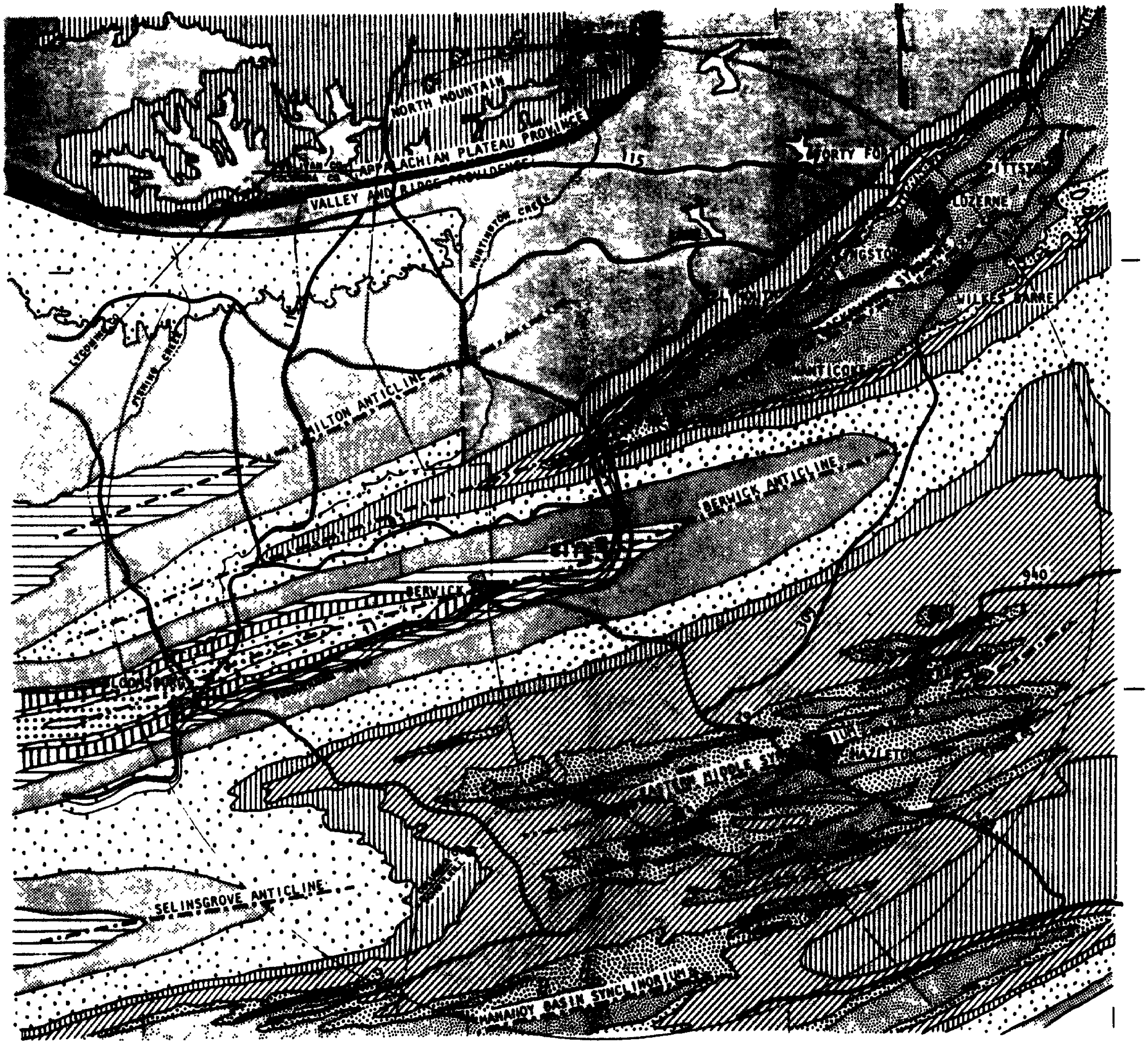
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

HYDROLOGIC CROSS SECTION
FROM WEST TO EAST ALONG
GROUNDWATER FLOW PATH
AT THE SUSQUEHANNA SES

FIGURE 2.4-33, Rev 55

AutoCAD: Figure Fsar 2_4_33.dwg



LEGEND:

PENNSYLVANIAN

- LLEWELLYN FORMATION
- POTTSVILLE FORMATION

MISSISSIPPIAN

- MAUCH CHUNK FORMATION
- POCONO FORMATION

DEVONIAN

- CATSKILL FORMATION } (UNDIFFERENTIATED)
- MARINE BEDS
- MAHANTANGO, MARCELLUS AND ONONDAGA FMS (UNDIFFERENTIATED)

SILURIAN

- KEYSER, TONOLOWAY AND WILLS CREEK FORMATIONS
- BLOOMSBURG FORMATION
- CLINTON FORMATION
- MAJOR ANTICLINAL OR SYNCLINAL AXIS
- DIVISION BETWEEN PHYSIOGRAPHIC PROVINCES

0 5 10

MILES

HISTORICAL

BASED ON:
C. GRAY, V.C. SHEPPS, AND OTHERS, GEOLOGIC MAP OF PENNSYLVANIA, 1:250,000 HARRISBURG, PA: PA. DEPARTMENT OF ENVIRONMENTAL RESOURCES, TOPOGRAPHIC AND GEOLOGIC SURVEY, MAP 1, (1960), WITH CERTAIN MODIFICATIONS BASED ON FIELD MAPPING WITHIN 5 MILES OF STATION PERFORMED BY DAMES AND MOORE IN SPRING OF 1977.

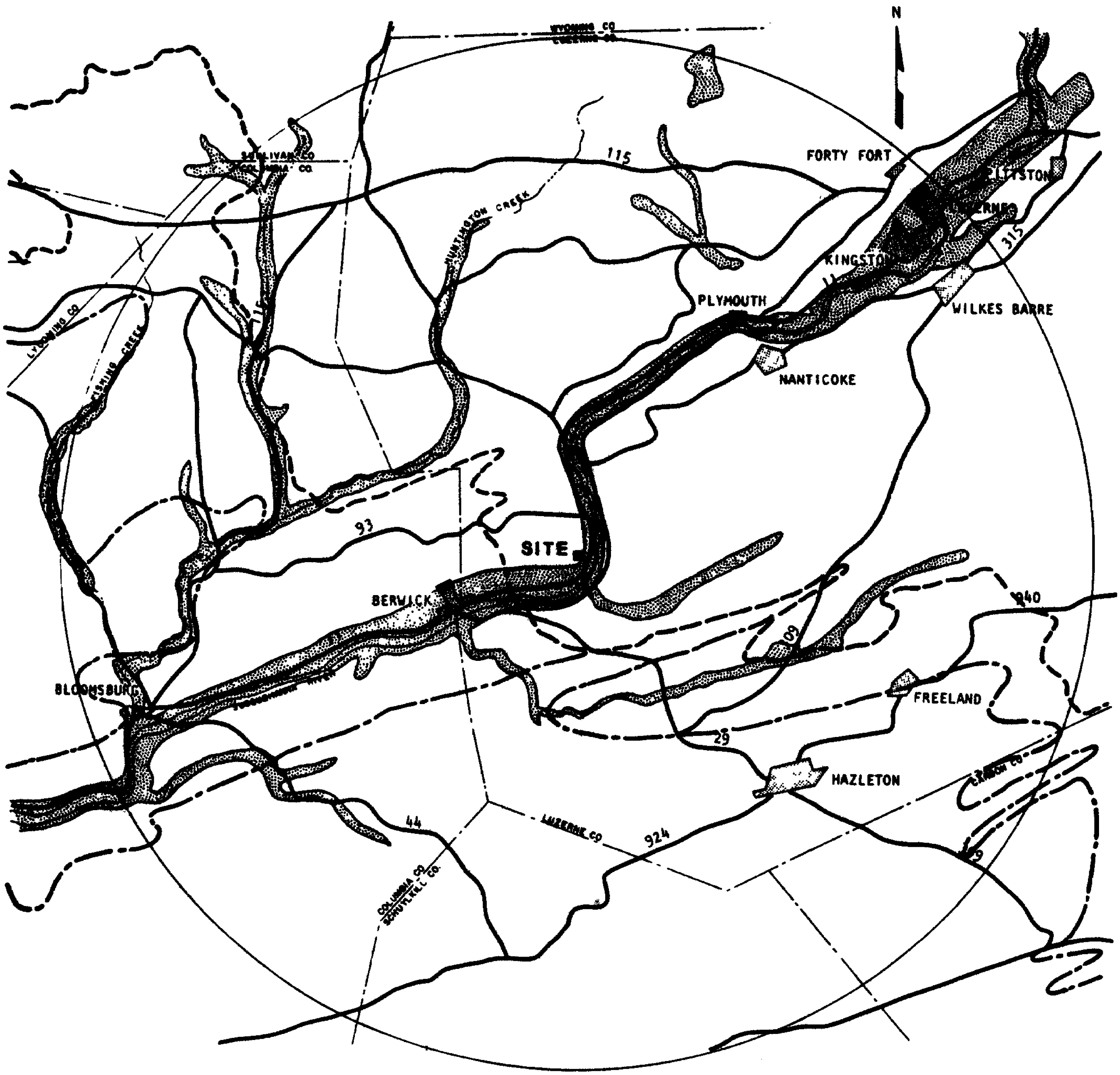
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

BEDROCK GEOLOGIC MAP OF AREA
WITHIN 20 MILES OF
SUSQUEHANNA SES

FIGURE 2.4-34, Rev 55

AutoCAD: Figure Fsar 2_4_34.dwg



LEGEND:

BORDER OF WISCONSIN DRIFT

BORDER OF ILLINOIAN DRIFT

PLEISTOCENE SAND AND GRAVEL DEPOSITS

0 5 10

MILES

REFERENCE:
P. SEABER, AN APPRAISAL OF THE GROUND-WATER RESOURCES OF THE UPPER SUSQUEHANNA RIVER BASIN IN PA. (AN INTERIM REPORT), U.S.G.S., WATER RESOURCES DIVISION, (1968). FIGURE 3 OF THIS REFERENCE.

HISTORICAL

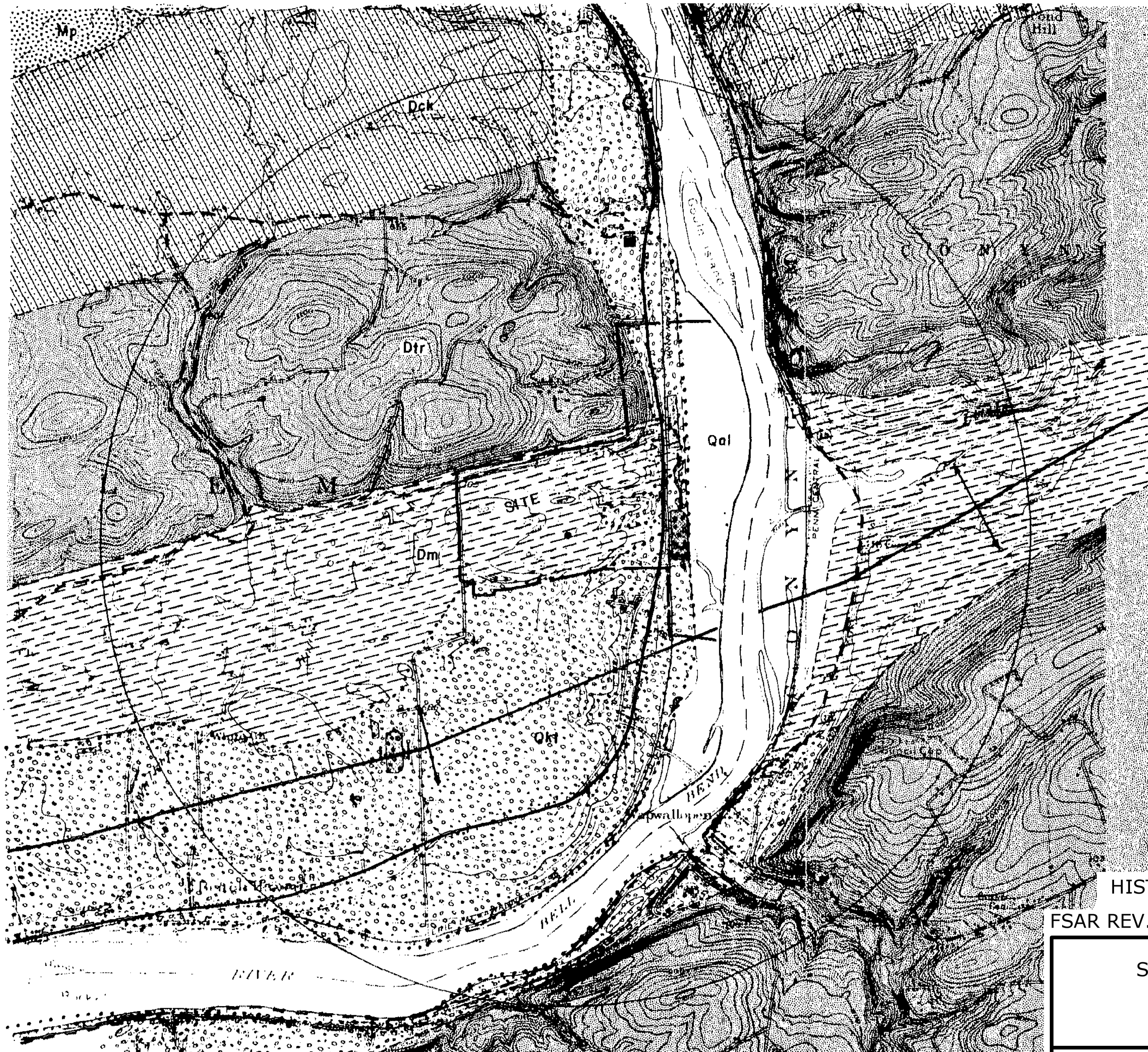
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

MAJOR PLEISTOCENE SAND &
GRAVEL DEPOSITS WITHIN 20
MILES OF SUSQUEHANNA SES

FIGURE 2.4-35, Rev 55

AutoCAD: Figure Fsar 2_4_35.dwg



REFERENCE :

THE BASE FOR THIS MAP WAS PREPARED FROM THE FOLLOWING 7.5 MINUTE U.S.G.S. TOPOGRAPHIC QUADRANGLES: BERWICK PA., 1969; MIFFLINVILLE, PA., 1969; SYBERTSVILLE,PA. 1969. CONTACTS WERE DETERMINED DURING FIELD MAPPING PERFORMED BY DAMES & MOORE IN SPRING OF 1977.

SYMBOL :

STATION PROPERTY BOUNDARY

APPROXIMATE STATRIGRAPHIC CONTACT

BERWICK ANTICLINE

LEGEND :

Qal QUATERNARY ALLUVIUM

Qkt PLEISTOCENE KAME TERRACE

MISSISSIPIAN

Mp POCONO FORMATION

DEVONIAN

Dck CATSKILL FORMATION

Dtr TRIMMERS ROCK FORMATION

Dm MAHANTANGO FORMATION

1/2 0 1
MILE
1 0 1
KM

HISTORICAL

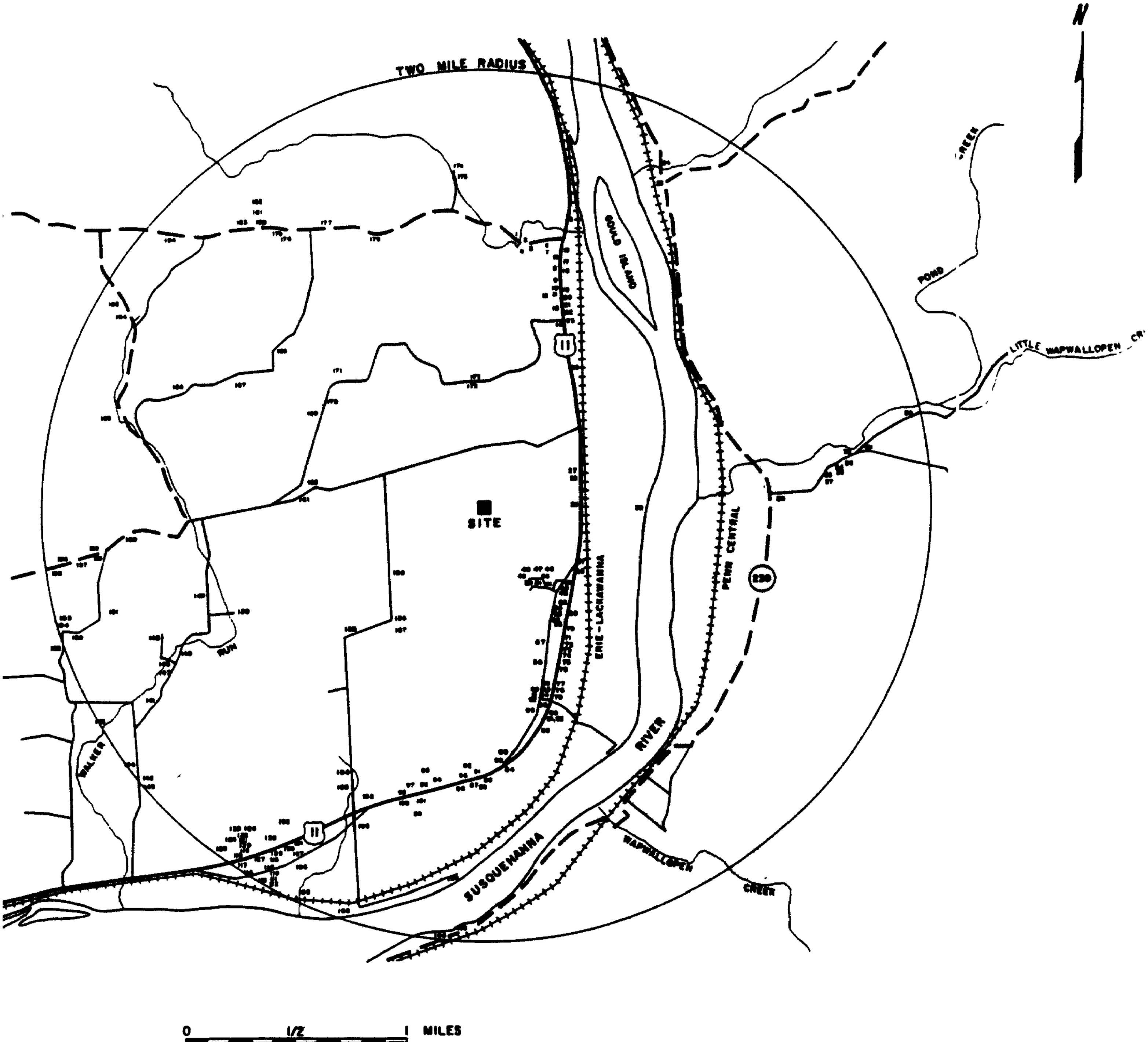
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

GEOLOGIC MAP OF AREA
WITHIN TWO MILES
OF THE STATION

FIGURE 2.4-36, Rev 55

AutoCAD: Figure Fsar 2_4_36.dwg



HISTORICAL

FSAR REV. 65

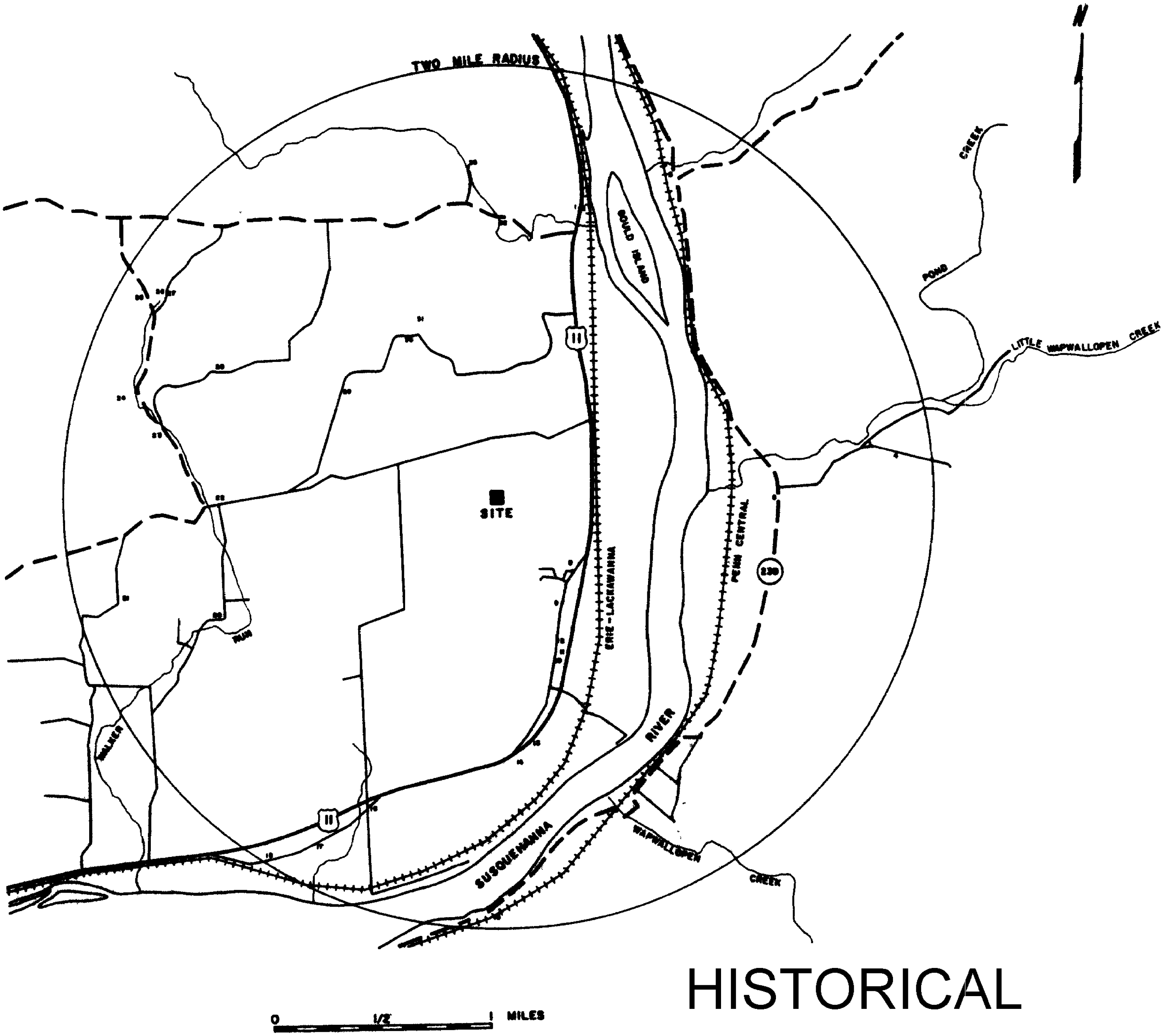
NOTE:
NUMBERS INDICATE APPROXIMATE LOCATION OF WELLS

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

WATER WELLS
WITHIN TWO MILES
OF THE STATION

FIGURE 2.4-37, Rev 55

AutoCAD: Figure Fsar 2_4_37.dwg



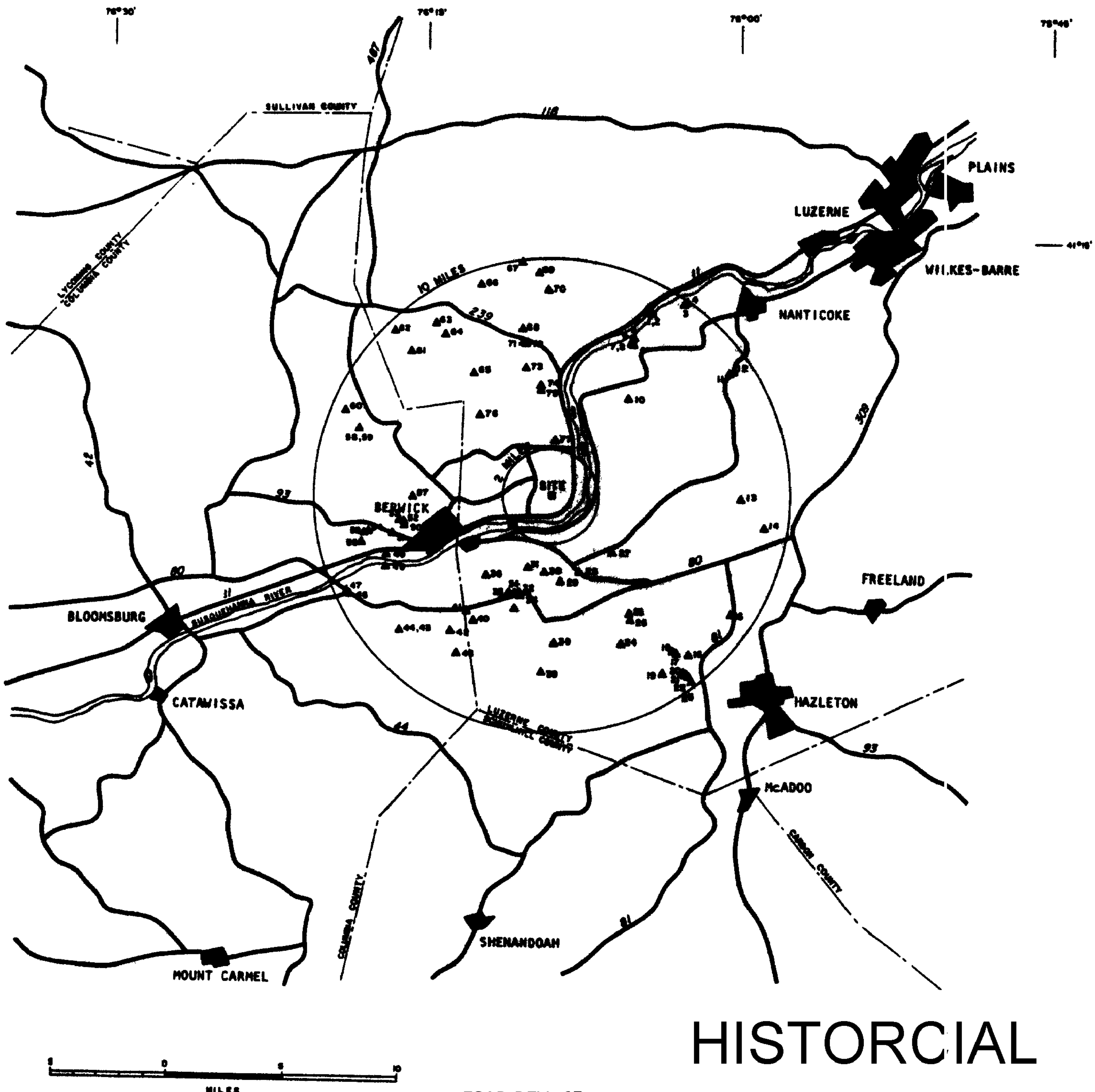
HISTORICAL

NOTE:
NUMBERS INDICATE APPROXIMATE LOCATION OF SPRINGS

FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
SPRINGS USED FOR WATER SUPPLY WITHIN TWO MILES OF THE STATION FIGURE 2.4-38, Rev 55

AutoCAD: Figure Fsar 2_4_38.dwg



HISTORCIAL

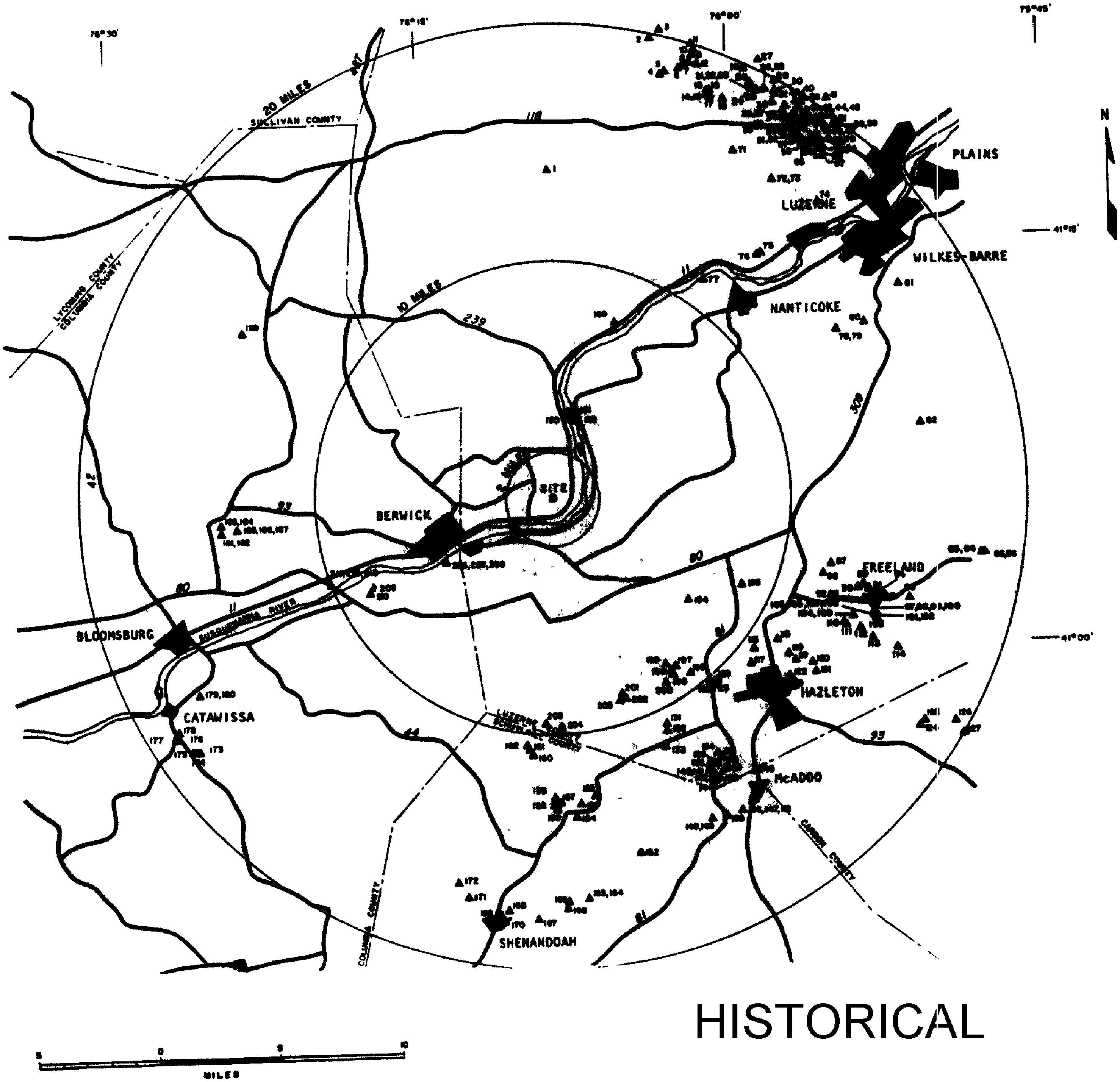
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

MAJOR WATER WELLS
TWO TO TEN MILES FROM
THE STATION
EXCEPTING PUBLIC SUPPLY WELLS

FIGURE 2.4-39, Rev 55

AutoCAD: Figure Fsar 2_4_39.dwg



HISTORICAL

FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

PUBLIC SUPPLY WELLS
TWO TO TWENTY MILES
FROM THE STATION

FIGURE 2.4-40, Rev 55

AutoCAD: Figure Fsar 2_4_40.dwg

Security-Related Information
Figure Withheld Under 10 CFR 2.390

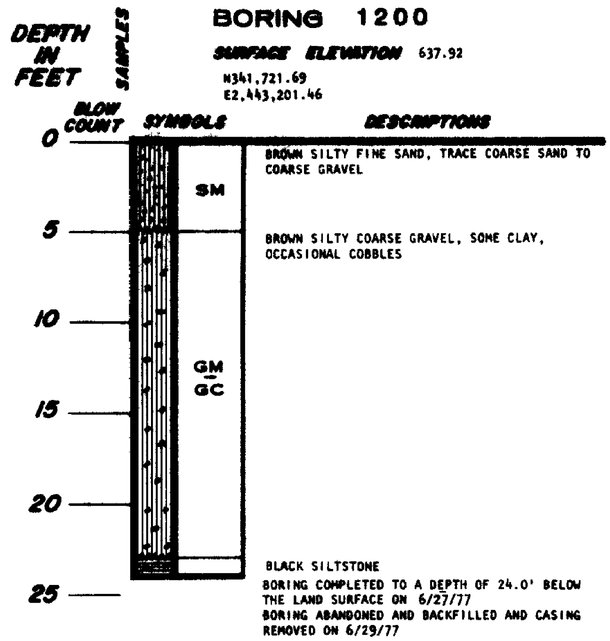
SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
MAP OF SUSQUEHANNA SES SHOWING ISOPACH CONTOURS OF OVERBURDEN THICKNESS FIGURE 2.4-41

Security-Related Information
Figure Withheld Under 10 CFR 2.390

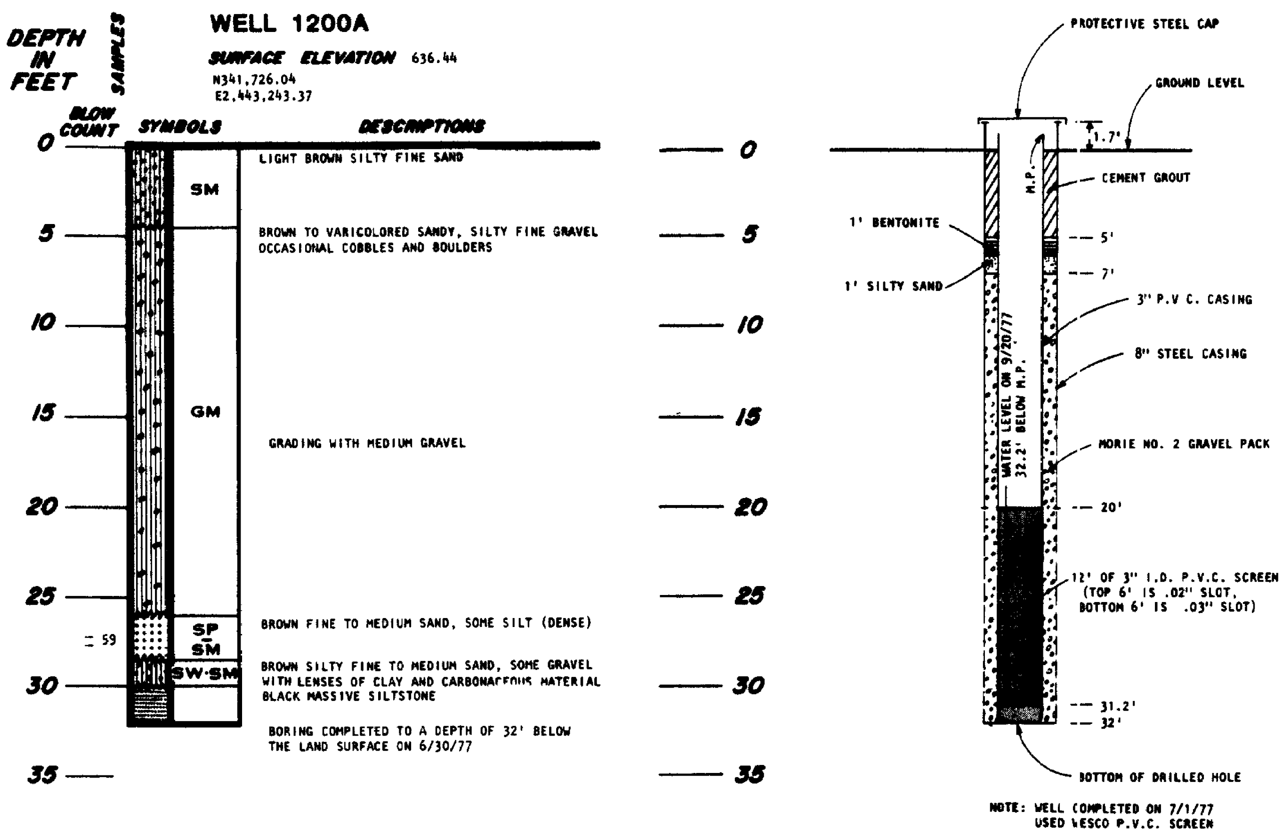
SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
MAP OF SUSQUEHANNA SES SHOWING TOP-OF-BEDROCK CONTOURS FIGURE 2.4-42

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
MAP OF SUSQUEHANNA SES SHOWING GROUNDWATER CONTOURS IN APRIL 1977 FIGURE 2.4-43



LOG OF BORING 1200



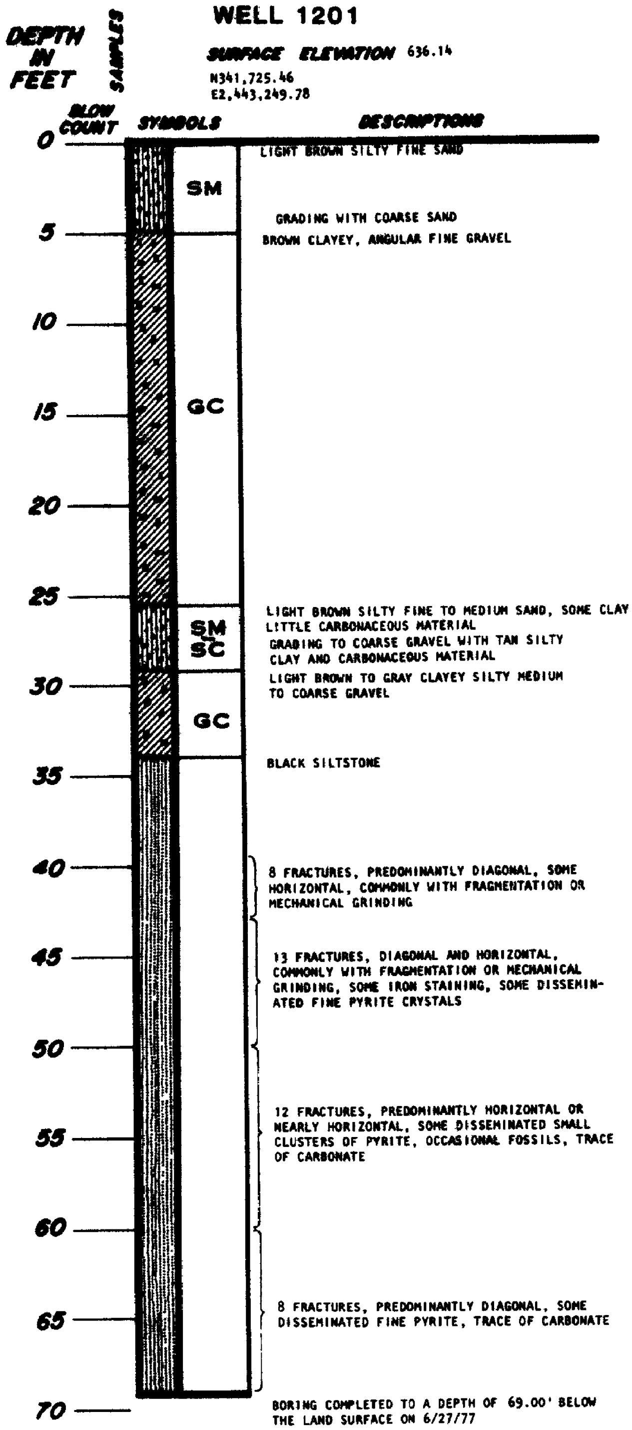
LOG OF WELL 1200A

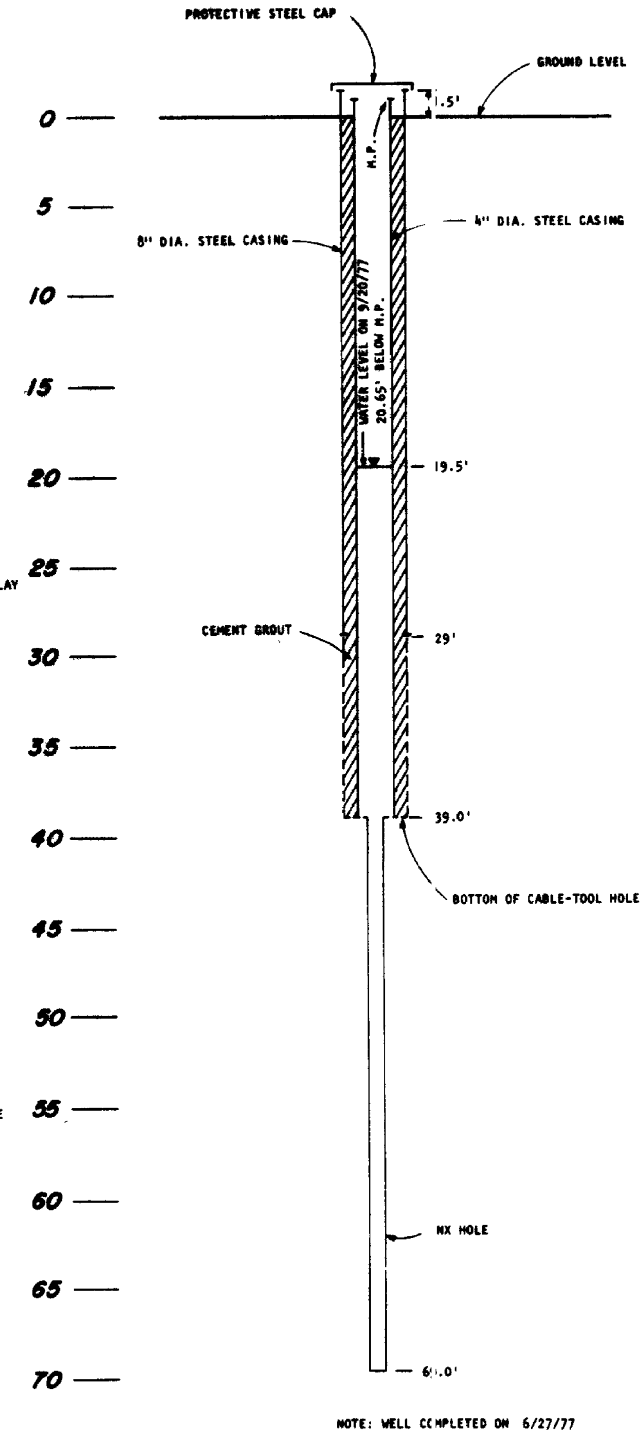
HISTORICAL

FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
LOG OF BORING AND OBSERVATION WELL CONSTRUCTION DETAILS - 1200, 1200A FIGURE 2.4-44, Rev 55

AutoCAD: Figure Fsar 2_4_44.dwg





LOG OF WELL 1201

CONSTRUCTION DETAILS OF WELL 1201

HISTORICAL

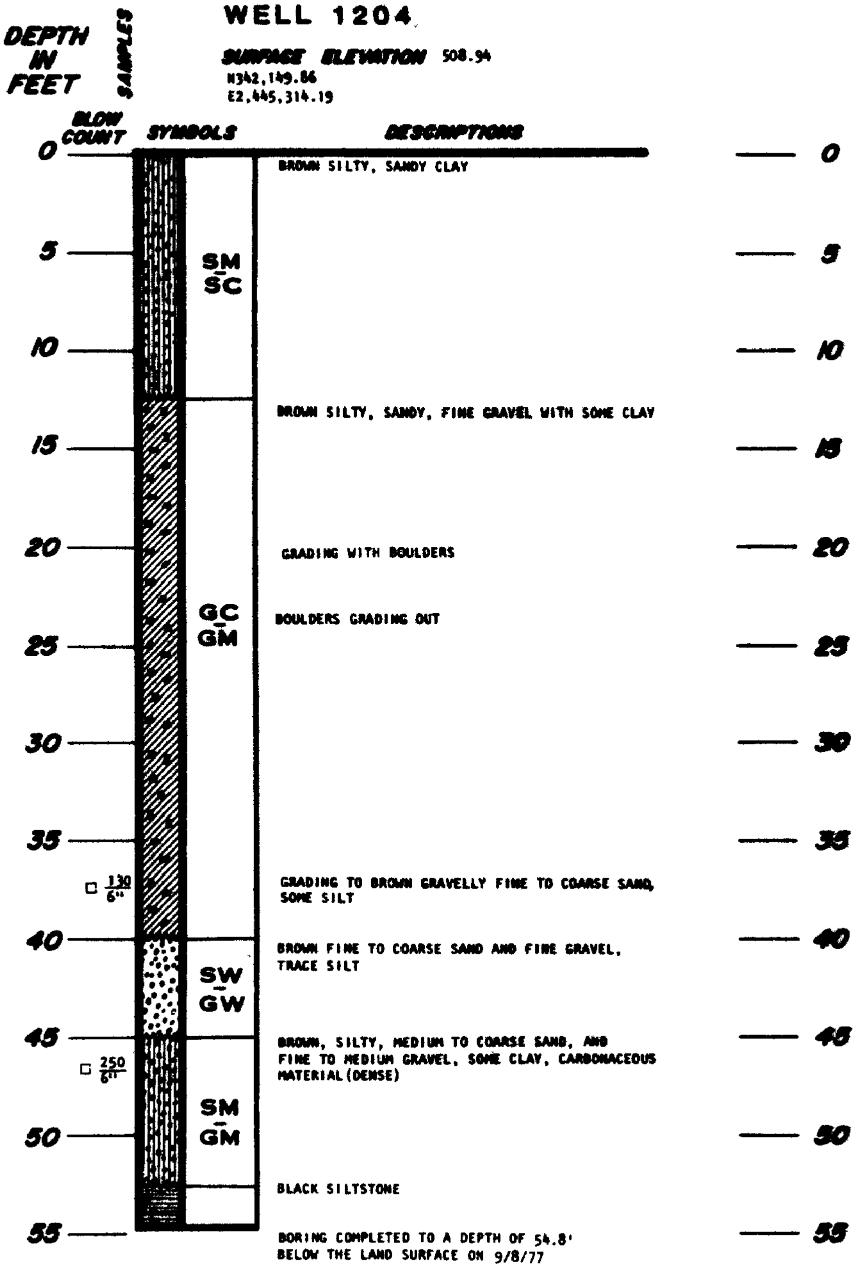
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

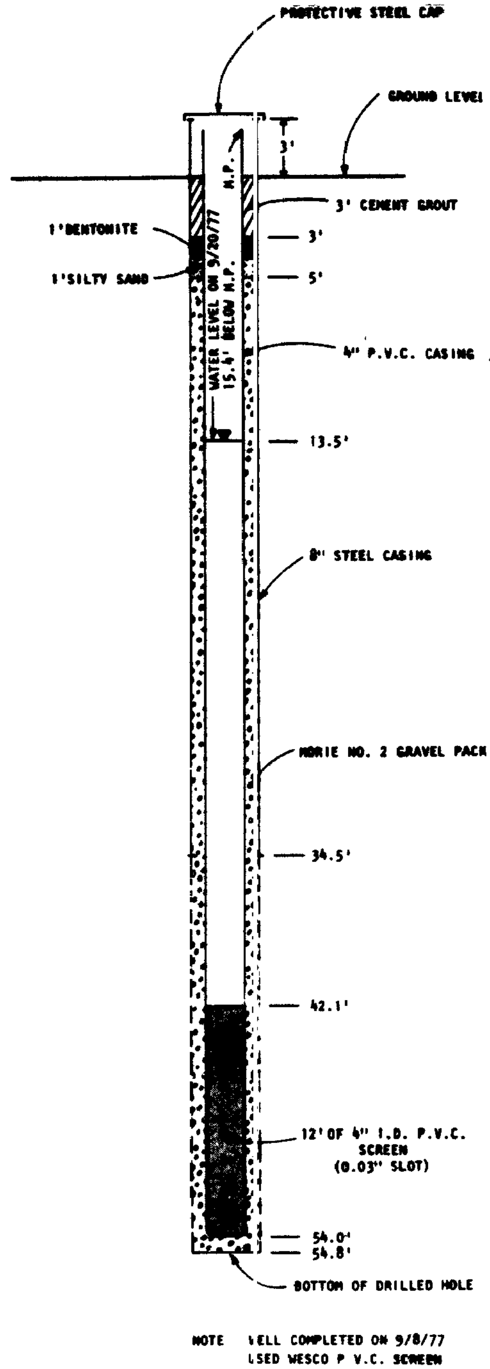
LOG OF BORING AND OBSERVATION WELL
CONSTRUCTION DETAILS - 1201

FIGURE 2.4-45, Rev 55

AutoCAD: Figure Fsar 2_4_45.dwg



LOG OF WELL 1204



CONSTRUCTION DETAILS OF WELL 1204

HISTORICAL

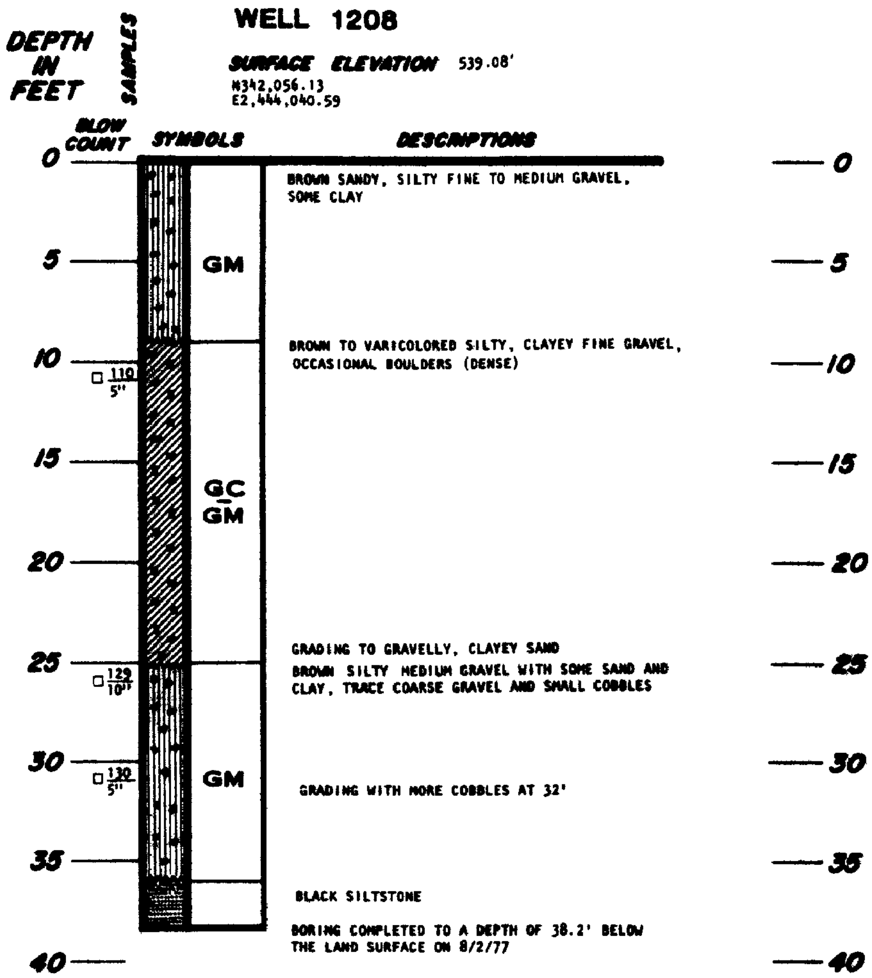
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

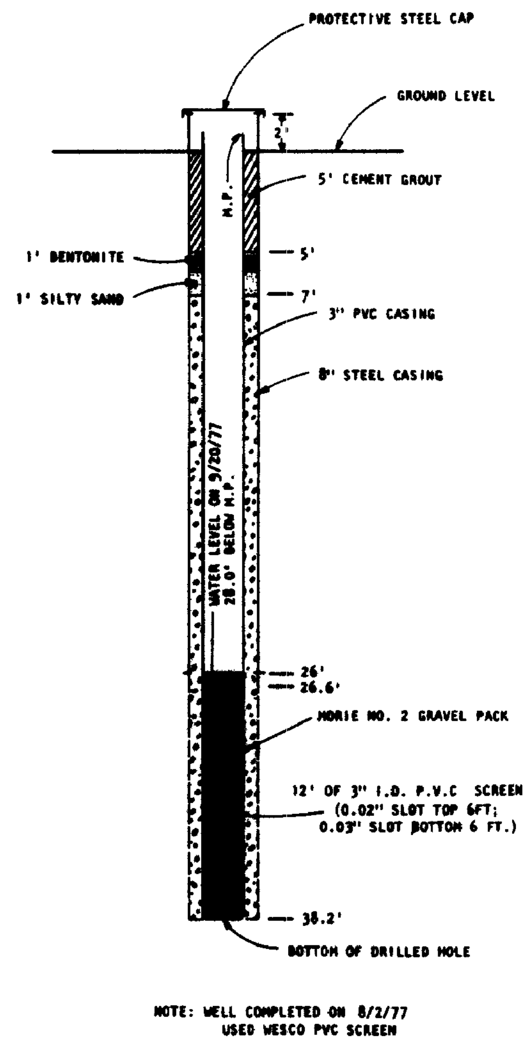
LOG OF BORING AND OBSERVATION WELL
CONSTRUCTION DETAILS - 1204

FIGURE 2.4-46, Rev 55

AutoCAD: Figure Fsar 2_4_46.dwg



LOG OF WELL 1208



CONSTRUCTION DETAILS OF WELL 1208

HISTORICAL

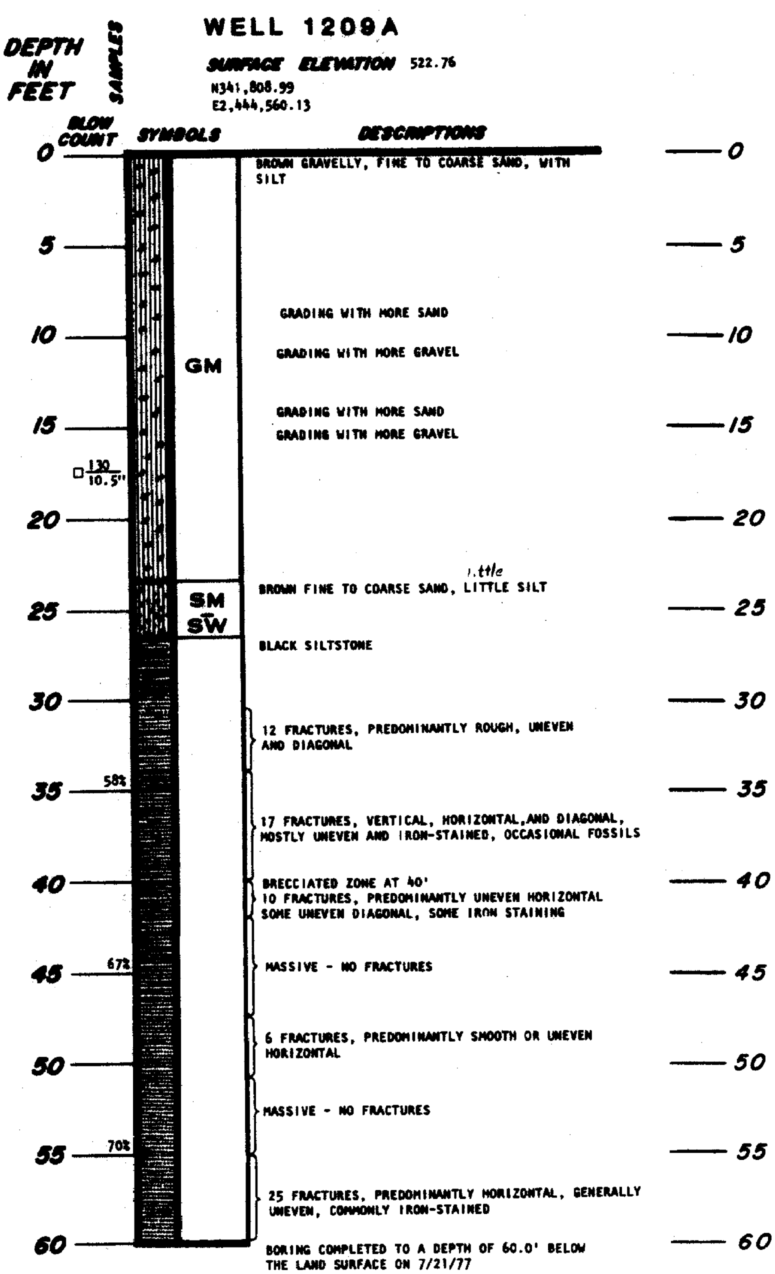
FSAR REV. 65

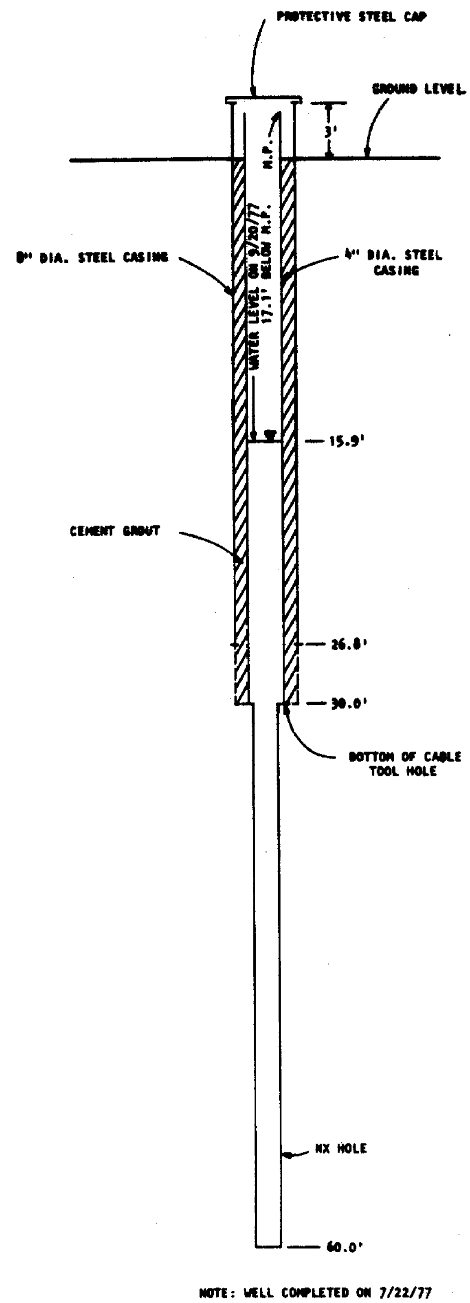
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORING AND OBSERVATION WELL
CONSTRUCTION DETAILS - 1208

FIGURE 2.4-47, Rev 55

AutoCAD: Figure Fsar 2_4_47.dwg





LOG OF WELL 1209A

CONSTRUCTION DETAILS OF WELL 1209A

HISTORICAL

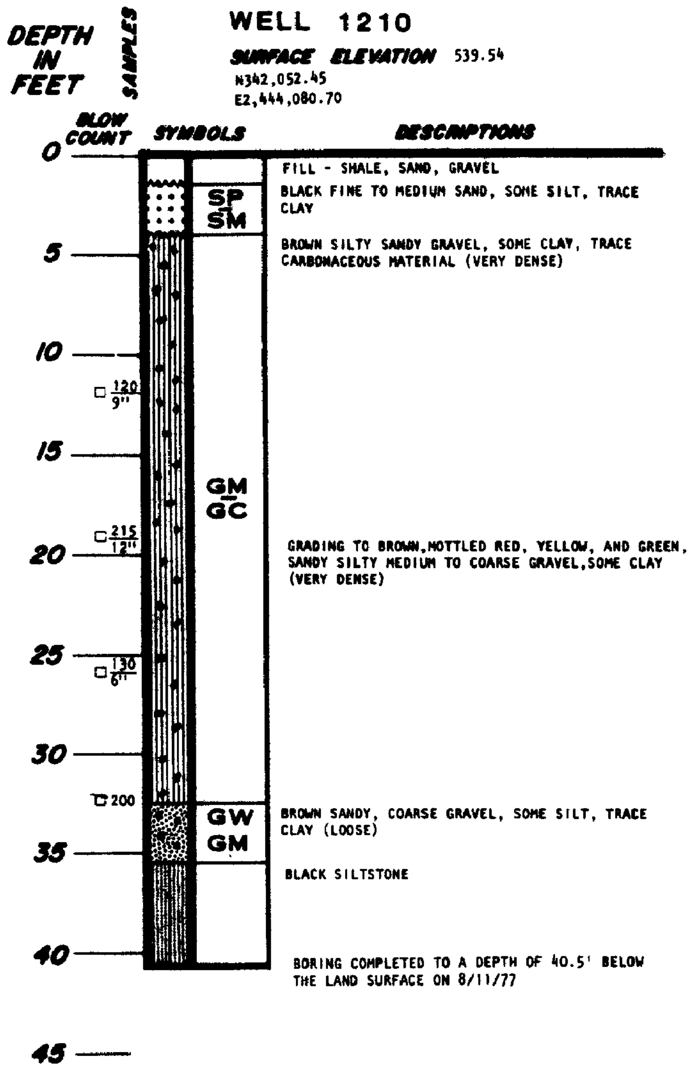
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

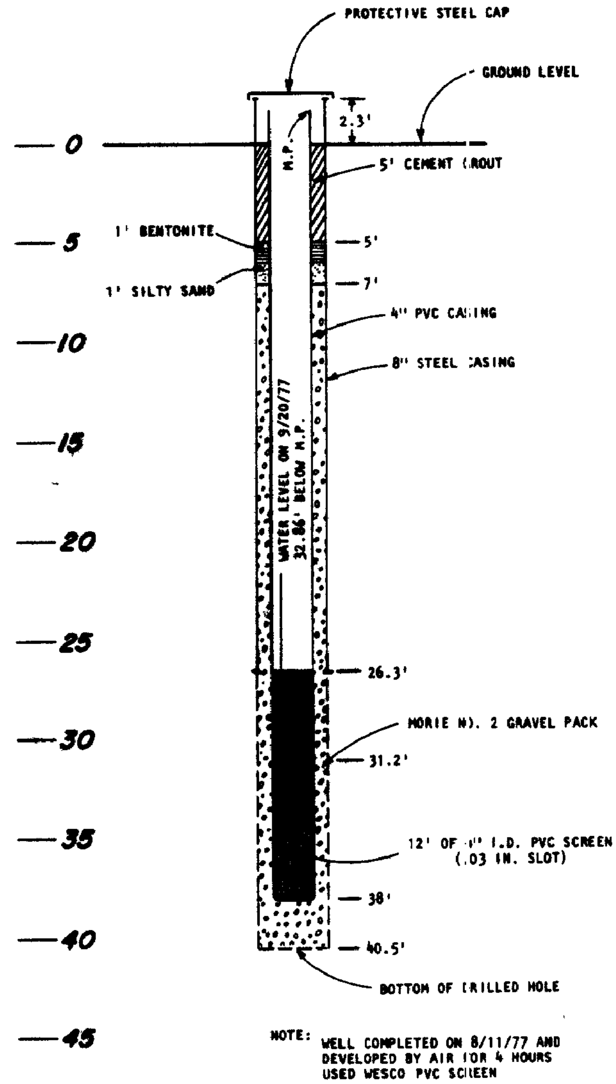
LOG OF BORING AND OBSERVATION WELL
CONSTRUCTION DETAILS - 1209A

FIGURE 2.4-48, Rev 55

AutoCAD: Figure Fsar 2_4_48.dwg



LOG OF WELL 1210



CONSTRUCTION DETAILS OF WELL 1210

HISTORICAL

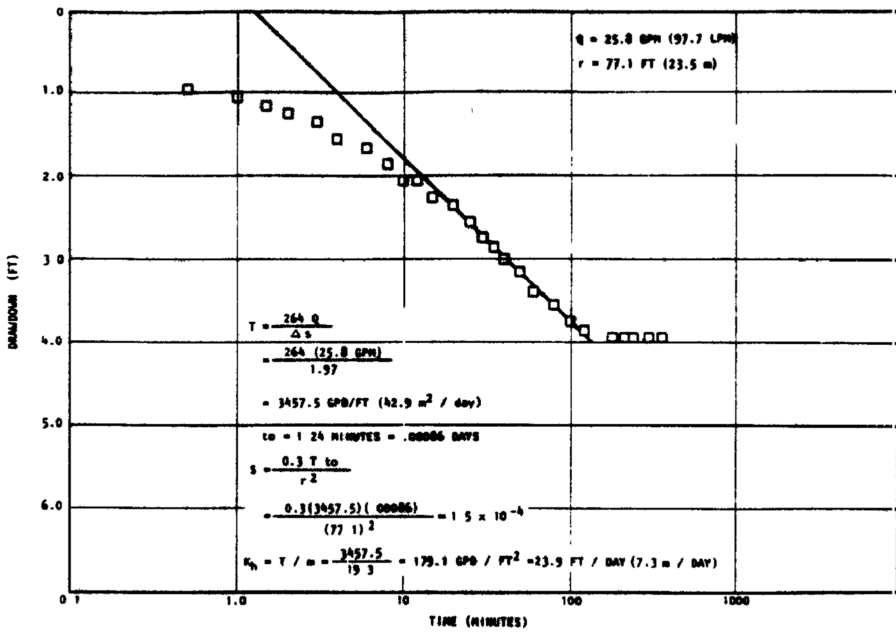
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
LOG OF BORING AND OBSERVATION WELL CONSTRUCTION DETAILS - 1210 FIGURE 2.4-49, Rev 55

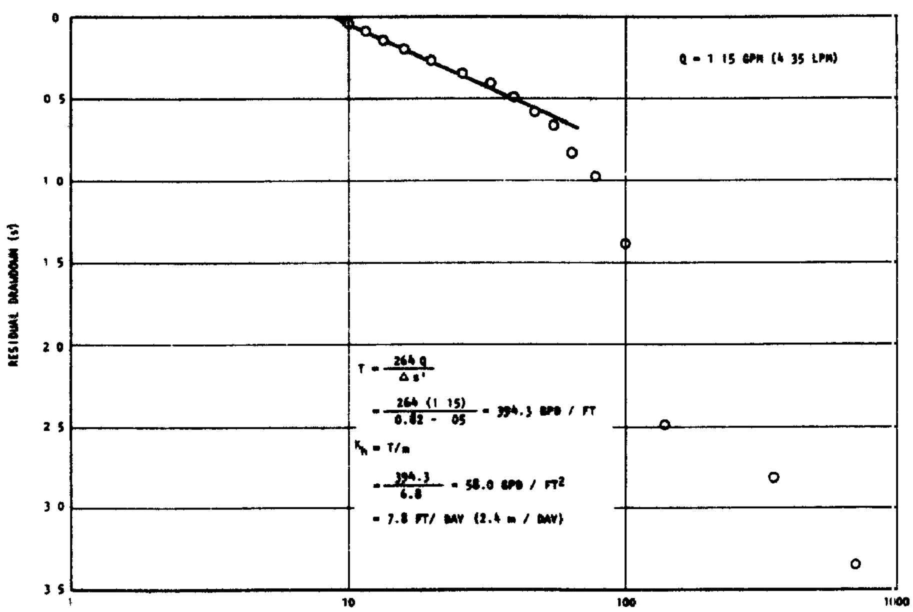
AutoCAD: Figure Fsar 2_4_49.dwg

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
MAP OF SUSQUEHANNA SES SHOWING GROUNDWATER CONTOURS IN JUNE 1971 FIGURE 2.4-50



TIME-DRAWDOWN CURVE AT WELL 11 DURING PUMPING OF WELL 1204 – SUSQUEHANNA SES



RESIDUAL DRAWDOWN CURVE AT WELL 1210 FOLLOWING 348 MINUTES OF PUMPING WELL 1210 SUSQUEHANNA SES

FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

HISTORICAL

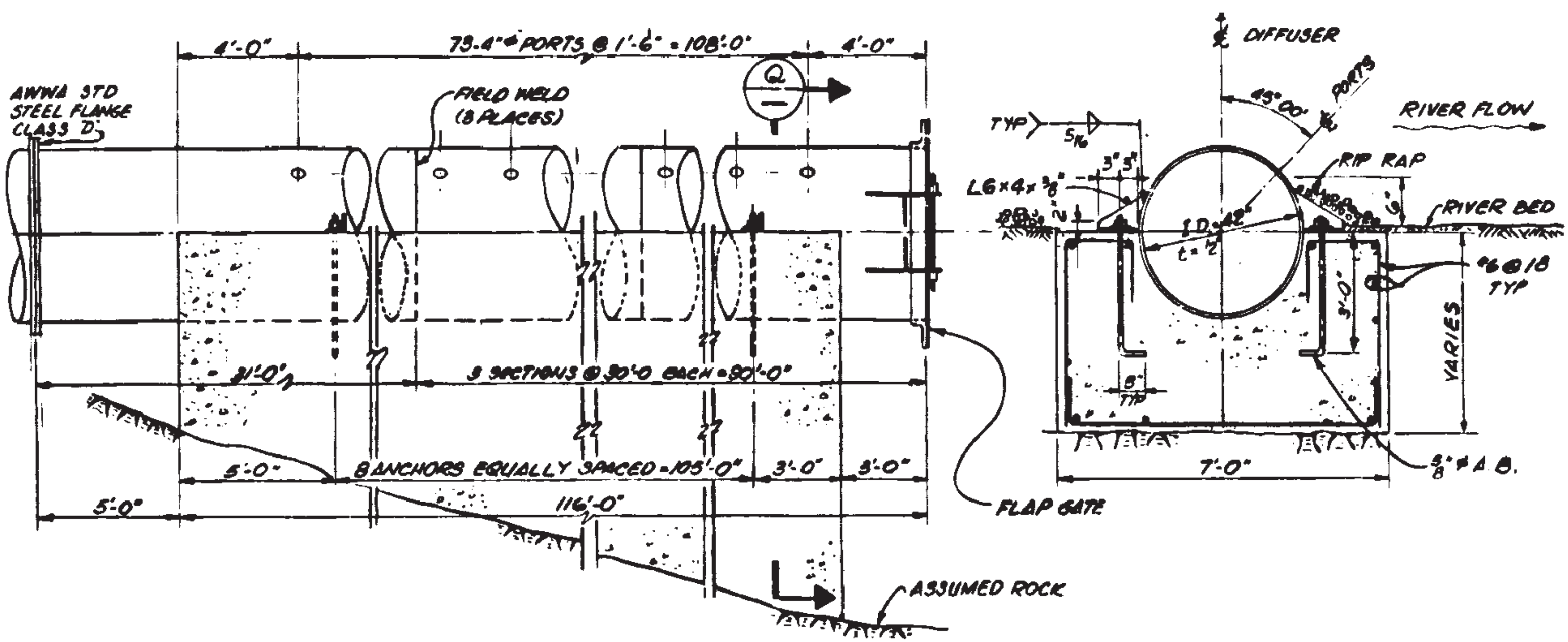
DRAINDOWN CURVES FROM PUMPING TESTS OF OBSERVATION WELLS 1204 AND 1210

FIGURE 2.4-51, Rev 55

AutoCAD: Figure Fsar 2_4_51.dwg

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
RIVER INTAKE STRUCTURE FIGURE 2.4-52



FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

RIVER DISCHARGE DIFFUSER

FIGURE 2.4-53, Rev 47

AutoCAD: Figure Fsar 2_4_53.dwg

THIS FIGURE HAS BEEN
REPLACED BY DWG.
A-12, Sh. 1

FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
Figure 2.4-54 replaced by dwg. A-12, Sh. 1 FIGURE 2.4-54, Rev. 57

AutoCAD Figure 2_4_54.doc

THIS FIGURE HAS BEEN
REPLACED BY DWG.
FF62005, Sh. 1

FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
Figure 2.4-55 replaced by dwg. FF62005, Sh. 1 FIGURE 2.4-55, Rev. 55

AutoCAD Figure 2_4_55.doc

2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING

2.5.1 BASIC GEOLOGIC AND SEISMIC INFORMATION

2.5.1.1 Regional Geology

2.5.1.1.1 Physiography and Geomorphology

The site is located (Figure 2.5-1) in the Valley and Ridge Physiographic Province which is bordered on the southeast by the Reading Prong and on the northwest by the Appalachian Plateau Physiographic Province (Figure 2.5-2).

The Valley and Ridge Province is characterized by folded Paleozoic sedimentary rocks of varying erosional resistance. These strata form a series of level ridges and intervening valleys which trend generally northeast to southwest. Higher ridges are formed on the more resistant, inclined sandstone whereas lower ridges are underlain by other competent formations. The valleys occur in less resistant limestone and shale.

The Valley and Ridge Province attains a maximum width of about 80 miles along a line drawn northwest through Harrisburg, Pennsylvania. In Pennsylvania, folds generally plunge away from this line to the northeast and to the southwest. Because of the folding, resistant strata form broad, zig-zag outcrop patterns across the province. These strata form steep slopes, that flank anticlinal valleys, and canoe-shaped synclinal valleys. Lithology and structure control the drainage pattern, the principal direction of which is to the southeast. Major drainage generally follows the strike of less competent strata and crosses the strike at water gaps where transverse structures, such as a high concentration of fractures, exist. Minor drainage trends normal to the regional strike or along major fracture sets, and usually intersects major streams at right angles to form a trellis pattern.

The Great Valley Section, in the southeastern third of the province, consists of broad, rolling valleys of low relief formed in Paleozoic soft limestones and calcareous shales. To the southeast, the Reading Prong exposes the oldest rocks (Precambrian) within 50 miles of the site (Figures 2.5-2 and 2.5-3). In general, the Reading Prong consists of high grade metasedimentary and metavolcanic rocks along with dominantly acidic plutonic rocks. These rocks experienced deformation commencing with the Grevillian orogeny about 1 billion years ago which imparted the dominant structural fabric, i.e., foliations, lineations and polyphase folds, present today. Succeeding tectonism during the Paleozoic and Mesozoic eras, and possibly even more recently, have also affected these rocks. According to Drake (Ref. 2.5-1) the rocks of the Reading Prong are allochthonous. The Triassic Lowlands of the Piedmont Province lie to the east and southeast of the Reading Prong, and contain the youngest rocks in eastern Pennsylvania (Figures 2.5-2 and 2.5-3). The rocks in the lowlands are dominantly red clastic sediments with associated basic intrusives and flows. Diabase dikes near Pottstown, Pennsylvania yielded K/Ar whole rock ages of 151 to 198 million years (Ref. 2.5-2, p. 3-25).

Northeast of the site, folds are broader and more open and give way to the gentle synclinal Pocono Plateau Section which is underlain by Devonian sandstone and shale. To the northwest, the Valley and Ridge Province terminates abruptly at the Allegheny Structural Front.

Beyond the front lies the Appalachian Plateaus (Figure 2.5-2), a gently rolling highland formed on broad folds of low structural relief that plunge gently to the southwest. The strata consist predominantly of an upper Paleozoic cyclic sequence of sandstone, shale, limestone and coal.

The Susquehanna River, which flows past the site, has two important features associated with it. First the river makes several sharp bends along its length with the closest being adjacent to the site. East of the site the river maintains a west-southwest course which parallels the regional tectonic fabric. However at Shickshinny, Pennsylvania, about 5 miles north of the site, it makes a sharp right-angle bend and flows in a south-southeast direction for about 5 miles. Just below the site it again swings sharply and resumes its west-southwest flow direction. This phenomenon has been cogently explained by Itter (Ref. 2.5-). He noted that this area was submerged during the Cretaceous and coastal plain sedimentation ensued. Sedimentation completely covered the pre-existing drainage pattern. Following coastal plain sedimentation, the area underwent broad uplift while the North Branch of the Susquehanna River apparently flowed southeastward, across the Pocono Plateau to Trenton, New Jersey (Ref. 2.5-3, p. 12-13). Tributaries must also have developed on this coastal plain. Following downcutting of the coastal plain sediments the streams encountered bedrock. Of these downcutting streams the present-day Susquehanna River, south of the confluence of the North and West Branches, apparently was able to incise more rapidly than other major streams. This resulted in stream capture and the pronounced bends seen along the river today.

The second feature of import is the buried valley of the Susquehanna. This buried valley occurs in bedrock overlain by a broad, flat plain across which the present-day Susquehanna flows (Ref. 2.5-3, p. 26). It extends upstream as a series of elongate basins, for about 15 miles, from near Nanticoke, Pennsylvania (approximately 10 miles northeast of the site) to just above West Pittston (Ref. 2.5-4, p. 8). This valley is filled with alternating layers of water laid gravel, sand and clay. The development of this valley is attributed to the erosive action of the Wisconsinan ice sheet which must have flowed diagonally across the valley (Ref. 2.5-3, p. 27 and 2.5-4, p. 7). Subsequently, this valley was filled with sediment deposited by streams which emanated from this melting ice.

Most of the region, north and east of the site, has been scoured by at least three periods of glaciation in the last 150,000 years (Ref. 2.5-5, p. 15 and 2.5-6). The three major directions of ice advance were postulated as follows: 1) south and southeast from central Ontario, 2) south and southwest from approximately the Adirondack region, and 3) south and southwest from the Hudson Valley by way of the Catskills (Ref. 2.5-5, p. 18). Effects of glacial scouring are most notable on the Pocono Plateau.

At the present time, there is no positive evidence that any pre-Illinoisan glaciation occurred in northeastern Pennsylvania although elsewhere in the eastern United States, such evidence does exist for pre-Illinoisan glaciation. It is thus suggested that it should also have occurred here (Ref. 2.5-6). The recorded glacial events include the Illinoisan and two stages of the Wisconsinan, the Altonian which spans the interval from about 70,000 years to about 28,000 years B.P., and the Woodfordian which lasted from about 21,000 years to approximately 13,000 years B.P.

In earlier literature (Ref. 2.5-5) the terms Altonian and Woodfordian were not utilized. Instead the Wisconsinan was divided into the Binghamton, Olean, Valley Heads and Mankato substages. However, it is not precisely known how the Altonian and Woodfordian subdivisions

relate to the older terms except that the Mankato is somewhat younger than the Woodfordian (Ref. 2.5-6).

In the site region a lobe of Illinoisan ice extended down the valley of the Susquehanna River from just above Berwick to the West Branch of the Susquehanna at Northumberland, Pennsylvania. The exposed length of deposits left by this lobe is about 40 miles whereas the maximum width does not exceed 8 miles. Illinoisan drift is present on the slopes to within 60 feet of the present river level suggesting that only moderate deepening of the Susquehanna Valley has occurred since deposition of the Illinoisan drift (Ref. 2.5-7, p. 24-25).

Valley trains of Wisconsinan gravel were examined along the Susquehanna River and its tributaries by Leverett (Ref.2.5-7). He noted that from just east of Berwick downstream (westward) to the West Branch at Northumberland, the surface of a Wisconsinan gravel train is well defined and generally occurs at about 40 to 60 feet above the river.

Moderately eroded terraces underlain by freshly appearing gravels mark the upper level attained by waters derived from the Wisconsinan ice sheet along the Susquehanna River and its tributaries. These terraces occur at lower elevations than those exposing Illinoisan gravel and have also been much less extensively eroded than Illinoisan gravel (Ref. 2.5-7, p. 16). In general the relative heights of the terrace levels representing each of the four Wisconsinan sub-stages are fairly constant. For example, in the site vicinity relative heights for the Mankato, Valley Heads, Binghamton and Olean are 9, 15, 30 and 45 feet respectively (Ref. 2.5-5, p. 77). The surfaces of most of the terraces have been eroded subjecting the observed height of any terrace to an error of as much as 30 percent. However, no evidence is presented indicating differential vertical offset of these terraces.

At Berwick several well developed kame terraces and terrace remnants of frontal kames which formed at the end of marginal kames occur (Ref. 2.5-, p. 91). The four lowest marginal kames were identified by Peltier as the First Olean, Second Olean, Third Olean, and Fourth Olean kame terraces which are respectively 86, 98, 110, and 158 feet above the river.

2.5.1.1.2 Stratigraphy and Lithology

2.5.1.1.2.1 The Appalachian Basin

The Valley and Ridge Province, in which the site is located, is part of a structural entity known as the Appalachian Basin. As defined by Colton (Ref. 2.5-8, p. 6-7), the Appalachian Basin is not a physiographic province. Rather, it is an elongate feature extending from the Canadian Shield in southern Quebec and Ontario, southwestward to central Alabama (Figure 2.5-4). It is bounded on the west by the Findlay Arch and on the south by the boundary between Paleozoic and Cretaceous strata. The eastern edge is marked by the surface contact between slightly-to-unmetamorphosed Paleozoic rocks on the west and more intensely metamorphosed Paleozoic and Precambrian rock on the east. In Pennsylvania this boundary coincides with the boundary between the Valley and Ridge and Piedmont Physiographic Provinces.

Isopach maps and stratigraphic columns show the respective thicknesses and relationships of the Cambrian through Pennsylvanian sequences in the Appalachian Basin (Figures 2.5-5 and 2.5-6).

In the Appalachian Basin, as outlined by Colton (Ref. 2.5-8), the Lower Cambrian clastic sequence is a wedge-shaped mass which is thickest along the eastern margin of the basin and thinnest along the northern and western margins. The rocks along the eastern margin are dominantly Early Cambrian whereas the rocks along the northern and western margins are mainly Late Cambrian. The Lower Cambrian sequence is conformably overlain in most of the Appalachian Basin by a suite of dominantly carbonate rocks with lesser amounts of quartz sandstone. This overlying suite consists mainly of rocks ranging in age from Middle and Late Cambrian to Early and Middle Ordovician and was designated by Colton (Ref. 2.5-8, p. 19) as the Cambrian-Ordovician carbonate sequence. This sequence ranges in thickness from about 600 ft. in northern New York State to a little more than 10,000 ft. in southeastern Tennessee. A belt of maximum thickness extends along, and approximately parallel to, the eastern edge of the Appalachian Basin from southeastern New York State to northern Alabama (Ref. 2.5-8, p. 23).

This carbonate sequence is conformably overlain in most of the basin by dominantly non-calcareous clastic rocks, the majority of which are Late Ordovician in age. These Upper Ordovician clastic rocks are thickest along the northeastern margin of the basin in Pennsylvania and show a generally uniform thinning to the north, west and southwest. An exception to this generally uniform pattern of thinning occurs in Pennsylvania and adjacent New York State where the sequence thins more abruptly against the southwestern extension of the Adirondack axis (Ref. 2.5-8, p. 23).

Although the boundary between these rocks and the older Cambrian-Ordovician sequence is conformable, the boundary with the overlying Silurian clastics is marked by an unconformity in the northeast and southwest portions of the Appalachian Basin. "Volumetrically the unconformity is greatest in eastern Pennsylvania and contiguous parts of New Jersey" (Ref. 2.5-8, p. 23). This unconformity was considered by Colton as evidence of Late Ordovician or Early Silurian diastrophism.

The Early Silurian rocks are mainly clastic and extend across most of the Appalachian Basin. These rocks are thickest (2,600 ft.) and coarsest in the northeastern part of the basin where they are composed mainly of sandstone and conglomerate.

Carbonate rocks, classified by Colton (Ref. 2.5-8, p. 31) as the Silurian-Devonian carbonate sequence, range in age from Middle Silurian to early Middle Devonian and occur throughout much of the basin. They, like the older sequences, are thickest in the east and thinnest in the west. The thickest section is found in northeastern Pennsylvania where it is about 3300 ft. thick. In northeastern Pennsylvania the lower half of this sequence actually consists of a thick wedge of red clastic rocks comprising, among others, the Bloomsburg Red Beds. West, northwest and southwest of this area the red beds grade into an alternating suite of variegated shale and siltstone, carbonates and evaporites. The margins of this suite are predominantly dolomite and limestone.

Rocks of Middle and Late Devonian age consist of a moderately thick sequence of shale, mudrock, siltstone and sandstone and extend throughout most of the basin. In most areas these Middle to Upper Devonian clastic rocks rest conformably on the strata of the Silurian-Devonian carbonate sequence. Like the underlying rocks this Devonian suite is wedge-shaped with the thickest part near the eastern margin of the basin and the thinnest part near the western periphery. The northeastern part of the basin, which includes east-central Pennsylvania, contains the thickest accumulation (more than 10,000 ft.) dominated by coarse-grained sedimentary rocks (including red beds). As the thickness decreases the average grain

size of the rocks shows a corresponding diminution, being medium-grained where the rocks are of intermediate thickness and fine-grained where the section is thinnest (Ref. 2.5-8, p. 34).

The Middle to Upper Devonian clastic suite is conformably overlain by Mississippian rocks in most of the basin, but the contact is slightly disconformable along much of the eastern margin. However, in parts of northeastern Pennsylvania the entire Mississippian is missing. This anomalous unconformity is probably due to erosion prior to Pennsylvanian sedimentation (Ref. 2.5-9, p. 35). Generally the Mississippian sequence defines a crudely wedge-shaped mass. The greatest accumulation occurs in southeastern Virginia (6800 ft.), but Wood (Ref. 2.5-10, p. C39) reported a thickness exceeding 6,000 ft. in eastern Pennsylvania.

Pennsylvanian rocks overlie those of Mississippian age with the basal boundary, in much of the basin, marked by a sudden change from older, thinly-bedded, relatively fine-grained rocks to younger, massively-bedded, conglomeratic quartz sandstone. The Pennsylvanian sequence is commonly thickest and coarsest-grained along the eastern periphery. In eastern Pennsylvania, where only the lower half of the Pennsylvanian is preserved, a thickness of 4600 ft. of principally sandstone, conglomeratic sandstone and conglomerate was recorded (Ref. 2.5-10).

2.5.1.1.2.2 The Valley and Ridge Province

The Valley and Ridge is a physiographic province which is situated within the Appalachian Basin and consists of a nearly continuous sequence of rocks extending from the Cambrian to the Pennsylvanian. Within this sequence are two major clastic intervals, the Cambrian-Silurian Taconic cycle and the Devonian-Pennsylvanian Appalachian cycle (Ref. 2.5-11, pg. 231). Each cycle consists of pre-orogenic carbonates and orthoquartzites overlain by turbidite flysch deposits which are, in turn, succeeded by molasse. The first phase of the older cycle is represented by Cambrian to Middle Ordovician carbonates (Ref. 2.5-12, p. 4). The flysch phase is represented by siltstone, silty-shale and gray sandstone of the Upper Ordovician Reedsville. The molasse phase comprises the Upper Ordovician Bald Eagle and Juniata Formations and the Lower Silurian Tuscarora Formation. The transition from the molasse phase to the renewal of marine conditions is delineated by the successively younger Rose Hill, Keefer, Mifflintown and Bloomsburg Formations of Middle Silurian age. Upper Silurian to Lower Devonian carbonates (Wills Creek to Onondaga) identify the first phase of the Appalachian cycle (Ref. 2.5-12, p. 4). Within this younger cycle direct passage from the carbonate phase to the turbidite phase was interrupted by deposition of a local sub-aqueous delta identified in central Pennsylvania as the Mahantango Formation. Following sedimentation of the Mahantango, the turbidite beds of the Upper Devonian Trimmers Rock Formation, constituting the second phase of the Appalachian cycle, were laid down. The molasse phase was initiated by the Upper Devonian Catskill Formation which, at an outcrop along the Lehigh River (Figure 2.5-7), is in gradational contact with the underlying Trimmers Rock (Ref. 2.5-13, p. 8). The molasse phase culminated twice, first in the Misissippian Pocono Formation and later in Pennsylvanian rocks (Ref. 2.5-12, p. 4).

2.5.1.1.2.3 Stratigraphic Units Within the Site Vicinity

Stratigraphic nomenclature used throughout this FSAR follows the recent usage of the Pennsylvania Geologic Survey who have not recently used the terms Susquehanna Group, Hamilton Group or Fort Littleton Formation in the site vicinity (See for example Ref. 2.5-12 and 2.5-17).

Middle Silurian to Pennsylvanian rocks within 10 miles of the site have been folded on the Berwick Anticlinorium. The units exposed across the fold are:

- The Middle Silurian Bloomsburg
- Upper Silurian Wills Creek
- Upper Silurian Tonoloway
- Middle Devonian Marcellus
- Middle Devonian Mahantango
- Upper Devonian Trimmers Rock
- Upper Devonian Catskill
- Upper Devonian-Lower Mississippian Pocono
- Middle Mississippian-Pennsylvanian Mauch Chunk
- Pennsylvanian Pottsville and "Post-Pottsville" (Llewellyn) Formations

In central Pennsylvania the Bloomsburg Formation was deposited in a brackish, shallow water, marine environment which is transitional between fluvial, continental sediment to the east and marine carbonates, shale and marl of the interfingering Wills Creek Formation to the west (Ref. 2.5-14, p. 119). It is a thick-to massive-bedded, dominantly grayish-red silty claystone with two sandstone intervals which occur both at the base and near the top (Ref. 2.5-14, p. 119). The sandstone intervals are medium-to-thin-bedded, poorly sorted hematitic sub-graywacke. The Bloomsburg is highly calcareous in the vicinity of Lewisburg, Pennsylvania, approximately 40 miles southwest of the site.

In central Pennsylvania the Bloomsburg is separated from the Marcellus Formation by about 1770 ft. of dominantly limestone and calcareous shale. These lithologies belong, in stratigraphically higher order, to the Wills Creek (Upper Silurian), Tonoloway (Upper Silurian), Keyser (Upper Silurian to Lower Devonian), Old Port (Lower Devonian) and Onondaga (Lower to Middle Devonian) formations (Ref. 2.5-12, Table 1).

The Wills Creek Formation gradationally overlies the Bloomsburg and consists of interlayered dark gray to greenish shale, red siltstone, light gray-green to olive siltstone and silty shale (all calcareous) and light gray dolomite to argillaceous dolomite. Medium gray limestone may be present.

The Tonoloway Formation gradationally overlies the Wills Creek and is composed of medium to dark gray, thinly laminated to thinly bedded limestone with some thin beds of medium gray calcareous shale. The Tonoloway is dolomitic at several locations.

The Upper Silurian to Middle Devonian Keyser, Old Port and Onondaga Formations were not mapped north of, nor east of Bloomsburg, Pennsylvania. The Keyser, therefore, does not appear to occur within ten miles of the site but was mapped further southwest (Subsection 2.5.1.1.3.3).

The lower Keyser is dominantly medium gray, fossiliferous, "pseudo-nodular" limestone which is cobbly when weathered. The upper Keyser contains laminated to thin bedded limestone similar to the underlying Tonoloway.

The Old Port and Onondaga Formations do not occur in the site vicinity but do crop out north of Bloomsburg (Subsection 2.5.1.1.3.3). The Old Port consists of dark gray, whitish weathering

chert, underlain by calcareous shale and thin gray limestone. The chert is locally overlain by gray to buff, medium to coarse grained fossiliferous sandstone.

The lower Onondaga is medium gray, highly fissile shale which is calcareous toward the top. The upper Onondaga is medium to dark gray, dense, fossiliferous argillaceous, locally carbonaceous, microcrystalline limestone.

The Marcellus, in central New York State, where it was defined, is about 350 ft. thick and consists predominantly of black shale with lesser amounts of black limestone (Ref. 2.5-15, p. 103). In east-central Pennsylvania, between Harrisburg and Williamsport, the Marcellus is a uniformly massive black, carbonaceous shale with several thin to thick bedded fine grained sandstone units (Ref. 2.5-16, p. 156). According to Faill (Ref. 2.5-17) the Marcellus refers exclusively to black shale overlying the Onondaga Formation.

The Mahantango Formation, which underlies most of the site, consists primarily of silty mudrock, shale, siltstone and sandstone with local occurrences of conglomerate, limestone and ironstone (Ref. 2.5-18, p. 13-14). In eastern Pennsylvania the Mahantango overlies the Marcellus shale and is, in turn, overlain by the Harrell Shale (Ref. 2.5-19, p. 18), a feature corroborated by Kaiser (Ref. 2.5-18, p. 6) who indicated that the Mahantango is defined by black shale, both at its base and its top.

Kaiser (Ref. 2.5-18, p. 18) informally divided the Mahantango into lower, middle and upper members. The basal member consists of an olive-gray shale with a basal sandstone and the middle member contains siltstone and shale, but where it is sandy it is identified as the Montebello. The upper member comprises an olive-colored shale, siltstone and sandstone with the sandstone locally highly ferruginous, finer grained, darker colored and more argillaceous than the underlying Montebello. Faill (Ref. 2.5-17, p. 23-24) divided the Mahantango into five members which are, in stratigraphically higher order, the Turkey Ridge, Dalmatia, Fisher Ridge, Montebello and Sherman Creek. According to Wells and Faill (Ref. 2.5-12, Table 1) the Turkey Ridge is a light to olive-gray, fine to coarse-grained sandstone and the Fisher Ridge is predominantly a laminated olive gray to medium-gray silty shale. The Montebello is an olive-gray, medium-light gray to dusky yellow, fine to medium-grained, locally conglomeratic, fossiliferous sandstone with interbedded siliceous siltstone and silty claystone which display cycles of reverse graded bedding. The Sherman Creek (Ref. 2.5-17) or Sherman Ridge (Ref. 2.5-12) comprises olive-gray, fossiliferous, silty claystone with two interbedded siltstone and fine sandstone units which coarsen upward.

Faill (Ref. 2.5-17, p. 23-24) noted that the Middle Devonian rocks are cyclic with each cycle marked by black to dark gray, silty claystone at the base and displaying an upward increase in grain size to conglomeratic sandstone. Immediately overlying the coarsest rock units there is a marked decrease in grain size with claystone or siltstone marking the base of the overlying cycle.

The cyclic nature of the Middle Devonian strata is reflected within the Mahantango. These internal cycles are asymmetric and are smaller scale reflections of the cyclicity recorded throughout the entire Middle Devonian. That is, they commence with black to dark or olive-gray silty claystone which grades upward into argillaceous siltstone, silty sandstone and fine to medium-grained, locally conglomeratic, silty sandstone. The cycles within the Mahantango are repetitive and range, in thickness, from approximately 7 to 250 ft. The thicker cycles can usually be traced over distances of from 5 to 35 miles. (Ref. 2.5-20, p. 113).

The average thickness of the Mahantango is about 1650 ft. with respective maximum and minimum thicknesses of about 2900 ft. at McCullochs Mills, Pennsylvania and 840 ft. at Riverside which is northeast of McCullochs Mills and about 20-22 miles west of the site (Figure 2.5-7). Overall the Mahantango shows a general thinning to the north, a feature reflected in the Montebello sandstone member which is thickest just west-northwest of Harrisburg and thins to the west, north and east. North of the 41st parallel, which lies just south of the site, the Montebello has totally disappeared (Ref. 2.5-18, p. 13-14).

In the Anthracite region of Pennsylvania the Mahantango and the Marcellus were combined to form a lithotectonic unit. As defined by Wood and Bergin (Ref. 2.5-21, p. 151) this lithotectonic unit actually includes the Marcellus, Harrell and Brallier Shales and the Tully Limestone. However, the Brallier overlies the Mahantango at about the Tully horizon (Ref. 2.5-19, p. 18). The Tully, in this area, has been incorporated into the Mahantango. Thus, in this report, the lithotectonic unit of Wood and Bergin is considered as containing the Marcellus, Mahantango,Tully and Harrell. In the southwestern and western parts of the Anthracite region this unit is 1100 ± ft. thick whereas in the central and eastern parts it is about 3000 ± ft. thick; the average thickness is 2000 ± ft. (Ref. 2.5-21, p. 148). However, a greatly thickened section of this unit occurs in the P. Good No. 1 Well (Figure 2.5-7) on the crest of the Berwick Anticlinorium, east of the site. This excess thickness is believed to be due to faulting and disharmonic folding (Ref. 2.5-21, p. 148).

At the site the Mahantango consists of a lower, gray, calcareous siltstone (120-150 ft. thick) overlain by a dark gray, locally fossiliferous siltstone which is intermittently calcareous. These two members are lithologically similar to and occur within the same stratigraphic interval as the Harrell Shale and the underlying Tully Limestone; thus, the latter two units were incorporated into the Mahantango.

In the site vicinity, the Mahantango is represented only by the uppermost member, the Sherman Creek which is dominantly a dark gray to blue gray, olive gray to brown weathering mudrock. Siltstone and fine grained sandstone units crop out locally. Both calcareous and non-calcareous strata occur at several localities.

In the site vicinity an interval of light, medium-gray argillaceous limestone, near or at the top of the Mahantango, was recognized as a Tully Limestone equivalent and was included within the Mahantango. Calcarous silty mudrocks which may be Tully equivalents also occur (Subsection 2.5.1.2.2). Faill (Ref. 2.5-17, p. 24) also included the Tully as part of the Mahantango because of its lithologic similarity to the Sherman Creek Member. The overlying Harrell Formation, a poorly exposed, dark silty shale which appears to be in gradational contact with the Mahantango was incorporated into this map unit (Subsection 2.5.1.2.2).

Fossils are relatively abundant within the Sherman Creek member of the Mahantango Formation and include various genera of brachiopods, bryozoa, pelecypods, coral, trilobites and crinoid fragments. Fossil casts are abundant with occasional molds and rare preservation of internal structure and original shell material.

Concretions (commonly rusty weathering), spheroidal weathering and prominent closely spaced steeply dipping cleavage, which may quite easily be mistaken for primary bedding fissility, are other features characteristic of the Mahantango. Due to its predominantly argillaceous nature and cleavage, the Mahantango is fairly easily eroded and is thus topographically expressed as a relatively low area.

In the general area marked by the confluence of the Susquehanna and Juniata Rivers, the Trimmers Rock Formation comprises an interlayered assemblage (about 2000 ft. thick) of thin to medium-bedded, medium gray siltstone and medium gray, slightly silty and somewhat fissile shale. Thin layers of fine-grained sandstone occur in the upper part (Ref. 2.5-12, Table 1). Graded bedding, along with groove and flute casts, occur in some of the siltstone beds indicating deposition by turbidity currents. Load casts or ball and pillow structures are also present in some siltstone layers. These lithologies and sedimentary structures are also present in the Trimmers Rock at an outcrop along the Lehigh River about 16 miles southeast of the site (Ref. 2.5-13, p. 8) (Figure 2.5-7). At this exposure both bedding thickness and grain size increase upward in this formation which is about 1165 ft. thick.

In the site area the Mahantango and Harrell grade upward into the Trimmers Rock (Subsection 2.5.1.2.2). The Trimmers Rock is dominantly interbedded, medium to olive gray, thinly laminated siltstone, silty shale and fine grained, laminated to massive sandstone. These rocks weather to brownish gray color. Sedimentary structures include fining-upward sequences, groove casts, current lineations, load casts, ball and pillow and flow roll structures. Ripple marks were also locally identified. These structures indicate deposition by turbidity currents in a marine environment. Fossils are often restricted to relatively thin layers of brachiopods. Other fossils include pelecypods and crinoid fragments.

The upper Trimmers Rock Formation consists of light to medium grayish green silty shale and micaceous, dark greenish gray siltstone (both of which weather to a dark reddish brown color), a reddish brown silty fine to medium grained sandstone to siltstone and an olive green vitreous, fine grained sandstone to siltstone. The uppermost units of the Trimmers Rock Formation grade upward into the basal Catskill Formation. This gradation between the Trimmers Rock and the Catskill has apparently caused problems concerning the placement of the contact between them. However, Faill and Wells (Ref. 2.5-22) have placed this contact at the base of a thick sandstone unit which is the lowest occurrence of upward fining cycles, a feature characteristic of the Catskill. This unit was selected by Faill and Wells because it is easily mappable. Glaeser (Ref. 2.5-13, p. 4) evidently concurred with Faill and Wells for he considered the base of the Catskill to lie at the first occurrence of distinctive sandstone units which are found above or near the top of the turbidite suite which constitutes the bulk of the Trimmers Rock Formation.

The Catskill Formation at the Lehigh River outcrop is about 7675 ft. thick and consists mainly of siltstone and sandstone with some conglomerate and shale. Upward fining cycles were recognized at various intervals throughout the entire formation both at Lehigh River (Ref. 2.5-13, Figure 2) and near Halifax, Pennsylvania (Figure 2.5-7), (Ref. 2.5-22, p. 107).

Complete or nearly complete sections of Upper Devonian rocks are preserved, both in outcrop and in the subsurface, at the Lehigh River outcrop, the Richards Well and the Hudson Realty Well (Figure 2.5-7). Based on these occurrences Glaeser (Ref. 2.5-13, p. 35) estimated the original thickness of the Upper Devonian section at various locations in northeastern Pennsylvania. He then compared these estimates to the amount of section preserved today and ultimately estimated the amount of section lost. The amount of missing section ranges from 0 ft. to about 6125 ft. (Ref. 2.5-13, p. 38) a variation due mainly to the location of the sections with respect to structure. For example, in the P. Good Well, which occurs on the nose of the Berwick Anticlinorium, about 8 miles east of the site (Figure 2.5-7), about 5203 ft. are missing. Glaeser (Ref. 2.5-13, p. 36) assumed that these sections were lost due to erosion.

In the site area, the contact between the Catskill and the underlying Trimmers Rock was drawn at the base of the first relatively thick reddish brown to maroon sandstone or brownish red siltstone and more massive reddish brown (maroon) micaceous fine grained sandstone. This mappable contact appears to occur at or near the lowest fining upward sequences. The lower Catskill Formation contains sedimentary structures such as intraformational clasts of green shale within light gray fine sandstone, oscillation ripple marks and roots which are indicative of the marine to non-marine transition zone.

The Pocono Formation which overlies the Catskill, consists typically of medium and coarse grained light gray to white, rusty weathering quartz sandstone with thin layers of quartz pebble conglomerate. Olive gray, fine grained sandstone, reddish gray medium to fine grained sandstone and siltstone and greenish gray medium grained, cross bedded sandstone also occur within this formation. Cross bedding is common.

Grayish red sandstone layers occur near the base of the Pocono. These were recognized along the east side of the Susquehanna River South of Mocanaqua and along the road between Aldean and Folstown. North of the site, the Pocono consists of an interlayered sequence of predominantly medium gray, thick, well laminated, gray weathering quartz sandstone and subordinate, red, flaggy quartz sandstone. Near Folstown, well laminated red sandstone is interlayered with, but decidedly subordinate to, well laminated, rusty weathering, light gray, coarse grained sandstone and fine grained gray sandstone. Coarse to medium grained, mainly grayish to greenish gray sandstone featuring rather subtle cross bedding dominate the upper portion of the exposure. These strata, along with an underlying thin zone of light greenish gray sandstone, in turn, underlain by green shale and mudrock, has been selected as marking the basal Pocono. Beneath all of these units, is a red, well laminated, argillaceous siltstone which is interpreted as marking the top of the Catskill. Red shale, which marks the base of the outcrop underlies cross bedded medium, light gray, olive gray weathering quartz sandstone. The lower (topographically and stratigraphically) portion of the outcrop is dominated by red lithologies in contrast to the upper part in which no red lithologies were exposed. Besides the obvious color change the sandstone above the inferred contact is coarser grained and more subtly cross bedded than sandstone which occurs between the red units near and at the base of the outcrop. Thus, contrary to other interpretations the Catskill-Pocono contact appears to be gradational in the site area rather than unconformable.

The upper Pocono Formation in the vicinity of Shickshinny consists of medium to light gray conglomeratic sandstone with rounded to sub-rounded quartz pebbles and shale fragments and rusty weathering, fine to medium grayish green, micaceous siliceous sandstone and finely laminated greenish gray, rusty weathering, siliceous quartz sandstone. Rusty weathering, medium light gray, medium to coarse grained quartz sandstone is interbedded with thin layers of dark gray silt. Shale and medium gray quartz lithic sandstone fills channels.

The overlying Mauch Chunk Formation is generally bright red in color and consists of mudrock, silty shale, siltstone and fine to medium grained cross bedded, well laminated sandstone. In the southern part of the Anthracite region, this sequence of red beds is 2,400 ± feet thick and is overlain by a sequence of alternating red sandstone and shale beds and gray conglomerate and sandstone beds 300 to 600 feet thick. This upper sequence represents a transition zone in which red beds typical of the underlying Mauch Chunk are interbedded with gray beds typical of the overlying Pottsville Formation (Ref. 2.5-10). Detailed stratigraphic studies indicate that the beds of the transition zone (upper Mauch Chunk) intertongue with and laterally replace the

lower beds of the Pottsville Formation from south to north. The upper Mauch Chunk is, therefore, late Mississippian and Early Pennsylvanian in age (Ref. 2.5-10).

Both lower and upper members of the Mauch Chunk Formation are exposed in the site area. The lower member is exposed immediately above the conformable contact with the underlying Pocono Formation at several locations. The upper part of the formation along the south limb of the Lackawanna Synclinorium is marked by interlayered red and olive gray sandstone, siltstone, and silty shale. Locally the siltstone contains layers of rounded, circular to elliptical calcite filled voids. Elsewhere the Mauch Chunk contains greenish gray to grayish green medium to coarse grained, locally micaceous sandstone, thinly laminated gray, fine grained sandstone and siltstone and massive medium grained sandstone.

The Pottsville and Llewellyn formations represent the coal bearing zones of the Anthracite Region and have, for the purpose of this report, been combined and treated as a single formation. The Pottsville is composed of coarse pebble conglomerate, quartzose sandstone, subgraywacke, siltstone, shale and anthracite. This formation ranges in thickness from about 1,400 feet in the southern Anthracite field to about 600 feet in the Western Middle Anthracite field (Ref. 2.5-10).

Strata overlying the Early to Middle Pennsylvanian Pottsville have been informally termed "Post-Pottsville" rocks (Ref. 2.5-24). "Post-Pottsville rocks" in the Southern and Western Middle Anthracite fields were named the Llewellyn Formation by Wood (Ref. 2.5-10). This name is informally used for the grayish and brownish conglomeratic sandstones, quartz sandstones, subgraywackes, and siltstones overlying, but not subdivided from, the Pottsville Formation in the site area. Usage of the name Llewellyn for "Post-Pottsville" rocks in the Northern Anthracite field is consistent with Bergin (Ref. 2.5-23).

Within five miles of the site, the Pottsville and Llewellyn, collectively consist of quartz pebble conglomerate in a quartz sandstone matrix, quartz pebble conglomerate in a carbonaceous quartz sandstone matrix, coarse grained dark to medium gray, massive and flaggy, carbonaceous sandstone and shale, dark gray to black siltstone and coal. The non-carbonaceous quartz pebble conglomerate displays cross beds.

2.5.1.1.3 Regional Tectonics

2.5.1.1.3.1 Tectonic Provinces

The Appalachian orogen in the northeastern United States was divided into two parts, the mobile belt and the craton (Ref. 2.5-25 and Subsection 2.5.2.2). The mobile belt in this area lies along the east coast with its western edge parallel to, and west of, the eastern limit of North America (Figures 2.5-8A and 2.5-8B). In general the mobile belt is underlain partly by Precambrian crustal rocks and partly by presumably mafic crust. However, in the Maritime Provinces of Canada as well as in southeastern Massachusetts it is underlain by a volcanic-sedimentary sequence which formed less than 600 million years ago. These three grossly-grouped lithologies, i.e. Precambrian crustal rocks, mafic crust and volcanic-sedimentary rocks of the Avalon Platform provided the basis for dividing the mobile belt into the 1) eastern cratonic margin, 2) the Central New England tectonic province and 3) the Avalon Platform tectonic province respectively (Ref. 2.5-25). In the eastern cratonic margin the Precambrian basement is overlain by 1) Late Precambrian clastic rocks and associated mafic

dikes and volcanics, 2) a miogeosynclinal assemblage and 3) a eugeosynclinal assemblage (Ref. 2.5-25). The eastern cratonic margin is marked by a zone of faulting, contrasting structural styles and contrasting metamorphic facies. The Central New England province is bounded on the east by the Avalon platform and on the west by the Inner Piedmont. It features a thick, dense, presumably mafic crust overlain by eugeosynclinal sediments. It is also marked by intense deformation and Lower Paleozoic metamorphism. The Avalon Platform province is characterized by crystalline, continental crust (Late Precambrian) intruded by Ordovician to Devonian age plutons. In juxtaposition with, and to the west of the mobile belt, lies the craton which is underlain by Precambrian crystalline rocks that were deformed during the Grenvillian orogeny, about 1 billion years ago. Based on gross geologic structure the craton was divided into an eastern belt and a western basin. The eastern belt is coincidental with the Highlands Tectonic Province (Figures 2.5-8A and 2.5-8B) which is characterized by Grenvillian (Precambrian) rocks deformed during Paleozoic crustal convergence. The western portion, which is subdivided into the Fold and Thrust Belt and the stable interior is characterized by the absence of basement involvement during Paleozoic crustal convergence (Ref. 2.5-25). The Fold and Thrust Belt, in which the site is located, is in contact with the Highlands Province of the eastern craton and exposes tightly folded and faulted Paleozoic sedimentary rocks. To the west of, and in sharp contact with, the Fold and Thrust Belt lies the Stable Interior which is underlain by very gently folded, shelf delta type deposits. The Fold and Thrust Belt and the Stable Interior coincide approximately with the Valley and Ridge (including the Great Valley) and Appalachian plateaus Physiographic provinces respectively.

2.5.1.1.3.2 Structural Elements within the Craton

The site is situated upon the Scranton gravity high (Figure 2.5-9) which extends southwestward from Albany, New York to Harrisburg, Pennsylvania where it abruptly terminates (Ref. 2.5-26, p. 198; Ref. 2.5-27, p. 711). To the west of this termination, regional gravity patterns suggest a northwest trending Precambrian fault with left lateral displacement of several tens of miles (Ref. 2.5-27, p. 711).

The high, itself, is located in both the Fold and Thrust Belt and the Stable Interior. The maximum Bouguer anomaly values associated with this feature occur in northeastern Pennsylvania and adjacent New York State where the overlying sedimentary section is at least 39,370 ft. thick (Ref. 2.5-28, p. 52 and 2.5-26, p. 201). Of all the models (which include tensionally induced rifting) proposed to explain this feature (see Ref. 2.5-26, p. 203-209) the favored one involves warping of the mantle with the anomaly due to an extensively broad mass occurring deep within or at the base of the crust. This structure is apparently related to the tectonic evolution of the Appalachian system (Ref. 2.5-26, p. 213 and 218-219).

Principally the Fold and Thrust Belt contains deformational features indicative of regional crustal compression (Figure 2.5-8A and 2.5-8B). In the area northeast of Roanoke, Virginia the structural style, at the surface, is dominated by folding with faulting subordinate, whereas southwest of Roanoke reverse faults predominate over folding at the surface (Ref. 2.5-29, p. 125). Another feature present in this belt, and indeed in the entire Appalachian Orogen, is an arcuate configuration which is especially well expressed in central Pennsylvania.

The largest folds in the Fold and Thrust Belt exceed 125 miles in length but folds of microscopic to hand specimen scale have also been recognized. The largest folds, with wave lengths ranging from 6 to 11 miles, were classified by Nickelsen (Ref. 2.5-30, p. 16) as first order folds, whereas the hand specimen and microscopic size folds were classified as fifth order folds.

Second through fourth order folds are intermediate in size. The largest folds are not restricted to the Fold and Thrust Belt for they also occur in the adjacent Stable Interior (Ref. 2.5-30).

Generally these folds do not display an ideally parallel form, rather their hinges are usually narrow relative to their wave lengths. They are somewhat akin to similar folds yet they lack the characteristic features of similar folds which include attenuated limbs, with correspondingly thickened axial regions, and a sinusoidal form (Ref. 2.5-31, p. 10). Thus, according to Faill, the fold geometry is neither parallel, similar nor intermediate for it shows features that are not associated with either geometric type, i.e., the bedding in the limbs is planar and the hinges are narrow (Ref. 2.5-30, p. 19).

Although the folds lack the characteristic geometry of parallel folds, they are flexural slip in nature for they display wedge faults, uniform bed normal thickness across the fold and slickensides on bedding surfaces (Ref. 2.5-32, p. 1289 and 2.5-31, p. 11). In addition to these flexural slip folds, kink bands, a few inches to hundreds of feet wide, are visible in outcrop. Kinematically and geometrically the kink bands and the flexural slip folds are congruent and, therefore, related (Ref. 2.5-32, p. 1289).

Kink bands are usually considered to be small scale structures, however they occur on a much larger scale in the Fold and Thrust Belt with smaller kink bands and folds present in the limbs of the larger-scale structures. Faill attributed the existence of large scale kink bands to the wide spacing between bedding surfaces.

From southeast to northwest across the Fold and Thrust Belt, and westward into the Stable Interior, the folds become progressively less tightly appressed. This gradual change from tight to more open folds is illustrated by changes in the inter-limb angle which is about 50°-70° on the east side of the Great Valley and approximately 80° on the west side. In the central Valley and Ridge (Fold and Thrust Belt) the limbs subtend an angle of about 100° and in the Appalachian Plateaus (Stable Interior) this angle is nearly 180° (Ref. 2.5-32, p. 348). This change is also expressed by differences in structural relief which diminishes from possibly 35,000 ft. on the South Mountain Anticlinorium on the southeast (Ref. 2.5-33, p. 348) to 7,000-9,000 ft. in the central Valley and Ridge to about 4,500 ft. in the western Valley and Ridge. In the eastern Plateaus area 2,500 to 3,000 feet of structural relief occur. Across the Plateaus area this relief continues its progressive decrease with the most westerly folds showing less than 300 ft. (Ref. 2.5-33, p. 349). In fact in the Plateaus region the folds are so broad and gentle that structural contour maps are required in order to analyze them. Between the Valley and Ridge (Fold and Thrust Belt) and the Appalachian Plateaus (Stable Interior) there is an abrupt decrease in structural relief. This area has been termed the Appalachian Structural Front (Ref. 2.5-34).

LANDSAT images of an area along the west branch of the Susquehanna River at Lewisburg, Pennsylvania suggested the presence of a nearly northwest trending cross or tear fault. This structure shows about one mile of left lateral separation of a prominent ridge underlain by the Tuscarora Formation. However, geological reconnaissance mapping confirms that this left lateral topographic offset is caused by a kink fold as shown on the Geologic Map of Pennsylvania (Ref. 2.5-24).

Faults, like the folds, occur on all scales within the Fold and Thrust Belt and show displacements ranging from inches to hundreds of feet. The largest faults range in length from about 7 miles to 200 miles (Ref. 2.5-33, p. 349).

According to Faill and Nickelsen (Ref. 2.5-31, 20) most faults seen at the surface can be classified as wedge faults or cross faults. Root (Ref. 2.5-33, p. 349) stated that all major faults in the Fold and Thrust Belt and the Stable Interior are moderate to steep thrusts with dips ranging from 40°-70° to the southeast. However, in the southern Great Valley (affecting part of the Fold and Thrust Belt), he also identified steeply dipping, west facing thrust faults and tear or cross faults (Ref. 2.5-34). Wood and Bergin (Ref. 2.5-21) noted that in the southeastern part of the Anthracite region (of the Valley and Ridge Province) there are hundreds of reverse, tear and bedding faults, whereas in the northern part faults are far more scarce with only reverse faults, showing minor displacements, having been recognized. Glass (Ref. 2.5-36, p. 9) identified at least eighty major, essentially vertical faults in the Appalachian Plateaus Province. Generally these faults are normal to the regional tectonic grain although variations between NO5W and N89W were observed. Wedge, splay and reverse (thrust) faults are categorized together because they all result in crustal shortening and duplication of strata. Faill (Ref. 2.5-32, p. 1298) indicated that splays off decollements and wedge faults are identical. However, Root (Ref. 2.5-35) distinguished between thrusts dipping steeply to the west and those dipping steeply to the east with the former equated to wedge faults and the latter to splay faults off decollements. The Hunting Valley-Cream Valley Faults and the Sweet Arrow Thrust (Figure 2.5-7) appear to fit into this category.

Wedge faults intersect bedding at a low angle (10°-30°) and terminate in bedding planes (Ref 2.5-32, p. 1295), although a number of them terminate in folds (Ref. 2.5-32, p. 1298). Commonly they occur as isolated structures on the limbs of folds but they have also been observed in fold hinges. In outcrop wedge faults generally occur in interlayered sequences displaying contrasting mechanical properties and only cut across beds of sandstone or siltstone that are surrounded by shale (Ref. 2.5-32, p. 1297 and 2.5-31, p. 23). This same relationship also holds on a much larger scale, since most of the large, mappable faults occur in interlayered sequences of contrasting lithologies.

Root (Ref. 2.5-35, p. 105-106) identified faults in the southern Great Valley which presently dip steeply to both the east and west. The steeply inclined, east dipping reverse faults generally parallel the trace of anticlinal hinges and cut the vertical to overturned, west facing anticlinal limbs. These faults developed as steeply dipping schuppen structures which splayed off subhorizontal reverse faults (decollements) (Ref. 2.5-37, Figure 5, 2.5-33, P. 350 and 2.5-35, Figure 5). Reverse faults, which dip steeply to the west, are also present in the west-facing subvertical to overturned limbs and parallel the structural fabric. However, some have been rotated during folding and now display the geometry of east-dipping normal faults.

In the Anthracite region, faults (akin to the steeply east-dipping splays described by Root) are inferred to occur in the cores of anticlines (Ref. 2.5-21). Wood and Bergin interpreted that many of these faults were folded along with the rock units, however folded splay faults have not been depicted in other publications (e.g. Ref. 2.5-37 and 2.5-33).

Cross (transverse) faults are commonly vertical to nearly vertical structures which are approximately perpendicular to the regional tectonic grain. They are less common than wedge faults although they have been mapped in the southeastern part of the Anthracite region, in the Great Valley and in the Appalachian Plateaus region (Ref. 2.5-21, p. 149, 2.5-35 and 2.5-36). They are also evident in the Cambro-Ordovician rocks of the Conestoga Valley near York and Lancaster, Pennsylvania (Ref. 2.5-24). Commonly cross faults display strike-separation and, in fact, have been described in the Appalachian Plateaus Province as wrench faults (Ref. 2.5-36). According to the verbal description provided by Glass (Ref. 2.5-36, p. 6) at least some of these

faults could be identified as paired conjugate wrench faults, for those that strike nearly north show left lateral separation whereas those which strike in a more westerly direction display right lateral separation. Despite his verbal description the map pattern shows a general pattern of anastomosing fault traces which are more or less subparallel to each other and normal to the trend of the Allegheny Structural Front (Ref. 2.5-36, p. 8). It is conceivable that both wrench faults and cross faults occur in the Appalachian Plateau but attempting to distinguish them is of little import for both are predictable cogenetic products of regional horizontal compression. Two cross faults in the southern Great Valley have lateral movements associated with them (Ref 2.5-35, p. 104), and the map pattern in the Conestoga Valley shows lateral separations of lithologic units along the cross faults there (Ref. 2.5-24). Another feature common to most cross faults is that reverse or wedge faults frequently terminate against them. According to Root (Ref. 2.5-35, p. 103), however, their most distinctive feature is that the rocks on either side of these faults have experienced different amounts of horizontal shortening.

2.5.1.1.3.3 Structural Elements in the Site Vicinity

The site rests on the easterly plunging nose of the Berwick Anticlinoirum. Across this folded structure, at depths of 17,500-25,000 ft $\pm$ 3,500 ft, the strata (based on seismic reflections) are nearly horizontal to slightly north dipping (Ref. 2.5-38, p. 134). According to Wood and Bergin (Ref. 2.5-21) either a major decollement or a series of decollements probably exists within the Marcellus Shale throughout most of the Anthracite region. Although not seen at the surface the presence of decollements is inferred based upon the existence of disturbed outcrops "differing styles, wavelengths and amplitudes of folds above and below the Marcellus, and by wells that penetrated either duplicated sections or greatly thickened sections" (Ref. 2.5-21, p. 151).

It is worthy to note, however, that neither this fault nor any other fault appears as a surface feature associated with the Berwick Anticlinorium in either Wood and Bergin's paper (Ref. 2.5-21, Figure 2) or Gwinn's paper (Ref. 2.5-38, Figure 1). The 1960 edition of the Pennsylvania Geologic Map (Ref. 2.5-24) does indicate a fault along part of the north limb of the Berwick Anticlinorium just north of Bloomsburg. This fault approximately 8 miles long, separates the undifferentiated Wills Creek, Tonoloway and Keyser Formations from undifferentiated Onondaga, Marcellus and Mahantango Formations (Ref. 2.5-24, map unit Skw from map unit Dho, respectively). According to the state map, the missing interval between these two sets of units includes the Mandata and Oriskany Formations. Recent mapping indicates that the Tonoloway Formation is juxtaposed to the Marcellus Formation (Figure 2.5-10, Stations DJ-4B, -33 and -34). The fault on the geologic map of Pennsylvania was probably postulated to explain the missing stratigraphic interval on the north limb of the Berwick anticlinorium, which includes dominantly carbonate rocks of the Keyser, Old Port and Onondaga Formations; these units are present west of the indicated fault. The south limb of the Berwick anticlinorium shows the same relationships occur on the north limb; that is, that the Keyser, Old Port, and Onondaga Formations are missing along the more western portion. However, no fault has been indicated to account for the missing section along the south limb (Ref. 2.5-24).

Mapping on a scale of 1:24,000 (Figure 2.5-10) indicates that west of the confluence of Fishing Creek and Little Fishing Creek (and also west of the interpreted fault), the Tonoloway, Old Port, Onondaga and Marcellus Formations were recognized but no limestone of the Keyser Formation was identified. Thus, one of the units (the Keyser Formation) which is interpreted as missing due to faulting is absent from the stratigraphic section about 4,000 feet west of the postulated fault, despite the fact that the Old Port and Onondaga are present there.

Field investigations at two locations along the trace of the proposed fault (Stations DJ-4A, DJ-4B, DJ-32, DJ-33, and DJ-34; Figure 2.5-10) failed to reveal any evidence of cataclasis except for dip slip slickensides associated with a small flexural slip fold in the Marcellus Formation at DJ-4B. In both cases the Tonoloway limestone and Marcellus shale were exposed within 100 feet of the hypothesized fault. In the former case, however, at DJ-4A and DJ-4B the Tonoloway and Marcellus are separated by less than 20 feet. Consideration was given to the difficulty of detecting faulting within argillaceous units; however, where faulting was recognized within the study area, all lithologies (i.e., limestone, shale, mudrock, siltstone, and sandstone) showed some evidence of cataclasis. While continuous exposure across the postulated trace was not available, no positive evidence for faulting was observed and thus, at best, the fault can only be inferred. The only location along or near the trace of the postulated fault where significant cataclasis occurs is at Station DJ-31 (Figure 2.5-10) about 1,500 feet north of the "fault trace" where shale of the Mahantango Formation is exposed. Here the strata show noticeable variations in both strike and dip directions. For example, at the west end of the exposure strata swing from an attitude of N45°E: 35°SE to N40°W: 15°SW. In the center the attitude changes from N20°E: 30°SE to N10°W: 35°E back to N15°E: 45°SE, and at the eastern end, strata are oriented about N60°E: 40°NW and N35°E: 45°SE. At the western end of the outcrop the change in orientation represents a fairly smooth continuum whereas at the eastern end both continuous and discontinuous changes in orientation occur. The discontinuous changes are marked by oblique slip faults with slickensides displaying rakes of about 60° to 70°. Three such faults trend N55°E: 35°NW, N80°E: 35°NW, and N88°E: 29°NW. No unequivocal movement plan was identified, but structures recognized elsewhere in the Valley and Ridge Province, as well as the area within five miles of the site, show that reverse faults form in response to the release of stored strain energy in tightly appressed kink bands; thus, these faults are interpreted as relatively small scale accommodating reverse faults. This exposure (DJ-31) occurs within an area in which there is map scale folding producing deflections in the lithologies which is not unlike patterns seen elsewhere in the Valley and Ridge Province.

In the P. Good Well, located on the Berwick Anticlinorium east of the site (Figure 2.5-7), faulting has been interpreted at an approximate depth of 5,800 feet which is well above the depth range (17,000 to 25,000 ft) at which a major decollement is implied (Ref. 2.5-38). Thus, the decollement alluded to by Wood and Bergin (Ref. 2.5-21) as well as the fault seen on the Geologic Map of Pennsylvania (Ref. 2.5-24) 12 miles west of the site may actually be a splay fault(s) off a more deeply buried decollement. This suggested splay fault may also be, in part, responsible for the excess thickness seen in the P. Good Well of lithotectonic Unit 2, as defined by Wood and Bergin (Ref. 2.5-21).

In summary, no direct unequivocal evidence exists to either postulate or refute the existence of the fault which has been mapped on the north limb of the Berwick Anticlinorium west of the site (Ref. 2.5-24). However (1) the absence of the Keyser Formation on the north limb, west of the western limit of the inferred fault, and (2) the map pattern in which the units missing from the north limb of the anticlinorium are also absent from the south limb, yet the fault restricted to the north limb suggests that the missing section of the north limb is perhaps better explained by an unconformity than by faulting. Regardless of whether or not a fault is interpreted, data from the P. Good Well show that the limestone sequence missing from both the north and south limbs of the Berwick Anticlinorium is absent from the nose of the fold as well. Thus, if a fault is postulated, it pre-dates the formation of the Berwick Anticlinorium and does not pose a safety-related problem to the site.

Flanking the Berwick Anticlinorium on the north and south respectively, are the Lackawanna and Eastern Middle synclinoria. The Lackawanna Synclinorium, the axis of which passes about 3-1/2 miles north of the site, is about 120 miles long and displays a wavelength of 8 to 9 miles and an average amplitude of 4,000-5,000 ft (Ref. 2.5-21, Table 2). Near the axis of this fold the Mocanaqua decollement is exposed. The axis of the Eastern Middle Synclinorium is approximately 3-4 miles south of the site. This fold is about the same size as the Lackawanna Synclinorium (Ref. 2.5-21, Table 2).

The Nittany Anticlinorium is a large structure which is located just to the west of the Berwick Anticlinorium about 50 miles west of the site. Based on seismic records garnered from longitudinal and traverse profiles, there exists a series of well defined, subhorizontal velocity interfaces which are correlative with similar velocity contrasts in the Cambrian sequence and at the top of the Precambrian basement (Ref. 2.5-38, p. 134). The deepest interface was estimated to occur at a depth of about 25,000 $\pm$ 3,000 ft.

The Birmingham Fault is located in the core of the Nittany Anticlinorium and is the only fault in Pennsylvania associated with a major, well-exposed decollement known as the Sinking Valley Fault (Ref. 2.5-33, p. 350). This fault is a steeply, east dipping splay (thrust) which is about 33 miles long.

2.5.1.1.3.4 Relationship Among Structural Elements

In the opinion of all previous workers all of the structural elements encountered in the Valley and Ridge Province are genetically related, a feature which is clearly demonstrated in several instances. First, cross faults and reverse faults are generally spatially related with the reverse (or wedge) faults commonly terminating against the cross faults. In fact, there are instances where one passes into the other. A prime example of this is in the Carbaugh-Marsh Creek Fault in the southern Great Valley. This fault is composed of two segments, an east dipping reverse fault which parallels the regional grain and passes continuously into a nearly east trending, subvertical cross fault across which right lateral separation has occurred (Ref. 2.5-37, p. 8, 9 and 822 and 2.5-35, p. 102-103). Rock units across this cross, or transverse, fault segment have shortened independently of one another. Because of this independent behavior of rock units across the transverse portion of this fault, Root (Ref. 2.5-35, p. 103) concluded that this segment existed prior to most of the regional deformation.

Faill and Nickelsen (Ref. 2.5-31) and Faill (Ref. 2.5-32) pointed out that slickenside orientations on: 1) bedding surfaces associated with flexural slip folding, 2) wedge faults, and 3) cross faults are similar) indicating kinematic compatibility among these three structural elements. Furthermore, the lines of intersection of the wedge faults and bedding are subparallel to the fold axes. Splay faults with large displacements occur in the hinges of anticlines which posses a kink band geometry suggesting that these folds were produced by the splays which originate at depth along unexposed decollements (Ref. 2.5-38, 2.5-37 and 2.5-33, p. 349). Faill (Ref. 2.5-32, p. 1298) also conceded that possibly all major anticlinoria are underlain by splay faults. Prior to Gwinn's work, the existence of major decollements and thin skinned tectonics in the Appalachian orogen was somewhat debatable but the results of his latest study (Ref. 2.5-38) indicate clearly that these subsurface structures exist.

In the southern Great Valley, Root (Ref. 2.5-35) deduced the sequential development of folding and faulting. He suggested that the cross faults existed prior to, or at the initiation of, most of

the regional deformation, although he indicated that their origin is not understood. Nonetheless, he concluded (Ref. 2.5-35, p. 109) that the cross faults which are subvertical and normal to the regional grain were, during folding, equivalent to ac fractures. This is logical since the orientation of cross faults with respect to the other structural elements is not consonant with having originated as shear fractures; rather it is likely that they formed early in the tectonic history of this area as extension (ac) fractures which were subsequently utilized as strain discontinuities (tear faults) during the same protracted period of stress application. This interpretation is supported by Nickelsen and Hough (Ref. 2.5-39) who stated that systematic extension fractures in shales are grossly transverse to northeast-trending fold axes, and formed early and independently of folding and faulting. Root continued (Ref. 2.5-35, p. 111) by indicating that the earliest folds were broad and open and as horizontal shortening continued and the folds became more appressed; west dipping steep thrusts (wedges) formed in the upper strata. Continued shortening resulted in the development of subsidiary folds, east dipping thrusts (splay faults) and the rotation of the earlier formed, west dipping thrust to a steeper inclination. In the case of the Carbaugh-Marsh Creek Fault the east dipping splay fault linked up with a portion of the cross faults to form the Carbaugh-Marsh Creek Fault system.

Faill and Nickelsen (Ref. 2.5-31, p. 37) noted that deformation was initiated by vertical compaction during sedimentation. Regional horizontal compression followed, while the strata were still horizontal, with the earliest stage marked by microfolding in shales and limestones. Major decollements probably also occurred during this early stage and were followed by buckling and kink band folding. As the folds tightened, faults in the hinge along with some wedge and cross faults, developed. These faults were accompanied by tightly spaced fractures (fracture cleavage) which formed parallel to the axial planes of the folds. Faill and Nickelsen further concluded that with time the deformed materials passed progressively from a ductile stage to a more brittle stage.

2.5.1.1.3.5 Age of Deformation

Root (Ref. 2.5-35) concluded that all the structural elements in the craton developed during a single orogenic event (Alleghenian) about 230 million years ago. This age is based on an inferred episodic lead loss about 230 million years ago recorded by Rankin (Ref. 2.5-40) in zircons from the Catoctin Formation. However, Faill and Nickelsen (Ref. 2.5-36, p. 19) implied that the deformation occupied a greater time span, possibly commencing prior to complete lithification of Silurian age sediment (e.g., the Lower Silurian Tuscarora Formation). Despite this apparent difference concerning the age of the onset of deformation, there is general agreement that the major structural elements in the Fold and Thrust Belt and the Stable Interior are no younger than Late Permian to Middle Triassic in age (Ref. 2.5-37, 2.5-33, 2.5-35, 2.5-32, 2.5-31, 2.5-38, 2.5-41, 2.5-34, and 2.5-21).

There is considerable disagreement as to the age, nature and method of formation of the curvature that is so prominent along the entire Appalachian Chain. Drake and Woodward (Ref. 2.5-42, p. 49) concluded that this arcuation in central Pennsylvania is truly a rounded structure which formed in response to right lateral slip along the east trending Cornwall-Kelvin Fault, perhaps around Late Devonian time (Ref. 2.5-42, p. 29). Faill (Ref. 2.5-32, p. 1305-1306) noted that it is not smooth and continuous, as it appears, but instead is composed of straight segments, the joining of which marks the boundary between the northern and southern Appalachians. Furthermore, he concluded that this "curvature" and the major folds were contemporaneous. Root (Ref. 2.5-37, p. 825) noted the same observations as Faill and

indicated that these rectilinear elements oriented N17°E south of the Carbaugh-Marsh Creek Fault and about N40°-50°E north of the fault, assume their present position by rotation across the fault. He also added that all structural elements seen in the Piedmont to the southeast would be more compatible if the Piedmont were arcuate by the Early Paleozoic. This appears to be a contradiction since on the one hand, Root suggested that rotation across the Carbaugh-Marsh Creek Fault (presumably during the Late Paleozoic Alleghenian event) was responsible for this curvature, whereas on the other hand, he suggested that the curvature already existed by Early Paleozoic time. Fleming and Sumner (Ref. 2.5-42, p. 58) suggested that an embayment associated with the Late Precambrian-Early Paleozoic proto-Atlantic was responsible for this arcuation in central Pennsylvania. Rankin (Ref. 2.4-40) and Rodgers (Ref. 2.5-44) stated that Appalachian salients and recesses formed during the initial breakup of a continental mass which commenced about 820 million years ago. Thus, at present, although there is no unique hypothesis concerning the age and origin of this curvature there is general agreement that it is pre-Mesozoic in age.

Evidence of younger tectonics is confined to the mobile belt. The southern border of the Newark-Gettysburg basin is obscured in New Jersey by the overlap of Coastal Plain sediments; however, in Pennsylvania and Maryland, the Triassic sedimentary rocks lie unconformably upon lower Paleozoic quartzites and carbonates and, in a few areas, upon Precambrian gneisses, granites and metabasalts. Residual gravity anomalies indicate that southern "border faults" are covered by the younger Triassic sediments (Ref. 2.5-45).

The northern edge of the basin, in the area east of the Schuylkill River, borders on the granitic-gneissic complex of the New Jersey Highlands and its southwest extension, the Reading Prong. West of the Schuylkill River, rocks north of the border are Cambrian and Ordovician carbonates. Triassic rocks unconformably overlie adjacent older rocks along much of the northern border suggesting that this margin is not continuously faulted (Ref. 2.5-2). In Pennsylvania, only 35 percent of the margin is known to be faulted (Ref. 2.5-46). The northern border faults are characterized as an echelon fault zones that gives a crenelated appearance to the northern margin. Where overlap has occurred the contact dips approximately 20% to the south (Ref. 2.5-46).

The Ramapo Fault System, a continuation of the northeast trending system of border faults, crosses the New York-New Jersey State boundary north of New York City. No evidence of surface rupture, warping or offset of geomorphic features has been observed along its member fault zones (Ref. 2.5-47).

Several faults of apparently large displacement occur within the center of the Newark-Gettysburg basin (Figure 2.5-11). These are the Chalfont and Furlong Faults in Pennsylvania and the Flemington and Hopewell Faults in New Jersey. Orientation and direction of movement of these faults are not known. Although generally considered to be steeply south dipping normal faults (Ref. 2.5-48 and 2.5-49), Sanders (Ref. 2.5-50) has suggested predominant strike slip movement and Faill (Ref. 2.5-46) indicates they may be high angle reverse faults resulting from intersection of two different axes of monoclinal folding within the basin.

Smaller Triassic faults cross cut the basin margins and extend well into the surrounding rocks, but usually show less than 3,000 feet of displacement. Associated with these faults are local concentrations of small faults of constant attitude and sense of displacement (Ref. 2.5-2).

The Triassic basins and associated faulting are located in the mobile belt, whereas the site is situated on the craton which includes the Fold and Thrust Belt. No tectonic structures of Mesozoic or younger age have been recognized in the Fold and Thrust Belt.

Analysis of lineaments observable on LANDSAT imagery yielded data consistent with these observations. The greatest number of linears plotted in the Valley and Ridge Province within the Appalachian salient strike N10-25°W (Figure 2.5-11). This is roughly normal to fold axes in the area and thus the cluster of lineaments parallels the direction of extension fractures and cross faults. The fold axes and bedding are well expressed as a cluster of east-northeast trending lineaments.

Secondary trends oriented north-northeast and northeast may indicate jointing of Mesozoic age. These are the dominant lineament trends expressed in the Newark-Gettysburg Basin.

2.5.1.1.3.5.1 Relationship of Lineaments to Regional Geology

The remote sensing work for the Susquehanna SES FSAR utilized landsat satellite imagery. The imagery was obtained from the EROS Data Center, Sioux Falls, South Dakota. Analysis of the five frames was performed on 20" x 20" black-and-white prints at a scale of 1/500,000 (Imagery Access Nos. 1495-15222, 1079-15124, 1440-15172, 1403-15123, and 5359-14433).

Images from the Landsat satellite comprise individual frames recorded in different bands of the electromagnetic spectrum. For this analysis, two bands (5 and 7) for each frame were analyzed. These bands were utilized because band 7 (0.8 to 1.1 micrometer) shows drainage much better than the other bands while band 5 (0.6 to 0.7 micrometer) emphasizes cultural features. Each frame covers an area of approximately 13,225 square miles. This investigation encompassed the area within a 100 mile radius about the Susquehanna SES.

In the analysis of satellite imagery an acetate overlay was initially prepared showing all of the lineaments observed on band 7 of each frame. These overlays were then registered to band 5 of their corresponding frames, and any additional lineaments observed on these frames were then added to the overlays. It was possible to observe much of the study area (approximately 25 to 30 percent) in stereo due to the overlap of the adjacent frames. Using a mirror stereoscope corresponding bands of the adjacent frames with their previously analyzed overlays were studied. This added three dimensional view of the lineaments provided a means by which spurious, apparently culturally controlled lineaments (e.g. roads and transmission lines) could be eliminated.

A base map was prepared from the 1/250,000 topographic map series (Army Map Service), that covered the study area. This map was reduced to 1/500,000 scale to match the imagery scale and to facilitate transfer of data from the analyzed satellite imagery. A separate overlay was prepared from the geologic maps of New York, New Jersey and Pennsylvania, delineating the fold axes, intrusives, faults and physiographic provinces of the region. This overlay was brought to the common scale of the imagery and topographic base maps. Having all data at the same scale facilitated comparison and interpretation and also permitted elimination of other cultural lineaments.

Comparison of FSAR Figures 2.5-7 and 2.5-11 reveals that within the Valley and Ridge Province lineament trends strongly reflect the structural grain of the Appalachian Salient.

The greatest number of lineaments are roughly normal 75° - 105° (to the fold axes and consistent with the directions along which cross faults and extension fractures occur (FSAR Subsection 2.5.1.1.3.2). The next greatest number of lineaments trend parallel to the fold axes indicating their control by bedding, foliation, thrust faults, and wedge faults. Secondary trends, particularly evident within the Appalachian Salient, are N10°-15°E, N20°-25°E and N25°-35°E. These are the dominant trends observed in the Newark-Gettysburg Basin and probably reflect jointing developed under the early Mesozoic stress regime. Conversely the dominant N40°E trend observed in the Precambrian rock of the New England Uplands is virtually non-existent in the Valley and Ridge Province (except where fold axes trend near N40°E).

Within the Appalachian Plateau Province the dominant trends are also parallel or near normal to the fold axes. Secondary trends oriented north-northeast and northeast which may indicate jointing Mesozoic age are also evident in the Plateaus Province, particularly in the eastern portion of the study area which is nearest to the Newark-Gettysburg Basin.

Thus, the linear features observed during this investigation which may be of structural origin can be ascribed either to Paleozoic tectonics or the early Mesozoic stress regime. No evidence of younger (Cenozoic-Recent) structural elements was observable.

The relationships between the results of this analysis and the work of Saunders and Hick, 1976 are not geometrically direct. The problem is one of both scale and concept. Saunders and Hick worked with features traceable for hundreds of miles which they feel are reflective of fundamental crustal tectonics. The analysis for Susquehanna SES was, relatively speaking, local in both scope and approach. The greatest part of this study area encompasses allochthonous rocks which contain structural elements derived from "thin-skinned" tectonics as opposed to fundamental basement tectonics. Saunders and Hick have related the major geomorhphic lineaments of the United States to the currently accepted theories of plate tectonics and hypothesized that these lineaments have been of primary importance in controlling crustal tectonics. It may be stated that their work dealt with the cause while the Susquehanna SES analysis, due to the allochthonous nature of the terrain, dealt with the effect of plate tectonics. Whether or not their postulated mechanisms for the cause are accurate, it is generally accepted that the Atlantic Ocean has opened and closed at least twice in the geologic past. The analysis summarized in the FSAR Subsection 2.5.1.1.3.5 and Figures 2.5-7 and 2.5-11) revealed that the significant lineaments of the Valley and Ridge and Appalachian Plateaus provinces in the region of the Susquehanna SES are the effect of the Paleozoic closing of Iapetus (proto-Atlantic Ocean). Secondary lineaments of these provinces can be related to the initial opening of the Atlantic in early Mesozoic time.

<u>2.5.1.1.4 Regional Uplift and Subsidence</u>

Several investigators have presented evidence for present day crustal movement in the Atlantic Coastal Plain, the Fold and Thrust Belt and exposed shield areas (Ref. 2.5-51, 2.5-52, 2.5-53 and 2.5-54). Based on the accumulation of an eastward facing clastic wedge of sediments along the coastal plain, Owens (Ref. 2.5-52) concluded that post-Triassic diastrophism has affected the entire central and southern Appalachians with the latest recorded upwarping having occurred in the Pliocene to Quaternary. Brown and Oliver (Ref. 2.5-54) concluded that the "Appalachian Highlands are presently rising relative to the Atlantic Coast at rates of up to 6 mm/yr" with the elongate zones of relative movement paralleling either the major Appalachian Structural trend or the Appalachian drainage divide. Superimposed on this broader uplift are local zones marked by a slightly greater rate of vertical crustal movement. One of these, known

as the Harrisburg feature, occurs near the eastern limit of the Valley and Ridge Province along a line which extends northward from the eastern edge of the Blue Ridge Province (Ref. 2.5-54, p. 26). They further suggested that the entire thickness of the lithosphere is involved in these movements.

No instances of faulting due to tectonism have been associated with this regional scale activity. The only known instance of surficial displacements in the Fold and Thrust Belt occurs in New York (about 120 miles east of the site) and New England where small scale (less than one inch of vertical separation), high angle reverse faults that parallel the regional tectonic fabric offset glacial striations. Oliver and others (Ref. 2.5-51) have considered glacial rebound and surficial effects such as thermal changes, hydration or a chemical process in the shales as well as tectonic stresses as possible causes of these faults. While admitting the available data are inconclusive, they appear to favor the hypothesis that the faults are the result of expansion due to hydration or the release of continuing pressure by melting of overlying ice or other causes. They further suggest that if the faults are of tectonic origin, an apparently poor correlation between fault locations and modern seismicity indicates that episode of deformation is already completed (Ref. 2.5-51, p. 587). No structures of this nature have been found in Pennsylvania.

As discussed further in Subsection 2.5.1.2.3, the available data do not indicate that regional uplift is of significance to the Susquehanna SES.

2.5.1.1.5 Natural Hazards

A natural hazard has been defined by Burton and Kates as "those elements of the physical environment that are potentially harmful to man and his works". Thus, geological natural hazards would be potentially harmful geological elements of the physical environment. The geologic hazards to be considered are: subsidence due to coal mine collapse, subsidence due to karst collapse, and landslides.

The coal (anthracite) of northeastern Pennsylvania is located in the Northern, Middle, and Southern Anthracite fields. The southwest end of the Northern Field is the closest to the site being about 4 miles to the northeast (Figure 2.5-7). Within this Northern Field there are many well documented incidences of subsidence, particularly in the cities of Scranton, Wilkes-Barre, Nanticoke, and Pittstown (Ref. 2.5-55). There have also been incidences of subsidence in the Middle and Southern Fields, which at their closest points, are about 10 miles southeast of the site (Fig. 2.5-7). Thus, the site will not be affected directly by subsidence due to coal mine collapse.

The nearest major carbonate units are the Silurian Keyser and Tonoloway Formations which are composed of gray to dark gray, thick to thin bedded, crystalline to argillaceous limestones (Ref 2.5-24). These formations are not major cavern producers (Ref. 2.5-56) especially in this portion of Pennsylvania, and thus do not pose a hazard of collapse. As mapped, these two formations occur as relatively thin beds on both limbs of the Berwick Anticlinorium which come together in the town of Berwick, Pennsylvania. This location is about 5 miles west of the site; thus, these units would pose no subsidence problems at the site. Miller (Ref. 2.5-57) stated that Onondaga limestone crops out along road and railroad cuts near Beach Haven, Pennsylvania. Examination of these outcrops indicates that the rock is dark gray, brownish weathering calcareous silty mudrock interbedded with thin layers of silt to clay shale or with siltstone.

Lithology and fossil fauna indicates that these rocks belong to the Mahantango Formation (Subsection 2.5.1.2.2) which does not pose a subsidence problem at the site.

Radbruch-Hall (Ref. 2.5-58) placed the site in a region of moderate landslide incidence with a high susceptibility to landsliding. Moderate incidence means that generally less than 15 percent, but more than 1.5 percent, of the underlying rock or earth material is estimated to be involved in landsliding. A high susceptibility means that natural or artificial cutting, loading of slopes or anomalously high precipitation may cause landsliding involving more than 15 percent of the rock or soil. On this regional basis no specific statement can be made on local susceptibility to landsliding. However, some general statements can be made. Although moderate to steep, natural slopes of the local formations (Marcellus, Manhatango, which is stratigraphically equivalent to the Hamilton, and Trimmers Rock) are stable, cut slopes generally have only poor to fair stability due to rapid disintegration of the shales upon exposure to weathering. Much of the surface area in the vicinity of the site is covered with glacial till and outwash. The stability of this material in cut slopes needs to be carefully analyzed. Slope stability and landslide potential at the site are discussed in greater detail in Subsection 2.5.1.2.5.

2.5.1.2 Site Geology

2.5.1.2.1 Site Physiography

The Susquehanna Steam Electric Station is located in the Valley and Ridge Physiographic Province which is described in Subsection 2.5.1.1.1. The site is situated within a broad undulating valley developed in mudrock, shale and siltstone of the Devonian Mahantango Formation along the axis of the Berwick Anticlinorium (Figure 2.5-12).

Lee Mountain (about 2-1/2 miles north) and Nescopeck Mountain (about 4-1/2 miles south of the site, are held up by the more resistant sandstone and conglomerate of the Mississippian Pocono Group. Lesser ridges formed by sandstone of the Trimmers Rock Formation occur at the north end of the site and along the south bank of the Susquehanna River (about 2 miles south of the site).

Topography and drainage of the area is controlled to a large degree by the lithologic and structural characteristics of the bedrock. Ridges and valleys generally trend east-northeast parallel to the strike of the Paleozoic strata. The site fronts on the south flowing Susquehanna River which here flows perpendicular to the east-northeast trending axis of the fold, resuming its west-southwest flow about 1-1/2 miles south of the site, to follow the strike of the shale valley. The north-northwest segment of the Susquehanna River which flows normal to the strike appears to have been inherited from the course of the Ancient Little Schuylkill River (Ref. 2.5-3) (Refer to Subsection 2.5.1.2.4).

Topographic elevations in the site vicinity range from 500 to 1,100 feet above sea level. Higher elevations occur in the more rugged terrain further north and west of the site. The site itself contains generally gentle to moderately sloping hills and well developed drainage patterns. Existing surface elevations vary from about +750 feet in the western portion to about +500 feet in the east. Portions of the area were formerly cultivated. In those areas not cultivated, heavy to moderate woodlands and scrub brush are found. A steep sandstone ridge borders the north side of the site. A narrow east-west trending interior bedrock ridge, rising some 60 feet above the surrounding ground surface, is located just north of the center of the site. A rounded

bedrock knoll about 80 feet high occurs at the western edge of the site in the southwest quadrant.

The site is well-drained by eastward trending depressions near the north and south edges of the site. Plant grade at 650 feet above sea level (about 150 feet above the flood plain of the Susquehanna River) is at about the level of the Fourth Olean Kame Terrace (Ref. 2.5-5) which is well preserved southward between the site and the Susquehanna River.

The irregular bedrock surface underlying the site is the result of a combination of preglacial weathering and stream erosion, glacial scour, later erosion by glacial melt waters, and the varying resistance of the lithologic units to erosion. The maximum thickness of the overburden is on the order of 40 feet in the southern half of the site, with bedrock occasionally cropping out at the surface. North of the east-northeast bedrock ridge that is located near the center of the site just north of the reactor and turbine buildings, glacial deposits fill a bedrock valley to a depth exceeding 100 feet.

During excavation at the site, abundant evidence of glacial and glacio-fluvial scour of the bedrock surface was found in the form of channels, potholes, grooves, striations and fluted rock. A large, buried pothole over 30 feet wide and more than 30 feet deep was exposed in the Unit 1 turbine building excavation (Figure 2.5-13). Similar large buried potholes have been documented farther north along the Susquehanna River Valley (Ref. 2.5-3, p. 23-27 and 2.5-59, p. 195). At the site, numerous other smaller potholes and rounded pits and channels in unweathered bedrock were observed in the excavations. Smooth, east-northeast trending linear channels about 6 to 8 feet deep eroded in unweathered bedrock were observed north of the radwaste building. Similarly, somewhat larger features were excavated in the northeast and west rims of the Unit I cooling tower. This fluting of the rock surface observed in a number of places at the site was either gouged by ice, eroded by water or both, and apparently served as flumes for torrential glacial meltwater runoff which evidently at one time cascaded across much of the site area. Undoubtedly many steep or even undercut surfaces of the bedrock at the site are attributable to ice scour and intense fluvial erosion that was associated with the Olean and earlier glaciations. These features are discussed in Subsection 2.5.I.2.3.3.

As indicated in Subsection 2.5.1.2.5, landslide potential, surface or subsurface subsidence, uplift or collapse are not of concern at the site.

2.5.1.2.2 Site Lithology and Stratigraphy

2.5.1.2.2.1 Lithology and Stratigraphy in the Site Vicinity

Figure 2.5-12 illustrates the distribution of the geologic units within at least 5 miles of the site. The stratigraphic relationships of the various formations are shown on the site geologic column (Figure 2.5-14). Silurian and Devonian formations occur throughout the Valley and Ridge Province. Silurian and lower and middle Devonian strata consist of marine shale, mudrock, siltstone, sandstone and limestone. The upper Devonian strata are generally non-marine sandstone and shale.

A northeast trending fold, referred to as the Berwick Anticlinorium, completely encompasses the site area. This feature has been breached by erosion, exposing rocks of Silurian and Devonian age along the core and at the flanks of the anticlinorium. As it plunges to the east, progressively

younger formations are exposed. Silurian formations present west of the site include, from oldest to youngest: the Tuscarora sandstone, the Clinton ferruginous sandstone, the McKenzie greenish shale with limestone, the Bloomsburg red shale, the Wills Creek shale and the Tonoloway limestone. The Tuscarora sandstone caps Montour Ridge along the axis of the Berwick Anticlinorium in the vicinity of the West Branch of the Susquehanna River. Here as elsewhere in the Valley and Ridge Province, the Tuscarora is a prominent ridge former. The Clinton Formation contains a fossil iron ore which was formerly mined along Montour Ridge. The Bloomsburg Formation supports the eastern extension of Montour Ridge. The Wills Creek and Tonoloway Formations occur in the flanks of the Anticlinorium west of Berwick (Figure 2.5-10).

The basal Devonian formations are the Keyser limestone and Old Port sandstone. These formations crop out along the flanks of the Berwick anticlinorium. East of Bloomsburg they are no longer exposed having presumably been removed from the section by erosion or faulting (Subsections 2.5.1.1.3 and 1.5.1.2.3).

Stratigraphic units exposed in the map area are from oldest to youngest: the Devonian Mahantango (which includes the Marcellus Shale, Trimmers Rock and Catskill Formations), the Mississippian Pocono Formation, the Mississippian to Pennsylvania Mauch Chunk Formation, and the Pennsylvania Pottsville and post-Pottsville Formations.

Above the Marcellus, the Mahantango Formation is represented only by the uppermost member, the Sherman Creek which is dominantly a dark gray to blue gray, olive gray to brown weathering mudrock. Siltstone and fine grained sandstone units crop out locally, including at the site and at certain outcrop locations both calcareous and noncalcareous strata coexist (Figure 2.5-12, Stations DF-2, DF-3, and DF-6).

An interval of light, medium gray argillaceous limestone near the top of the Mahantango, was recognized as correlative with the Tully Limestone and was mapped as part of the Mahantango (Figure 2.5-12, Stations DF-53 and DF-45). The overlying Harrel Shale, a poorly exposed, dark silty shale which appears to gradationally overlie the Mahantango (Figure 2.5-12, Stations DF-30, DF-31, DF-43, and DF-44b) was also mapped as part of the Mahantango Shale.

Fossils are relatively abundant within the Sherman Creek member of the Mahantango Formation and include various genera of brachiopods, bryozoa, pelecypods, coral, trilobites, and crinoid fragments. Fossil casts are abundant with occasional molds and rare preservation of internal structure and original shell material.

Concretions (commonly rusty weathering), spheroidal weathering, and prominent, closely spaced steeply dipping cleavage, which may quite easily be mistaken for primary bedding fissility, are other features characteristic of the Mahantango.

The Mahantango grades upward into the Trimmers Rock (Figure 2.5-12, Stations DF-30 and DF-33). The Trimmers Rock Formation is dominantly interbeded, medium to olive gray, thinly laminated siltstone, silty shale and fine grained, laminated to massive sandstone. These rocks weather to a brownish gray color. Sedimentary structures include fining-upward sequences (Figure 2.5-12, DF-7, DF-8, and JW-10); groove casts, current lineations, load casts, ball and pillow structure, and flow rolls (Figure 2.5-12, JW-7B, JW-10, and JW-11). Ripple marks were also locally identified. These structures indicate deposition by turbidity currents in a marine environment. Fossils are often restricted to relatively thin layers of brachiopods (spirifers DF-9).

Other fossils include pelecypods and crinoid fragments (DF-17b). Channels were observed at DF-25.

The upper Trimmers Rock Formation consists of light to medium grayish green silty shale and micaceous, dark greenish gray silstone, both of which weather to a dark reddish brown color, a reddish brown silty fine to medium grained sandstone to siltstone, and olive green vitreous fine grained sandstone to siltstone (DF-26, DF-27, DF-28, DF-32, DF-36, DF-37 and DF-68).

The Catskill Formation rests conformably upon lithologically similar interlayered rocks of the upper Trimmers Rock but is predominantly red colored. It is this dominantly red color as well as sedimentary structures (roots, oscillation ripple marks; Station DF-68) which distinguishes the Catskill from the Trimmers Rock. The contact between the Catskill and the underlying Trimmers Rock was mapped therefore, at the base of the first relatively thick reddish brown to maroon sandstone (Figure 2.5-12, Station DF-68) or brownish red silstone and more massive reddish brown (maroon) micaceous, fine grained sandstone (Figure 2.5-12, Station DF-37c). At Station JW-65 (Figure 2.5-12), the upper Trimmers Rock consists of fine grained, medium gray sandstone overlain by a thin band of green mudrock which is, in turn, overlain by a fine grained, green, well laminated sandstone. The green sandstone grades upward into a fine grained, red, well laminated sandstone which marks the basal unit of the Catskill.

The basal red unit at Station DF-37c (Figure 2.5-12) is overlain by greenish gray, fine grained sandstone and olive green shale and at DF-68 it is overlain by thinly laminated, light green, silty shale and siltstone. These units are succeeded, upward, by interbedded maroon and light olive gray to greenish gray sandstone, siltstone, and shale (Figure 2.5-12, Stations DF-37c and DF-68). Stratigraphically younger units within the Catskill include: (a) reddish brown mudrock and silty mudrock, (b) brownish red medium grained sandstone, (c) reddish brown to maroon micaceous, fine grained sandstone, and (d) greenish gray micaceous, fine to medium grained sandstone. Sedimentary structures include intraformational clasts of green shale, oscillation ripple marks, roots, and prominent cross bedding. The Pocono Formation, which overlies the Catskill, consists typically of medium and coarse grained light gray to white, rusty weathering quartz sandstone with thin layers of quartz pebble conglomerate. Olive gray, fine grained sandstone, reddish gray medium to fine grained sandstone and siltstone (Figure 2.5-12, Station DF-14) and greenish gray medium grained, cross bedded sandstone (Figure 2.5-12, Station DF-67) also occur within this formation. Cross bedding is common.

Near the base of the Pocono, grayish red sandstone layers occur. These were recognized along the east side of the Susquehanna River south of Mocanaqua (Stations JW-1 and JW-64) and along the road between Alden and Folstown (Stations JW-28 and JW-29). At JW-1 and JW-64, the Pocono consists of an interlayered sequence of predominantly medium gray, thick, well laminated, gray weathering quartz sandstone and subordinate, red, flaggy quartz sandstone. At JW-28 well laminated red sandstone is interlayered with, but decidedly subordinate to, well laminated, rusty weathering, light gray, coarse grained sandstone and finer grained gray sandstone. Coarse to medium grained, mainly grayish to greenish gray sandstone featuring rather subtle cross bedding dominate the upper portion of the exposure at JW-29. This, along with an underlying thin zone of light greenish gray sandstone, in turn, underlain by green shale and mudrock, has been selected as marking the basal Pocono. Beneath all of these units at JW-29, is a red, well laminated, argillaceous siltstone which we have interpreted as marking the top of the Catskill. This red siltstone rests upon a green shale which overlies a cross bedded, medium light gray, olive gray weathering quartz sandstone. Red shale, which marks the base of the outcrop, underlies the quartz sandstone. The lower (topographically and

stratigraphically) portion of the outcrop is dominated by red lithologies in contrast to the upper part in which no red lithologies were exposed. Besides the obvious color change the sandstone, above the inferred contact, is coarser grained and more subtly cross bedded than the sandstone which occurs between the red units near and at the base of the outcrop. Thus, contrary to other interpretations (e.g. Ref. 2.5-16) we suggest that the Pocono-Catskill contact is gradational in this area, rather than unconformable.

The upper Pocono Formation in the vicinity of Shickshinny consists of medium to light gray conglomeratic sandstone with rounded to sub-rounded quartz pebbles and shale fragments, and rusty weathering, fine to medium grayish green micaceous siliceous sandstone (DF-56) and finely laminated greenish gray, rusty weathering, siliceous quartz sandstone (DF-57, DF-58). Rusty weathering, medium light gray, medium to coarse grained quartz sandstone is interbedded with thin layers of dark gray silty shale and medium gray quartz-lithic sandstone fills channels at DF-59.

The Mauch Chunk Formation is generally bright red in color and consists of mudrock, silty shale, siltstone and fine to medium grained, cross bedded, well laminated sandstone. The upper part of the formation along the south limb of the Lackawanna Synclinorium (Figure 2.5-12, Stations JW-22 to JW-24) is marked by interlayered red and olive gray sandstone, siltstone, and silty shale. Locally the siltstone contains layers of rounded, circular to elliptical calcite filled voids. Elsewhere the Mauch Chunk consists of greenish gray to grayish green medium to coarse grained, locally micaceous sandstone, thinly laminated gray, fine grained sandstone and siltstone (Figure 2.5-12, Station DF-54) and massive medium grained sandstone (Station DF-55).

The Pottsville and Llewellyn Formations represent the coal bearing zones of the Anthracite Region and have, for the purpose of this report, been combined and treated as a single formation. Collectively, the Pottsville and Llewellyn (formerly post-Pottsville) consist of quartz pebble conglomerate in a quartz sandstone matrix, quartz pebble conglomerate in a carbonaceous quartz sandstone matrix, coarse grained dark to medium gray, massive and flaggy, carbonaceous sandstone and shale, dark gray to black siltstone and coal. The non-carbonaceous quartz pebble conglomerate displays cross beds.

Pleistocene unconsolidated deposits of glacial drift blanket most of the region north of the site. They extend approximately 10 miles to the south and 50 miles to the west of the site. Deposits of various glacial advances are recognized in the region. The drift materials include glacial till and stratified waterlain deposits consisting of poorly sorted mixtures of clay, silt, sand, gravel and boulders. The youngest and best preserved deposits are those of the Wisconsinan glacial stage.

The site lies just behind the Pleistocene terminal moraine of Olean drift, deposited between 55,000 and 60,000 years ago (Ref. 2.5-60, Figure 4; 2.5-61, plate 3; and 2.5-5, p. 25). The Olean drift represents an early glacial substage of late Pleistocene, or Wisconsinan time and has been correlated with the Altonian substage by Sevon (Ref. 2.5-62) to distinguish it from the later Wisconsinan, or Woodfordian, drift farther north. The Olean drift is an assemblage of contemporaneous drifts deposited by several ice lobes that occurred from New Jersey westward to Indiana, believed to represent a regional glacial advance in early Wisconsinan time. A correlation chart of deposits of early and middle Wisconsinan age by Dreimanis and Goldthwait (Ref. 2.5-60, Figure 4) utilizing available geomorphic, lithologic, paleontologic and radiocarbon data, shows the Olean drift to be between about 55,000 and 60,000 years old; conservatively,

the Olean drift may therefore be considered to be in excess of 50,000 years old according to this correlation. Drifts recording later ice advances in Wisconsinan time are not present in northeastern Pennsylvania (Ref. 2.5-61, plate 4), so in this area evidence of the earlier Wisconsinan drift is preserved (Ref. 2.5-62 and 2.5-63).

The leading edge of the Olean terminal moraine is depicted by Denny and Lyford (Ref. 2.5-61, plate 4) as occurring in the Susquehanna Valley about three miles southwest of the site, just west of the village of Beach Haven. Ahead of (downstream from) this moraine are deposits left by an earlier Illinoisan glaciation (Ref. 2.5-7, p. 24); however, no Illinoisan deposits have been recognized north of the Wisconsinan terminal moraine in Pennsylvania (Ref. 2.5-5, p. 26), indicating that Olean ice overrode and reworked apparently all of the pre-existing Illinoisan drift. Nevertheless, it is possible that some buried drift at the site and elsewhere, particularly that located in bedrock depressions, may represent unrecognized remnants of overridden Illinoisan or earlier deposits.

The glacial deposits near the Susquehanna site have been studied in some detail by Peltier (Ref. 2.5-5). He describes (Ref. 2.5-5, p. 25) the various features and processes associated with the terminal moraine near Beach Haven. He characterizes the morainic material as "a gravel moraine... composed largely of poorly sorted, coarse kame gravel, medium-grained valley train gravel, and sand... During the early stages of kame terrace development, the marginal channels flowed at a level which was high above the valley... Continued ablation of the ice in the valley probably caused the marginal streams to flow at successively lower levels. These streams, where they flowed along the ice, both eroded the earlier deposits and filled in their channels... In this manner any till deposited at the ice front became buried or eroded." This description of erosion and deposition near the site by ice-margin streams at elevations above the valley floor is consistent with the development of large potholes and steep or even undercut erosional contacts at the site. Evidently waterfalls and large-volume torrential streams occurred at the site during retreat of the early Wisconsinan ice. (Additional discussion of the origin and features of glacial deposits at the site is presented in Subsection 2.5.1.2.3.3).

Peltier (Ref. 2.5-5, Figure 33) profiles discontinuous kame terraces along a 25-mile stretch of the Susquehanna River including the site. The highest such terrace formed by a stream marginal to Olean ice is indicated to occur at about 650 ft. msl at the site (about mile 165), or about 160 ft. above the river. Glacial deposits at elevations higher than this, which would include the glacial deposits in most of the site area, would be part of either the Olean terminal moraine or the ground moraine behind it.

The moraine in the Berwick-Beach Haven area is noncalcareous (Ref. 2.5-5, p. 24). Sedimentary rocks, mostly gray and red sandstone and siltstone, constitute over four-fifths of the material in the moraine (Ref. 2.5-5, Table 3). Peltier (Ref. 2.5-5, p. 24) considers that the remaining igneous and metamorphic types in the moraine indicate it was derived from the Mohawk tongue of an Olean ice lobe originating from the Adirondack area or east of it (Ref. 2.5-60, p. 83).

Unconsolidated sediments mantle most of the Susquehanna River Valley within 5 miles of the site. The valley deposits consist of glacio-fluvial deposits (outwash alluvial terraces, kame terraces), alluvium and colluvium. Unconsolidated deposits were examined at Stations DF-4, DF-15, DF-16, DF-37, DF-42, DF-44a, DF-44b, DF-52, DF-64, and DF-66 at all DJU stations. Thin deposits were noted at various other localities (Figure 2.5-12).

Station DJU-1 is located at a currently (Spring, 1977) operating gravel quarry exhibiting excellent exposures. This quarry contains well layered, brownish gray, very coarse sand to medium gravel interlayered with gray, medium, well sorted gravel with medium to coarse gravel and cobble layers. This is overlain by pebble to cobble gravel with coal and rare boulders interlayered with coarse sand to medium gravel with little coal. The dip of bedding tends to decrease or flatten toward the south. The middle level contains medium to coarse, well rounded, gravel with coarse sand containing lenses of cobbles and gravel below fine to medium gravel and coarse sand with some coal rich laminae. The overlying unit is generally finer grained and dominantly fine to medium sand, some silt with fine laminae of coal. This unit contains layers of cobbly gravel, silt plus fine sand, and coarse to medium finely laminated gravel and coarse sand. Local coarse sand to medium gravel plus cobble layers have steeper dipping beds which appear to flatten southward. On the uppermost level, tan cobbly gravel with rare boulders under tan fine grained sand with coal exhibiting possible load casts is exposed above slumped material. This is overlain by medium yellow brown silt with little fine sand. This silt contains rare sub-angular cobbles. Feint layering is visible in the thickly bedded silt.

The deposit described above is the largest good exposure of unconsolidated sediments observed during this mapping program. Based on sedimentology (well sorted, rounded gravels in contact with well sorted sands or well sorted silts which appear to indicate rapidly changing hydraulic regimes; gravels with intersticial silt) sedimentary structure (steeply dipping bedding whose dip flattens southward or downstream) and geographical location (against the valley wall); these sediments are interpreted as a kame terrace deposit. No faults were observed cutting this layered sequence.

Kame terrace deposits were observed at the other DJU stations. Ice contact deformation was observed at several locations (refer to Subsection 2.5.1.2.3).

A yellow-brown silt with some fine sand, occasional to rare pebbles or cobbles was observed at several locations (DJU-1 at Elevation 665 feet; DJU-2 at Elevation 600 feet; DJU-3 at Elevation 580 feet; DJU-4 at Elevation 595 feet; possibly at DJU-7 at Elevation 640 feet overlain by cobbly gravel; and DF-15 at Elevation 1040 feet). These deposits have been interpreted as loess (Ref. 2.5-5) but may represent relatively quiet fluvial conditions. Well rounded cobbly gravels observed at DF-52 and DJU-5 may represent either valley train or kame terrace deposits.

2.5.1.2.2.2 Lithology and Stratigraphy at the Site

At the site, the thickness of the surficial materials occurring south of coordinate line N342,000, which includes all of the principal plant structures except the spray pond facilities, ranges from zero to about 40 feet. These materials consist of till and kame outwash, typically grading upward from a basal gravelly boulder zone to a surface layer of silty fine sand and sandy silt. The surface layer may represent reworked loess. Rock fragments in the gravelly outwash are well rounded and are composed mainly of hard, well-cemented, white to brown or red sandstones of various textures. No calcareous fragments were noted. In places, the sands and gravels contain minor amounts of anthracite grains and rounded anthracite pebbles up to a foot in diameter. These anthracite fragments cannot have been transported less than 3-1/2 miles from Shickshinny, the nearest occurrence of coal beds.

In the spray pond area in the northern part of the site, permeable, gravelly outwash and alluvial material fill an east-west bedrock valley to depths in excess of 100 feet. Cobble and boulder

pockets were encountered at various depths in most of the boreholes drilled in this locality. The deposit is glacial in origin, possible in part pre-glacial and overridden by ice, and reworked by water derived from ablation of the ice mass in the manner described by Peltier. It consists of sequences of sand, gravel and boulders, overlain by sand and gravel, overlain in turn by sand and silty sand. A geologic map of the surficial materials excavated in the spray pond area is presented on Figure 2.5-15.

Bedrock at the site is in the upper part of the Middle Devonian Mahantango Formation, except for a strip along the northern margin of the site. The uppermost member of the formation, which forms the top of rock in the east-west bedrock valley north of about N342,000, is a dark gray, noncalcareous siltstone in which bedding is generally delineated by thin, inconsistent, light-gray, fine-grained sandstone stringers. Upward and with increasing sand content the Mahantango Formation grades into the Trimmers Rock Formation, which occurs north of about N342,500 at the northern edge of the site. The Trimmers Rock, a gray fine-grained sandstone which caps the high, northeast-trending ridge north of the site, is massive to flaggy and exhibits well-developed joint systems.

Beneath its uppermost member, the Mahantango is comprised of 120 to 150 feet of hard, dark gray calcareous siltstone. It is harder and more resistant to erosion than the uppermost member, forming the east-west trending bedrock ridge just north of the reactor location and underlying the site to past the southern limit of the site area. The principal plant structures are founded on it.

These two upper members of the Mahantango Formation are similar in lithology and occur at the same stratigraphic position as the Harrell shale and underlying Tully limestone. However, the characteristic fossils of the Tully are not present in the site area which require these members to be assigned to the Mahantango Formation.

As exposed in the foundations, the unweathered bedrock is a dark gray, massive to thick bedded slaty siltstone, homogeneous in appearance and lacking the bedding plane fissility that is normally associated with less well indurated shaly rocks. The rock also exhibits a variably developed slaty cleavage or fracture cleavage, further indication of its indurated nature. Typically the rock is slightly calcareous and has intermittent fossiliferous zones which display impressions of brachipods, crinoids, corals, bryozoa and trilobites. Scattered veinlets and joint fillings consist of white, crystalline calcite or a mixture of calcite and quartz.

The rock weathers to a brown color, with iron oxide stains on joint and cleavage surfaces. Weathering progresses initially by dissolution of calcite from joint and fracture fillings, followed by more pervasive weathering of the rock mass and refilling of joints and veinlets with clay and other weathered material. Advanced weathering on exposed, natural surfaces evidently proceeded mainly along cleavage planes, so that on well weathered outcrops the platy cleavage fabric dominates greatly over jointing or bedding. Lack of significant weathering of the rock surface is often associated with areas where there is evidence of considerable glacial scour or fluvial erosion of the rock. Additional information on the engineering characteristics of the bedrock at the site is given in Subsection 2.5.1.2.5.

2.5.1.2.3 Structural Geology

2.5.1.2.3.1 Major Geologic Structures in Site Vicinity

The major structural features in the vicinity of the site are the Berwick Anticlinorium and the Lackawanna and Eastern Middle synclinoria which are discussed in Subsection 2.5.1.1.3. Structurally the site is situated slightly north of the axis of the Berwick Anticlinorium. The term "anticlinorium" as used herein is defined as a series of minor, intermittent anticlinal structures so arranged that they form a general arch or anticline.

Virtually all structural elements in the site area are related to Paleozoic crustal compression. These elements include kink bands which occur on all scales (Ref. 2.5-31 and 2.5-32) and most likely account for the Berwick and Lackawanna folds, contraction (reverse and bedding-plane) faults, and small scale flexural slip folds. These structures occur on all scales (e.g., Ref. 2.5-30 and 2.5-31). Where exposed it can generally be inferred that the small scale kink bands, contraction faults, and flexural slip folds are cogenetic, developed early in the tectonic history and were rotated by later, larger scale, genetically related folds. For example, at Station JW-3 (Figure 2.5-12) bedding strikes N70°-75°1E and dips 70°-75°NW. A reverse fault strikes N80°E, dips 70°NNW, and displays slickensides which rake 85° in the direction S80°W. The axis of the associated drag fold plunges 15° in the direction N85°E. The enveloping bedding on a small kink fold at this same exposure is oriented N70°E: 70°NW and the kink band is oriented N68°E: 60°SE. Similarly, at Stations DF-34 and DF-55 the geometric relations among cleavage, faulting and folding strongly suggest these structures are all coeval.

Like the folds, contraction faults also occur at different scales. At least some of the larger faults appear to have developed in response to a space problem created by the development of tightly appressed folds. An example of this is noted at Station JW-30, where the strain energy associated with a tight kink fold was released along one fairly large reverse fault (which parallels bedding on the hanging wall and cross cuts bedding on the foot wall) and several smaller faults which strike parallel to the larger one yet dip in the opposite direction. At JW-30 bedding, the kink fold and the associated faults all show nearly parallel trends; slickensides on the faults and bedding surfaces deformed by the kink band rake approximately 90°. Similar observations have been made elsewhere in the Fold and Thrust Belt (Valley and Ridge Province) and, as described by Faill and Nickelsen (Ref. 2.5-31), all of these structures are kinematically congruent, i.e., cogenetic.

Besides the aforementioned structures, local evidence of lateral movement was recognized along the north-south segment of the Susquehanna River at Stations JW-3 and JW-60. At JW-3, slickensides rake 20° in the direction S05°E on a surface striking N05°W and dipping 70°W. At JW-60, slickensides rake 20° in the direction S10°W on a surface striking N10°E and dipping 50°E.

This movement appears to be related to cross faulting in which case it, too, would be cogenetic with the other structures (Subsection 2.5.1.1.3). In any case lateral movement along this segment of the river was too small to produce any perceptible displacement on the map scale of 1:24,000.

As indicated in Subsection 2.5.1.1.3, all of these structural elements developed during the Late Paleozoic. No evidence was observed in outcrops within five miles of the site which would suggest that they have been active since that time.

Minor structural features were observed in Pleistocene sediment at a few locations in the site vicinity. Two small faults (exhibiting 18 inches and 2.5 inches of vertical separation) were observed at the margin of an apparent ice melt collapse feature in kame or kame terrace deposits at station DF-47. Asymmetric, reclined folds in unconsolidated sand and silt were observed at stations DJU-3, DJU-7, DF-7 and DF-47 (Figure 2.5-12). A small scale fault oriented N30-35E: 73-75SE with approximately 2 mm of dip slip separation occurs at DJU-9. This apparent reverse fault dies out upward. A coal bearing sand which lies about 6 cm above the observed displacement is not disturbed. Similar features at the site have been related to syndepostional slump, differential compaction and ice contact phenomena (Subsection 2.5.1.2.3.3).

2.5.1.2.3.2 Geologic Structures at the Site

During preconstruction exploration at the site, geologic structures in the bedrock at the site were defined and evaluated. Since bedrock exposures at the site were scarce (see Figure 2.5-17), most of this information was obtained from borehole cores, supplemented by geophysical logging of boreholes, seismic refraction surveys, cross-hole and down-hole measurements, test pits and trenches, and geologic mapping of the surface. Presented herein is a summary discussion of the geologic structures at the site as defined from the preconstruction exploration, followed by a description and discussion of the geologic structures that were observed in the excavations for the principal plant facilities.

The principal structural features in bedrock beneath the site are shown on Figure 2.5-18. The axis of a minor anticline crosses the site generally along the east-west base line (approximately N341,700). To the north of this base line, the strata dip to the north at between 20 degrees and 35 degrees. South of the base line, dips are to the south at between 5 degrees and 15 degrees. The predominant strike of the strata is N75E.

The prominent joint directions are parallel and perpendicular to the strike of the strata. The major joints strike parallel to bedding. This joint set is nearly perpendicular and dips opposite in direction to the dip of the bedding. A more open but less frequent series of vertical joints strikes parallel to the direction of dip of the strata. High angle joints healed by secondary calcite and quartz mineralization are present in the vicinity of minor shear zones.

The most prevalent type of rock displacements occurring in the region generally are low angle thrust faults. It has been indicated (Ref. 2.5-21) that many low angle thrusts shear upward through competent rocks utilizing incompetent strata as glide zones. Small shear planes that step stratigraphically from one shale-siltstone layer to another by shearing across intervening sandstone or conglomerate strata have been reported exposed in numerous road cuts and strip pits (Ref. 2.5-21).

Based on interpretation of initial data obtained for the PSAR from 100 and 200 series borings, particularly those located near coordinate line E2,442,400 (location of Section A-B, Figure 2.5-22), two areas of minor shearing were recognized at the site; namely, one in the vicinity of N341,200, slightly east of the reactor facilities and the other in the vicinity of N342,700. The

evidence of shearing is manifested by the presence of slickensides, calcite-healed gash fractures, and breccia zones.

The shears are of the low-angle type generally parallel to the bedding and are mechanically associated with the forces that acted to produce the folding of the strata.

The shear zone which occurs to the north of N342,600 is characterized by a series of bedding plane slips associated with breccia, slickensides, thin clay seams, and numerous fractures. The shear zone is contained within the less competent upper member of the Mahantango Formation and the lower portion of the Trimmers Rock Formation. The shear zone probably terminates at depth in the more competent calcareous member of the Mahantango.

No evidence of displacement was encountered in the main body of the more competent strata of the calcareous member of the Mahantango Formation between N341,950 and N342,600. The stresses that acted on these strata were taken up by the development of joints and fracture cleavage. Detailed inspection of the rock cores extracted from this area reveals microshear offsets along the cleavage planes. The net effect of this mechanism is to thicken the strata as revealed by the stacking and shortening of sandy stringers. The cleavage planes are generally healed by secondary lithification of the rock matrix.

The contact between the top of the Mahantango Formation and the base of the Trimmers Rock Formation was encountered in borings 117, 108, 122 and 126, all located north of N342,550. Detailed examination of bedding planes observed in the rock cores from these borings indicated that the dip of the strata increases with increasing depth. This is confirmed by borehole geophysical data. The numerous breccia, slickensides, thin clay seams and fractures encountered in borings 122 and 126, and to a lesser extent in boring 108, represent a zone of en echelon shear planes, both parallel and subparallel to the bedding. These shears are related to the original tectonic stresses which produced the regional folding.

A petrographic examination of the clay and rock encountered in some of these borings in the northern part of the site was conducted by Dr. Charles Thornton of Pennsylvania State University. The examination indicated that the rock and clay in the broken zones were mineralogically similar to the intact rock obtained from the core above and below the broken zones. Since no secondary mineralization was encountered in association with the clay and broken rock, it appears that this condition was mechanically induced and is not a result of chemical alteration and/or weathering.

In the second area of minor shearing identified above, evidence of structural adjustment which may be called a shear zone is present as slickensides and healed breccia at various depths in borings 125, 127, 132 and 103 as indicated on the subsurface section (Figures 2.5-21A and 2.5-21B), and in borings 100, 217 and to a minor extent in 105 perpendicular to the section. These borings are in the area adjacent to and immediately east of the reactor facilities. Based on this evidence, this zone of structural adjustment strikes east-northeast and dips southerly at approximately 10 degrees. If this zone of structural adjustment extended northward beyond boring 102, it has been subsequently removed by erosion.

Detailed inspection of the microstructure in the rock core extracted from the borings at the site reveal shear-fold structural relationships similar to those encountered on a larger scale across the site. The displacements observed in the rock core are completely healed by secondary calcite and quartz mineralization.

It is probable that the bedrock at the site served as an intervening buffer or adjustment zone during the regional folding of the strata.

Stresses that formed the Berwick Anticlinorium and synclinal structures appear to have been absorbed within the rocks of Mahantango and underlying Marcellus formations, as flexural slip, disharmonic folding and glide thrusting. The stresses that were necessary to produce these structural features were compressional from the southeast. These structural features were formed no later than the close of the Paleozoic Era, approximately 200 million years ago. Based on thorough consideration of all the information provided by the pre-construction foundation exploration, it was concluded that the minor structural conditions observed at the site are not of significance with respect to siting or design for the use of the site for its intended purpose. An evaluation of subsequent data assembled from additional boring exploration and from geologic mapping of the foundations, confirms the initial conclusion.

During excavation and clean-up of the rock at Unit 1 reactor and turbine foundations, at the circulating water pumphouse, and along the trench for the hot water intake pipeline to Unit 1 cooling tower, a bedding plane shear showing strong slickensides was uncovered. This bedding plane shear is the same shear plane that was identified in the early phases of the site exploration and is herein referred to as "bedding plane shear A" (refer to Figures 2.5-18 and 2.5-19).

In the northeast corner of the Unit 1 reactor foundation, bedding plane shear "A" strikes N85°E and dips 7°SE. The surfaces of the bedding plane contain 1/4-inch to 3/4-inch thick laminae of calcite, siltstone and some quartz. The calcite laminae are approximately 1/16-inch thick, alternating with thinner siltstone laminae. The entire exposed area of this bedding plane contains prominent slickensides trending N30° to 40°W, with a 6° to 7°SE plunge. Up-dip and closer to top of rock, the bedding plane contains a 1/2 to 1-inch wide, iron-stained zone, and it also shows extensive leaching of the minerals filling the shear.

In places, the adjacent rock is weathered to a granular sandy soil. The calcite which fills the bedding plane shows no sign of crushing. The weathering and staining on the bedding plane shear occurs only near top of rock where surface water and groundwater could penetrate along the plane; at foundation grade which is well below the weathered zone, the unweathered laminae have the properties of firm rock. In places the bedding plane shear is apparently not a prominent feature in the unweathered rock. For example, it was identified only as a slickenside surface with associated jointing in boring 105 and as horizontal jointing planes in boring 351 (geologic section E-E', on Figure 2.5-19).

A second essentially parallel bedding plane shear striking N75°E and dipping 7°SE was exposed in the trench for the circulation pipe, at the intersection of column lines 19 and G. Slickensides trending N30°W with a 7°SE plunge are also exposed on this plane. The surface is coated with a 1/8 to 1/4-inch-thick layer of unweathered calcite. This shear plane is designated "bedding plane shear B" on Figure 2.5-18. Although similar in appearance to bedding plane shear A at this location, apparently this shear is more restricted in a real extent, because it was not recorded on the logs of nearby bore holes nor was it mapped in the radwaste foundation area where it should have been exposed if it had continued that far north.

It proved possible to collect intact samples from the sheared portion of bedding plane shear "A" for more detailed analysis, including petrographic thin sectioning. The mineralization along the bedding plane consists of thin, parallel bands of intergrown calcite and quartz. The bands, 0.5

to 5.0 mm wide, are separated by thin films of dark shaly material on which slickensided striations caused by shearing have formed. Within the bands, the majority of quartz grains shows recrystallization into interlocking, strain-free grains up to 5 mm long, but becoming cryptocrystalline in the thinner bands. These relationships suggest that the quartz-calcite mineralization was not a late, post-tectonic occurrence, but rather was probably introduced in association with shearing, which is known to have taken place at the end of the Paleozoic (refer to discussion at the end of this Subsection 2.5.1.2.3.2). Undeformed microscopic veinlets of calcite can be observed to cut across the bands at nearly right angles. These veinlets are not themselves offset, and therefore constitute mineralization that has not been crushed or deformed since its deposition. Similar instances of undeformed calcite veinlets crossing slickensided bedding planes are observed on a megascopic scale in the site excavation. Figures 2.5-20a through 2.5-20g illustrate such occurrences.

Bedding plane shear "A" was mapped in the excavations westward from the northeast corner of the Unit 1 reactor foundation to the west slope of the circulating water pumphouse excavation (Figure 2.5-18). It was also exposed in the trench for the Unit 1 cooling tower hot water intake piping and in two pedestal (No. 6 and No. 7) excavations for the tower itself. Although it displays minor undulations, the average strike of the bedding plane shear is close to N85°E eastward from the turbine and reactor foundations, approximately parallel to the axis of the minor anticline at the site and to the regional structural trend. Near the Unit 1 cooling tower, the bedding plane shear strikes about N70°E, consistent with measured bedding attitudes in that area. Representative dip measurements on the shear plane in the foundations were between 5° and 8°S, which is parallel to the dip of bedding. The trend of the slickenside lineation on this bedding plane shear across the foundation area ranges between S10°E and S40°E, most between S20°E and S30°E, a direction consistent with regional north-northwest compression during folding.

Drill hole data were utilized to project bedding plane shear "A" down-dip. Geologic sections E-E' and F-F' on Figure 2.5-19 show profiles of the shear through the reactor and turbine foundations. The source of data for these profiles is from foundation geologic mapping and elevation surveys, supplemented by subsurface data from the boring logs. It is evident that the foundation mapping and boring log data are in very good agreement, and that the minor shear zone originally identified in this area from exploratory borings is identical to the bedding plane shear "A" identified during construction (see Figures 2.5-21A and 2.5-21B, which was prepared before excavation for the plant structures began). Although this figure suggests that bedding plane shear "A" may not be completely parallel to bedding, no evidence was found during later exploration and excavation to indicate that the shear plane transects bedding.

Bedding plane shear "A" can be traced updip along the Unit 1 hot water intake pipeline trench to the excavation for Unit 1 cooling tower pedestals 6 and 7, where the shear plane crosses the axis of the minor anticline that trends through the site. At pedestal 6 which was excavated to Elevation 667 feet, the weathered bedding plane shear was exposed and dips gently south, conformable to bedding (Figure 2.5-18). At the adjacent pedestal 7 which was excavated to elevation 668 feet, the same weathered bedding plane shear was again exposed, but here it dips gently north, again conformable to bedding. At these locations the weathered shear is two to three inches thick. Where unweathered, the shear is tightly healed with calcite and quartz mineralization; where weathered, these minerals have been partially removed and replaced with claylike material. A roller-bit probe made during the Unit 1 cooling tower foundation exploration recorded a thin seam of soft rock in the vicinity of pedestals 8 and 9 at about elevation 662 feet,

which was probably a penetration of bedding plane shear "A", and, together with measured bedding attitudes, reveals a continuation of the northward dip of the shear plane. West and south of the circulating water pumphouse, undulations in the bedding are evidenced by local northward dips of 5 to 10 degrees. Elevations at which shears were intersected by boreholes 318, 321 and B-5 suggest that bedding plane shear "A" closely parallels the undulations of the strata in this area. These structural relationships are shown in profile in geologic section G-G' on Figure 2.5-19. The fact that the shear plane is folded in conformance to local structure demonstrates that the shear plane originated before or during the time of folding and effectively dates its formation at 200 million years ago or earlier, which is the minimum age of Appalachian deformation in the region (Refer to Subsection 2.5.1.1.3 and the discussion at the end of this Subsection 2.5.1.2.3.2).

Other slickensides were recorded on many joint planes at the site, particularly on low-angle joint planes. Most of these slickensides plunge southeast. The geologic map (Figure 2.5-18) shows these measurements. Numerous slickensided joint planes had been recorded in bedrock cores in the early stages of the site exploration see boring logs, holes 100-132 and 210-219, Figures 2.5-23a through 2.5-23t); they were also observed in rock removed during foundation excavation. Many of these low-angle slickensided joint planes are calcite-coated, and some are undulatory in form rather than planar. They were noted in some instances to splay out from the more prominent bedding-plane shears described above. Evidently, differential movement which occurred principally along bedding planes was transmitted laterally to the encompassing bedrock mass along these bifurcating slickensided joints or shear planes. Such slickensides and shears should be expected in view of the tectonic history and the nature of deformation which the region has undergone.

Significantly, regardless of the orientation of the planes on which slickensides occur (whether they dip north or south), the trend of the slickenside lineation is almost invariably in the northwest-southeast quadrant, clustering N20-35°W (or S20-35°E). This direction is completely consistent with the northwesterly-directed tectonic compressive stress that produced the regional folding and thrust faulting during the Appalachian orogeny, and is further evidence that the slickensides that occur at the site are geologically old; that is, they originated over 200 million years ago. Their consistent orientation suggests deformation during a single tectonic episode, rather than recurrent deformation at different times in geologic history.

Bedding plane shear "A" intersects the top of bedrock surface in the diesel generator and Unit 1 turbine and reactor area. During excavation, two exposures of this intersection were examined to determine the nature of this contact (exposures at intersections of grid line N341,400 with column line G and with column line N (Figure 2.5-18), and photographs (Figures 2.5-20b through 2.5-20e) were taken. Glacial deposits overlay the rock at these points. In each case the eroded rock surface was continuous across the trace of the bedding plane without displacement or offset. If displacement had occurred subsequent to erosion of the rock surface, this would be apparent as an angular, sharp projection of rock into the overlying glacial deposits; instead, the rock surface across the trace of the bedding plane is smoothed by erosion. Figures 2.5-20b through 2.5-20d show this relationship. In the area north of this intersection, the bedding plane shear had been eroded away, thus confirming the original evaluation based on exploratory borings (compare geologic section E-E', Figure 2.5-19 with geologic section B-C, Figures 2.5-21A and 2.5-21B). The erosion of the rock surface would necessarily have occurred prior to the deposition of the overlying glacial deposits, which have been established as being more than 50,000 years old (refer to Subsection 2.5.1.2.2.1). Consequently, this relationship shows that any displacement along bedding plane shear "A"

occurred more than 50,000 years ago. Actually, regional relationships plus the fact the plane is folded indicate that any displacements are a result of the tectonic forces which occurred prior to the late Triassic, over 200 million years ago.

Thus, the original preconstruction appraisal of shears which occur at the site as reported in the PSAR remains the same. These minor shears and structural conditions are consistent with the mode of deformation which occurred during the Appalachian orogeny, over 200 million years ago. They are not significant to the plant site or to the operation of the plant.

Cleavage. Secondary cleavage is variably as developed in the rock exposed at the site; in some places, such in the slopes of the ESSW pipe trench north of the circulation water pumphouse and in parts of the cooling tower areas, it forms the dominant structural feature of the rock, both on fresh and on weathered exposures. The strike of the cleavage is oriented east-northeast, approximately parallel to the trend of the major fold axes, and dips with variable steepness to the south, but generally in the range of 40-80°. Where the dip of the cleavage locally becomes fairly shallow, such as along the eastern perimeter of the south cooling tower, it is sometimes difficult to distinguish cleavage planes from bedding planes. The fact that the cleavage is oblique to bedding demonstrates its secondary origin, apparently during the episode of regional tectonic deformation, 200 million or more years ago.

Joints and Fractures. Jointing in the rock excavated for foundations is fairly well developed. Figure 2.5-18 maps the principal joints encountered at foundation grade, which is at a sufficient depth below top of rock to be in essentially unweathered material. Here joints are tight and either uncoated or coated with calcite or a mixture of quartz and calcite. Relatively few joints at foundation level contained significant iron staining; some iron-stained joints are mapped in the radwaste foundation area. Toward the surface these joints generally become more heavily iron-stained with greater degree of weathering, and calcite coatings tend to be leached out, resulting in open joints, in joints partly coated with quartz or in clay-filled joints in the zone of weathering.

The most abundant joint set in the principal foundations area (Figure 2.5-18) strikes east-northeast (N60°-85°E), roughly parallel to the major regional fold axes and to the secondary fold axis at the site. North of the anticlinal axis at N341,300, these joints strike N70-85°E and dip, with some scatter about the vertical, 75°S-75°N, most 85°S- 85°N. South of N341,100 similar but more numerous joints, shown diagrammatically on Figure 2.5-18, strike N50°-60°E, dip uniformly 50°-60°SE, and appear to comprise a distinguishable set. Less numerous but quite prominent joints with a similar east to east-northeast trend dip gently northward at 10°-18° and are best represented along the vicinity of coordinate line N341,200.

Other dominant joint sets are steeply dipping to vertical north-northwest to northwest joints, and north-south joints. Dips in both sets are usually greater than 70° with both east and west dips represented although the majority of those measured dip toward the west.

Many joints are filled with white calcite or a mixture of calcite and quartz, but there appears to be no preferential orientation for these filled joints. The low-angle joints are commonly slickensided (discussed above). In the turbine building excavation, two vertical, calcite-filled joints cut across bedding plane "B". The calcite in these vertical joints is continuous across the bedding plane with no offset, showing that the joints were formed and the calcite was deposited

in the joints subsequent to the development of the slickensides on the bedding plane. Photographs were taken of this exposure (Figures 2.5-20e through 2.5-20f).

In addition to these principal joints, high-angle, discontinuous, white calcite and quartz-calcite veinlets are typically exposed locally throughout principal plant foundations. These veinlets probably represent fractures that originated during Late Paleozoic tectonic deformation. They tend to occur most abundantly in the vicinity of bedding plane shears (discussed below) and as such may have arisen as gash fractures, as for example the veinlets mapped in the vicinity of N341,350-E2, 441,550 (Figure 2.5-18). At this same location is a singular occurrence of numerous west-dipping open vugs and seams up to several inches wide containing undeformed, euhedral quartz crystals up to 2 inches long. These seams were here exposed several feet above a bedding plane shear (see description above). Chunks of loose, coarsely crystalline white calcite also occur in the vugs. It is evident that these vugs had originally been relatively wide (up to 5 inches) gash fractures containing a coarsely crystalline quartz-calcite mineral filling; later the rock weathered and the calcite was selectively dissolved by circulating ground water (Refer to Subsection 2.5.1.2.5.1).

Bedrock Configuration at the Site. Figure 2.5-17, a map showing top of rock contours at the site, illustrates the general original configuration of the bedrock surface. It is evident that the major erosional feature of this surface is a buried, east-west bedrock valley in the northern part of the site, including the spray pond location. Here glacial or pre-glacial erosion has incised the bedrock surface approximately 100 feet below the general top of rock elevations to the south. In detail, the bedrock surface is very irregular due to the action of glacial plucking and subsequent glacio-fluvial erosion. The large pothole over 30 feet deep and 30 feet wide was found in the Unit 1 turbine building excavation; other smaller ones also occur at the site. Additional discussion of erosional features in bedrock at the site is presented in Subsections 2.5.1.2.1 and 2.5.1.2.3.3.

Relation of Site Geologic Structure to Regional Structure. Geologic mapping at the foundation excavations for the plant structures, together with subsurface borehole data, shows that bedding plane shear "A", the only shear plane traceable across a significant part of the foundations area is, within the accuracy of the data, parallel to bedding and follows the folds which the bedding defines, indicating that the bedding plane shear was either formed prior to folding, or, more likely, developed in conjunction with folding (refer to geologic section G-G' on Figure 2.5-19). Therefore, knowledge of the age of folding would provide a minimum date of origin of the bedding plane shears exposed at the site. With that objective in mind, the literature was examined first to determine whether or not the structures of the site are consistent with the regional structure, and second to date as accurately as possible the age of deformation.

The attitude of the sheared bedding planes and the trend of the slickensides on the planes may be compared to the nearest major tectonic structure (The Berwick Anticlinorium) to the site that is an obvious and consistent member of the pattern of regional deformation in the Valley and Ridge province. The strike of the sheared bedding planes (N75°-85°E) are essentially parallel to the axis of the Berwick Anticlinorium (N75°-80°E) immediately south with compressive forces from the southeast which caused the folding in the region, and of the Berwick Anticlinorium in particular. The Berwick Anticlinorium is one of a series of folds in the Pennsylvania Valley and Ridge Province. It is located in the northwestern part of the province near the Allegheny plateau. Rocks involved in this deformation within the Valley and Ridge province range as recent as Permian in age, and the intensity of deformation increases toward the southeast -- from broad,

gentle open folding at the Allegheny front to overturned, recumbent folds and nappes complicated by thrust faulting at the Blue Ridge.

Arndt and Wood (Ref. 2.5-64) have classified this progressive deformation resulting from compressive stresses originating to the southeast into a number of stages, each stage being categorized by effects of successively more intense deformation. Thus, the effects of deformation were transmitted with time northwestward over an increasingly greater distance, and deformation acted at any one locality with increasingly greater intensity with time. It follows that "the areas of most complex structures to the southeast underwent each of the first four stages of deformation, whereas the least intensively deformed area to the northwest was subjected only to the last orogenic force and contains features characteristic of only the first stage of deformation" (Ref. 2.5-64).

The first stage of deformation is characterized by horizontal strata cast into broad, open folds without significant thrust faulting. The second stage, which characterizes the area in which the Berwick Anticlinorium is located, exhibits low-angle thrusting and imbricate faulting followed by formation of subsidiary folds on the larger folds to develop anticlinoria and synclinoria. Structures in the vicinity of the site are consistent with this categorization. Subsidiary flexures at the site are broad, open features (refer to geologic section G-G', Figure 2.5-19), and low-angle thrust faulting is represented by the decollement in the site vicinity as discussed in Subsection 2.5.1.1.3. Subsequent stages, in which the folds are overturned and then additionally folded and faulted, are absent from the Berwick anticline area. Arndt and Wood (Ref. 2.5-64, p. B134) state, "the process of structural evolution appears to have been continuous and the result of a single orogeny that was not necessarily punctuated by pulsations.... The orogeny began after rocks of Pennsylvanian age were consolidated and prior to deposition of rocks of Late Triassic age." It is obvious from this model of deformation that thrust faulting was a logical and integral accompaniment to folding, rather than being part of some separate tectonic episode subsequent to folding.

In this process of deformation, "rocks of the more competent units characteristically folded into generally concentric, symmetric to asymmetric anticlines and synclines broken variably by faults. The rocks of the less competent units developed disharmonic folds broken by decollements, low angle thrust and bedding faults, and commonly separate discordant folds in the more competent rocks" (Ref. 2.5-21, p. 160). As a result, it is expected that rocks least able to withstand great shear pressures, such as shales, would display evidence not only of large magnitude differential movement as is found near the major thrust zones, but also of lesser but more prevalent minor structural adjustments, such as shears, incipient bedding plane faults, zones of closely spaced joints or fractures, slickensides, slaty cleavage, and so on. Thus, it would be surprising if the Mahantango Formation which occurs at the site did not show at least some of these features produced during Appalachian deformation.

"Northwestward-directed stresses of the late Paleozoic Appalachian orogeny were largely responsible for the development of the tectonic framework of the Anthracite region and the remainder of the Valley and Ridge province in Pennsylvania" (Ref. 2.5-21). There is general agreement that the time of the Valley and Ridge deformation, which is equated with the "Appalachian Revolution" (Ref. 2.5-28, p. 645), also termed the "Alleghany" or "Allegheny orogeny" (Ref. 2.5-65), ended before late Triassic time over 200 million years ago, but there is surprisingly little evidence to indicate a more exact dating of the events. As Rodgers (Ref. 2.5-34, p. 34) states, "traditionally...the deformation has been dated at the end of the Paleozoic, and in fact for generations American students were taught that it was the event that marked the end

of the era." The youngest known deformed strata are lower Permian in age (in the Georges Creek syncline just west of the province boundary in Maryland) (Ref. 2.5-34, p. 64). Therefore, typical Valley and Ridge folding and faulting occurred in the Permian and perhaps continued into the early Triassic; apparently it formed most if not all the major structural features of the province (Ref. 2.5-34, p. 64).

According to Woodward (Ref. 2.5-65, p. 2320), "there is no tangible evidence regarding the time of this deformation save that part of it must have occurred after the Pennsylvanian (or after the early Permian) and all of it before the late Triassic...Nothing fixes its appearance specifically at the end of the Permian; even its latest movements could have ceased by Middle Permian. They could also have continued through the Middle Triassic for any evidence to the contrary." The Upper Triassic shale and red bed deposits in their tilted and downfaulted basins provide an upper age limit for the Allegheny orogeny (Ref. 2.5-34, p. 115) because it is thought that the pervasive northwest-southeast compressive force field required for the northwest-directed thrust faulting and folding during the Allegheny orogeny could not have been present during the formation of the Upper Triassic basins, which required essentially extensional or tensional stress acting in the east-west or northwest-southeast direction.

In several places undeformed Triassic features are directly superimposed on Valley and Ridge structures, establishing an upper limit for Valley and Ridge deformation, of which the Berwick anticline is a part. Between the Schuylkill and Susquehanna Rivers, Triassic basin sediments rest directly on and truncate the recumbent folds and nappes (Ref. 2.5-66) developed in the southeast part of the Valley and Ridge province. These upper Triassic sediments were deposited on a peneplained surface; thus, the Valley and Ridge structures had become inactive and were exposed and eroded to near base level before upper Triassic time over 200 million years ago. Late Triassic diabase dikes are shown on the Tectonic Map of the United States (Ref. 2.5-67) crossing Appalachian fold structures about 20 miles northwest of Harrisburg near the mouth of the Juniata River. Since these dikes are neither deformed nor offset by Valley and Ridge faults, they also establish a pre-late Triassic age for Valley and Ridge tectonism.

According to Dr. Gordon H. Wood of the U. S. Geological Survey (verbal communication, 1974) there are no local specific field relationships in the Anthracite basin which could be used to supply a definite date for faulting and folding in the Anthracite basin. The only known date for Appalachian structures is supplied by regional relationships such as the Triassic events. However, Dr. Wood stated that all faulting related to Appalachian structures, except possibly for some very minor Triassic faulting, is Paleozoic in age.

On the basis of the foregoing discussion, it is concluded that thrust faulting, shearing, bedding plane faults and other similar features in the area near the site, and the slickensides and striations in the foundation rock underlying the site, were formed during the "Allegheny orogeny" or "Appalachian Revolution" which produced the folds and thrusts of the Valley and Ridge province, of which the Berwick Anticline is a part; thus, these events became tectonically inactive before upper Triassic time or over 200 million years ago. The slickensides and shearing which are evident on various bedding planes and joint planes in the foundation rock at the Susquehanna Site are therefore of no significance to the plant structures.

2.5.1.2.3.3 Geologic Features in Surficial Materials at the Site

Surficial material in the site vicinity consists of glacial drift deposited near the Olean terminal moraine (refer to Subsection 2.5.1.2.2.1). The glacial deposits near the Susquehanna site have been studied in some detail by Peltier (Ref. 2.5-5, p. 25). He describes the various features and processes associated with the terminal moraine near Beach Haven (3 miles southwest of the site) as follows:

> (The Moraine) is a gravel moraine and is composed largely of poorly sorted, coarse kame gravel, medium-grained valley train gravel, and sand. These gravels were deposited in marginal channels between a stagnant tongue of ice, which lay in the center of the valley, and the valley walls... During the early stages of kame terrace development, the marginal channels flowed at a level which was high above the valley, and, at the front of the ice, fell sharply to the valley floor. At the ice front a steep alluvial deposit, composed largely of coarse gravel, was formed. Continued ablation of the ice in the valley probably caused the marginal streams to flow at successively lower leves. These streams, where they flowed along the ice, both eroded the earlier deposits and filled in their channels; where they crossed the "terminal moraine" they cut channels in the previously deposited alluvium and laid down sand and gravel on more gently sloping gradients toward the river valley beyond it. In this manner any till deposited at the ice front became buried or eroded.

At the site, little till was exposed in the excavations for the principal plant structures, in conformance with Peltier's nearby observations. Essentially all of the glacial material excavated consist of stratified drift in the form of kame delta and terrace deposits, alluvial outwash and stream gravels, much of it probably reworked in the manner described by Peltier. Indeed, the scoured and fluted bedrock surface, large potholes, and steep and even undercut contacts between bedrock and glacial drift attest to the torrential flow of water which at one time evidently cascaded across the site; and the coarse boulder gravels and erosion channels within the outwash indicate energetic reworking of the materials. In keeping with this glacio-fluvial mode of deposition, contemporaneous sedimentary features, such as those resulting from slumps at undercut or eversteepened stream banks, from differential compaction of materials deposited on irregular surfaces, and from other adjustments during deposition, may be expected (see for example (Ref. 2.5-68, p. 184-185). A few minor features such as sedimentary creep or small slumps were observed in the stratified drift, as for example north of the radwaste building, where they are associated with an undulating, fluted rock surface. Apparently these features, which terminate above the rock surface, arose through differential compaction across the irregular bedrock surface.

None of the sedimentary features exposed in the glacial materials were observed to extend downward to intersect the bedrock surface. It is concluded that all such features observed in the surficial materials at the site are consistent with their known mode of origin by glacio-fluvial process that occurred near the terminal moraine of the Olean glaciation.

2.5.1.2.3.4 Hazards from Storage in Local Geologic Structures

The unconsolidated Quaternary deposits in the vicinity of the Susquehanna SES are unsuitable for storage or disposal. While storage in unconsolidated strata is feasible, the thickness and extent of the Quaternary strata in the site area is insufficient. For example, with regard to aquifer storage of natural gas, the minimum depth of overburden necessary to maintain

adequate deliverability at the well head is about 500 ft., while depths in excess of 1500 ft. are desirable for an efficient operation.

As discussed in Subsection 2.4.13, none of the bedrock formations in the site vicinity have a high primary transmissivity. Both the primary porosity and permeability of these well consolidated rocks are generally low. Ground water utilization is dependent upon secondary permeability developed through tectonic fracturing and jointing or solution processes. Thus, while the anticlinal structure in the site vicinity may provide geometry suitable for aquiter storage or disposal, no suitable reservoir strata are known to be present.

Deep well injection into fracture porosity zones in impermeable rock might be considered as a potentially feasible method of waste disposal in the site vicinity. However, based on existing literature and considering current technology, this method of disposal is the least desirable. Reservoir strata with some degree of primary permeability are preferred (Ref. 2.5-116).

It is believed that the Precambrian basement, at depths in excess of 30,000 feet in the vicinity of the Susquehanna SES, does not contain the Fold and Thrust Belt Fracture System (Subsection 2.5.1.1.3). The nature and extent of any fracturing in these rocks is unknown. Recent advances in drilling technology suggest that the technical capability to construct a disposal well at depths in excess of 30,000 feet may be available in the near future. In the U.S. there has been at least one instance of disposal of chemical waste into Precambrian age crystalline rock (Ref. 2.5-117). However, rocks of this type with transmissibilities dependent solely on fracture porosity are not generally considered to be suitable storage or disposal reservoirs (Refs. 2.5-118, 2.5-119 and 2.5-120).

A discussion of the potential hazard resulting from a subsurface storage facility would be dependent upon the type of facility and the type of material being stored. In view of the low potential for the development of such a facility in the near vicinity of the Susquehanna SES, a discussion of potential hazard is unwarranted.

2.5.I.2.4 Site Geologic History

The geologic history of this region can be traced from Precambrian times. Rocks beneath the Paleozoic strata at the site form the Grenvillian cratonic basement, approximately 1 billion years old. The sediments that were deposited to form the Precambrian rocks in the region were subjected to magmatic intrusion, metamorphism and erosion before the onset of Cambrian time.

Paleozoic sedimentary strata in the site vicinity are estimated to be on the order of 30,000 feet thick (Ref. 2.5-28 and 2.5-38). The deposition and deformation of these strata is related to the opening and closing of the Proto-Atlantic Ocean (Ref. 2.5-69).

Although deformation in the Appalachian Orogen culminated three times in the Paleozoic -- the Taconic, the Acadian, and the Alleghenian (Appalachian) orogenies -- the effects of the first two orogenies in the folded Appalachians in which the site occurs were mainly sedimentologic rather than structural, being evidenced as unconformities and as changes in provenance, lithology and in sedimentation characteristics, in contrast to the intense folding, faulting, volcanism and metamorphism which occurred at these times on the Mobile Belt during the Taconic and Acadian events. The Alleghenian orogeny, on the other hand, resulted in the structural

configuration at the site today. The structural evolution of the Fold and Thrust belt is described in Subsection 2.5.1.1.3.

Crustal divergence in Late Precambrian, Cambrian and Early Ordovician time allowed the accumulation of a thick sequence of miogeosynclinal sediment in the Appalachian Basin (Subsection 2.5.1.1.2). The Taconic Orogeny beginning in Middle Ordovician time signifies convergence and uplift in the Mobile Belt.

The highly deformed early Paleozoic strata are unconformably overlain by less deformed, coarser grained clastic sediment which is in turn overlain by the Siluro-Devonian carbonate sequence. This sequence is thickest in the east and thins westward.

Northeastward from the site the carbonate strata interfinger with clastic detritus. Late Paleozoic strata are clastic through most of the Appalachian Basin (Subsection 2.5.1.1.2).

These strata reflect the closing of the Proto-Atlantic Ocean. At the peak of the Taconic Orogeny along the cratonic margin to the east, ophiolitic rocks (presumably oceanic crust) were obducted from the eugeosyncline, and the miogeosynclinal strata (carbonate and detrital alike) were thrust onto the craton. The geologic setting at the site is the result of this activity. Folding and thrust faulting occurred through mechanical detachment from rigid basement rocks along decollements in shaly strata near the base of the Paleozoic section (Ref. 2.5-38). The site rests on the northern limb of one such fold, the Berwick Anticlinorium. The deformation progressively increased in intensity toward the southeast, from broad, gentle open folding northwest of the Allegheny front to overturned, recumbent folds and nappes complicated by thrust faulting at the Blue Ridge. The effects of final convergence and translation during the Late Paleozoic appear to be limited to the Mobile Belt (Subsection 2.5.1.1.3 and 2.5.2.2).

The Appalachians appear to have undergone erosion through most of the Mesozoic Era. Tectonic activity related to the opening of the Atlantic Ocean appears to have had no significant structural effect in the Fold and Thrust Belt and Stable Interior (Subsections 2.5.1.1.3 and 2.5.2.2) until the Cretaceous Period. At that time, subsidence of the Atlantic continental margin allowed transgression of the sea well inland of the site vicinity.

During Cenozoic uplift, major drainage in the area followed relatively straight southeastward courses through the Cretaceous sedimentary strata to the Atlantic. The Ancient Little Schuylkill River flowed past the site toward the present day Delaware Bay. The ancient north branch of the Susquehanna River flowed through Wilkes-Barre, Pennsylvania toward Trenton, New Jersey. As the Appalachians were exhumed, the east-northeast structural fabric began to exhibit control of the drainage pattern. The present course of the north branch of the Susquehanna River resulted from stream capture of the Ancient Little Schuylkill and ancient north branch through their east-northeast tributaries by the main branch of the Susquehanna River (Ref. 2.5-3).

Northeastern Pennsylvania has undergone at least three glaciations during the last 150,000 years and possibly one or more prior to that date. Till at the site was deposited during the Olean substage about 55,000 to 60,000 years ago (Ref. 2.5-5 and 2.5-6). Older Illinoisan drift occurs in the valley of the Susquehanna River between the Olean terminal moraine at Beach Haven (about 3 miles southwest of the site) and the confluence of the north and west branches of the Susquehanna River. Post-Olean advances did not reach the site vicinity (Ref. 2.5-5 and 2.5-6).

Peltier (Ref. 2.5-5) mapped discontinuous kame terraces along the Susquehanna River in the site vicinity. The highest such terrace formed by ice marginal streams occurs at about 650 feet above sea level at the site. Refer to Subsections 2.5.1.2.2 and 2.5.1.2.3.3 for further discussion of Pleistocene erosion and deposition at the site.

Since the retreat of the Wisconsinan ice sheets from the region, broad regional uplift appears to have occurred, probably at least in part as a result of crustal rebound subsequent to the removal of ice load. Erosion has continued and soil profiles have formed.

2.5.I.2.5 Engineering Geology Evaluation

Site subsurface exploration is described and discussed in Subsection 2.5.4.3.

Laboratory tests of foundation materials, and in situ geophysical tests of the foundation materials are discussed in Subsections 2.5.4.2 and 2.5.5.

Geologic mapping of the final foundations is described in Subsections 2.5.1.2.2,2.5.1. and 2.5.4.1.3. It was concluded from these studies and evaluations that the site geologic and foundation conditions are entirely suitable for the construction and operation of the plant.

2.5.1.2.5.1 Geologic Conditions Under Category 1 Structures

All Seismic Category 1 plant facilities, except the spray pond, Engineered Safeguard Service Water (ESSW) pumphouse and pipeline, and the diesel generator 'E' fuel tank are founded on bedrock. The ESSW pipeline trench is excavated partly in soil and partly in rock. The location of these facilities is shown on Figure 2.5-24.

The foundation rock is a hard, indurated siltstone, a member of the Devonian Mahantango Formation. In the foundations area it is quite massive and lithologically homogeneous, with bedding generally not well defined, and lacking the bedding plane fissility usually associated with less well indurated shaly siltstones and silty shales. In places the rock exhibits a slaty cleavage, further evidence of its indurated nature. All Category 1 rock foundations were excavated to unweathered bedrock. Geologic maps and sections of the Category 1 excavations in rock are shown in Figures 2.5-18 and 2.5-19. More detailed discussion of the foundation geologic conditions is contained in Subsections 2.5.1.2.2 and 2.5.1.2.3. Engineering properties of the foundation rock are described in Subsection 2.5.4.

The spray pond is situated over a glacial or preglacial, east-west trending bedrock valley as outlined by contours on top of bedrock (Figure 2.5-17). The valley is filled with dense gravelly and sandy glacial outwash and till deposits which attain a maximum thickness of about 110 feet adjacent to the spray pond area. They were deposited no later than the Olean substage (early Wisconsinan) of the Wisconsinan glaciation which occurred over 50,000 years ago. In general, the deposits are permeable and consist of a sequence of sand, gravel, and boulders overlain by sand and gravel, overlain in turn by silty sand. The entire sequence is highly variable in grain size distribution and sorting, and contains discontinuous pockets of similar materials. As a rule, grain size decreases and sorting increases toward the top of the sequence.

The southwestern tip of the spray pond is cut into bedrock while the remainder was excavated in these permeable glacial materials. The thickness of the glacial deposits beneath the bottom of the spray pond ranges from zero at the rock contact to 93 feet at the eastern end of the pond. The spray pond is lined to minimize seepage losses to the underlying permeable glacial deposits. The foundation of the pumphouse structure located at the southeastern corner of the pond is underlain by 35 to 60 feet of glacial material. The ESSW circulation pipelines between the pumphouse and the plant intersect bedrock at an elevation of 668 feet, approximately 260 feet southeast of the pumphouse (refer to Figure 2.5-17A). A geologic map of the spray pond area is presented on Figure 2.5-15. Further discussion of conditions at the ESSW pumphouse and spray pond are contained in Subsections 2.5.1.2.2, 2.5.3 and 2.5.5.

The area underlying the diesel generator 'E' fuel tank consists of a dense to very dense glacial outwash and till deposit. The deposit consists of a sequence of sand, gravel, cobbles, and boulders overlain by sand and gravel, overlain in turn by crushed stone.

2.5.1.2.5.2 Landslide Potential

Natural slopes adjacent or close to the principal plant structures are relatively flat. Most of these slopes are composed of soil; few rock slopes occur (Figure 2.5-17 shows areas of rock outcrops).

North of the spray pond the Trimmers Rock Formation forms a relatively steep ridge rising approximately 380 ft. above the pond. The south-facing slope of this ridge is essentially a rock slope underlain by flaggy, resistant sandstone thinly mantled with soil and rock fragments. The closest approach of this slope to the spray pond is along the northern perimeter of the pond; the toe of the slope, at elevation 710-720 feet, is 250 feet or more from the edge of the pond (at elevation 679 feet). The maximum slope along the ridge is about 2 horizontal to 1 vertical, with an overall slope of 3-1/2 horizontal to 1 vertical, a relatively flat slope for competent rock. Figure 2.5-56 shows a typical profile along the steepest portion of this slope north of the spray pond area. Bedding in the rock dips approximately 30° to the north into the slope; thus, it is favorably oriented for slope stability. Data of McGlade, et al. (Ref. 2.5-56, p. 147) indicate that natural slopes eroded on Trimmers Rock strata are "steep and stable."

No old landslides, rock slips, or landslide scars have been noted near the plant structures. It is concluded that the natural slopes present no significant hazards to plant structures.

Stability of excavated and fill slopes is discussed in Subsection 2.5.5.

2.5.1.2.5.3 Areas of Potential Subsidence, Uplift, or Collapse

Potential sources of subsidence or collapse in the Pennsylvania Valley and Ridgeregion include coal mines and karst terrain; however, neither of these conditions are known to occur within several miles of the site and therefore they are not significant to the site.

No coal beds are present beneath the site; the nearest coal measures are about 3-1/2 miles north of the site near Shickshinny, which is at the extreme western end of the anthracite producing belt in the Lackawanna syncline (Figures 2.5-25 and 2.5-26). The nearest underground coal workings are about 2 miles east of Shickshinny (Ref. 2.5-70); the nearest that

have been associated with surface settlement are near Nanticoke, Pennsylvania, approximately 10 miles northeast of the site. Rocks in the site area have no known potential for oil or gas production. The nearest oil or gas field is located 25 miles northeast of the site.

The shallowest carbonate rock that may be present beneath the site occurs below the Marcellus-Manhantango sequence as limy beds within the Upper Silurian, Tonoloway and Wills Creek Formations and the Lower Devonian, Keyser, Old Port, and Onondaga Formations (refer to Subsection 2.5.1.1.2.3 and Figure 2.5-14). Some of these units crop out on the flank of the Berwick anticlinorium north of Bloomsburg about 10 miles west-southwest of the site, but most are absent nearer than this to the site having been removed by erosion or faulting (Subsection 2.5.1.1.2.3). None have been mapped within five miles of the site (Figure 2.5-12). The Onondaga may occur in the subsurface near Berwick, five miles west-southwest of the site, inasmuch as mud-filled caves have been encountered during well drilling operations at Berwick; however, since the secondary porosity along joints and bedding planes in the Onondaga has been characterized as of only "medium" magnitude (Ref. 2.5-56), such cavities would be expected to be neither large in size nor extensively developed. If the Onondaga does extend eastward beneath the site, it would be at a depth probably exceeding 1,000 feet and beneath the Marcellus-Mahantango shale and siltstone sequence (Figure 2.5-14). At this depth of burial beneath the site, carbonate beds possibly present would have no significant potential for subsidence or collapse at ground surface.

The site is not known to be in an area experiencing localized doming or subsidence. Relative rates of regional uplift or subsidence are not well defined for this area, but some recent studies have been presented in the literature. Brown and Oliver (Ref. 2.5-54, Figures 5 and 7) show a releveling profile across the Appalachians at the latitude of Harrisburg about 60 miles south of the site. This profile suggests that the Valley and Ridge province at the latitude of Harrisburg is rising uniformly at the rate of about 5 mm per year. They find in general that "the Appalachian Highlands are being uplifted with respect to the Atlantic Coast at rates up to 6 mm per year" (Ref. 2.5-54, p. 31). Because the measurements are referenced to a given station, it is not possible to determine absolute vertical crustal velocities. Since Brown and Oliver (Ref. 2.5-54. p. 31) also state the "Atlantic Coastal Plain... is tilting away from the continental interior" the data may indicate simply that the Appalachian Highlands are nearly stationary, or that they are subsiding more slowly than the coastal region. Inasmuch as differential rates of this magnitude are greater than the geologic record suggests could be sustained over geologic time, the authors presume an oscillatory mode of continental interior uplift or coastal depression with time on the order of 106 years per oscillation.

Superimposed on these broad, regional differentials are smaller, secondary variations in the rate of vertical movement within the Appalachian Highland on the order of 1 to 3 mm per year total amplitude (Ref. 2.5-54, Figures 7 and 8). The location of some of these secondary maxima or minima appear to correlate spatially with seismicity; others do not. The wave length between such secondary maxima is on the order of 300 km, a distance suggestive of origin in "large scale movements of the earth's crust" (Ref. 2.5-54, p. 27). Although the authors speculate there may be, in some areas, a possible association of these secondary peaks in the vertical velocity profiles with seismicity, they acknowledge (Ref. 2.5-54, p. 30) that "without further data it is impossible to demonstrate that the relationship is more than coincidental." In any event, flexure of a very few millimeters over hundreds of kilometers is very broad indeed and is not significant to structures.

In eastern New York, post-glacial offsets in shales and slates are documented by Oliver, et al. (Ref. 2.5-51). The nearest locality listed by Oliver, et al. (Ref. 2.5-51) is near Hyde Park, New York, about 120 miles east of the site. The authors wonder whether the cause of these offsets "might stem from crustal rebound following removal of the ice load or from still more broadly based tectonic or orogenic forces" (Ref. 2.5-51, p. 586). However, it is significant that these high-angle reverse offsets of the glacial striations in bedrock are on the order of only one inch or less of displacement. The authors also mention other possible mechanisms such as "...thermal changes, hydration, or a chemical process in the shales," and conclude the data "do not seem adequate to resolve this point" (Ref. 2.5-51, p. 569). Another assessment of the same data concluded the offsets arise from either frost heave or glacial rebound. If related in some way to frost action or the severity of frost, then one might expect the effects of heave, such as on precise leveling monuments, to be more pronounced with altitude. Precisely such a correlation of secondary velocity maxima with elevation was noted and discussed by Brown and Oliver (Ref. 2.5-54, p. 27).

Additional independent evidence on the magnitude of general Appalachian uplift, or lack thereof, in the Pliocene and Quaternary is provided by Owens (Ref. 2.5-52), who based his assessment on the assumption that uplift in the source area is accompanied by erosional transport of clastic material to adjacent basins. He found that Pliocene and Quaternary sediments of the Atlantic coastal plain from New Jersey southward are only 50 feet or less in thickness, indicating no great intensity of uplift through this period; and moreover that there are no marked accumulations of sediment in centers of deposition, suggesting a general, uniform uplift, or even static conditions, of the entire Appalachians.

Therefore, the total geologic record strongly suggests that unusual regional crustal instability has not recently occurred in the Appalachians. Brown and Oliver (Ref. 2.5-54, p. 31) conclude, "Although the rates of relative vertical movements determined from leveling seem large by comparison with rates deduced from some forms of geological evidence...these velocities do not seem unreasonable in terms of other types of geological information." Further, "the rates of vertical crustal movement presented . . . compare very favorably with those found in other portions of the world." Relative rates of vertical uplift observed for the site region therefore appear to be quite typical compared to observations elsewhere, and accordingly do not seem to be reflective of abnormal conditions.

The balance of evidence favors Appalachian crustal activity restricted to generally uniform uplift, probably differing slightly in local areas, and perhaps having an oscillatory character with time. Little if any evidence has been presented to demonstrate that these may be significant to engineering planning or seismic risk evaluation. No faulting has been shown to be involved in this recent activity, and correlation with seismicity is likewise not established. In areas adjacent to the Appalachians, small-scale post-glacial offsets have not been correlated with seismicity and tectonic origin of the offsets has not been established.

It is concluded that the available data do not indicate that existing or future uplift or subsidence, as from man's activities or from geologic conditions such as regional warping, will be significant to the site.

2.5.1.2.5.4 Behavior of Site During Prior Earthquakes

There is no evidence at the site of any effects, such as landslides, fissuring or subsidence, which could be attributed to the occurrence of prior earthquakes.

No Pennsylvania earthquakes have been reported as felt in the site vicinity. Within historical times, approximately fourteen earthquakes originating outside Pennsylvania could have been felt at the site, with the probable maximum intensity of IV on the Modified Mercalli Scale. The nearest of these earthquakes occurred 90 to 100 miles from the site (Wilmington, Delaware, 1871, epicentral Intensity VII). Ground motion at this intensity (IV) would have had no effect on the site.

2.5.1.2.5.5 Zones of Deformation or Structural Weakness

As reported in the PSAR, the preconstruction investigation defined a number of structural features at the site such as folds, joints, fractures, cleavage, slickensided joint planes, and bedding plane shears. The PSAR stated (p. 2.5-16),

> The prominent joint directions are parallel and perpendicular to the strike of the strata. The major joints...(strike) parallel to bedding. This joint set dips nearly perpendicular and opposite in direction to the dip of the bedding. A more open but less frequent series of vertical joints strikes parallel to the dip of the strata. High angle joints healed by secondary calcite and quartz mineralization are present in the vicinity of minor shear zones. The observed fractures, while relatively numerous in the upper 20 feet of rock, decrease with depth. At a depth of about 20 feet into rock, the fractures are tight (generally healed with calcite filling) and would not adversely affect foundation performance.
>
> Minor bedding plane slips at depth have been observed in the site area, both north and south of the interior ridge. Those slips have not experienced movements in more than 200 million years. A minor slip of this nature could be exposed in any large excavation anywhere in the area; however, it would not affect the structural design of the facilities.

Excavation during construction confirmed the PSAR evaluation and supplied additional data. During excavation, numerous slickensided joint planes were exposed and mapped (Figure 2.5-18). Thin, slickensided bedding plane shears, well healed with laminar quartz and calcite mineralization, were also exposed in the foundations. The field data indicate these shear planes are folded in the same manner as the bedding (Figure 2.5-19). They are, moreover, cut by vertical, calcite-filled joints which are continuous across the shears with no offset. In addition, where the shears can be traced to an intersection with a smooth, glacially eroded surface forming the top of rock, the eroded surface displays no displacement or offset across the shear plane. Since the erosion of the rock surface would necessarily have occurred prior to the deposition of the overlying glacial deposits, which have been established as being more than 50,000 years old (refer to Subsection 2.5.1.2.2.1), this relationship shows that any displacement along the shearing occurred more than 50,000 years ago. In reality, the most probable age of the shearing is pre-Triassic, or over 200 million years ago. This is indicated by regional relationships plus the fact that the shear plane is folded (A detailed presentation and analysis of the relationship between site and regional structure is presented at the end of Subsection 2.5.1.2.3.2).

All features arising from tectonic deformation at the site are geologically old. In the foundation rock, all shears and joints are tight or are fully healed with calcite and quartz mineralization; accordingly, they do not adversely affect the strength, bearing capacity or compressibility of the foundation rock. They are therefore not significant to the plant structures. The conclusion stated in the PSAR (P. 2.5-18) that "the minor structural conditions observed at the site are not of significance with respect to siting or design for the use of the site for its intended purpose," has been confirmed.

2.5.1.2.5.6 Zones of Alteration or Irregular Weathering

Bedrock beneath the seismic Category I structures is a dark gray, indurated, massive siltstone. It is not susceptible to rapid weathering; no appreciable slaking of fresh rock exposures was observed in the foundations. Depth of weathering below original top of rock varies from zero to about 20 feet. The rocks occupying depressions in the bedrock surface are generally unweathered. However, open fractures were encountered to a depth of about 20 feet. Below that depth, fractures are not common but where they do occur, they are generally "healed" with calcite filling.

Weathering appears to have progressed initially downward by dissolution of calcite from calcite-coated joints, seams and shear planes, and refilling by clay or other weathered material. In one exceptional case, weathering along joints and fractures, as distinguished from weathering of the rock itself, was observed nearly 70 ft. below original top of rock. In this instance, which was between the circulating water pumphouse and the Unit 1 cooling tower, the bedrock originally formed a knoll and contained numerous low angle, slickensided, calcite and quartz-filled joint planes and abundant vertical, calcite-filled fractures, forming an intersecting, permeable network of channels through which water readily percolated downward, dissolving the soluble carbonate joint and fracture fillings. Notwithstanding the depth of weathering here, the design elevation of the bottom of the circulating water pumphouse is below the depth to which this zone is weathered, and this structure was founded on sound unweathered bedrock.

2.5.1.2.5.7 Potential for Unstable or Hazardous Rock or Soil Conditions

Foundation rock at the site is hard, indurated, unweathered siltstone, a member of the Middle Devonian Mahantango Formation. Similar materials underlie the site to a depth of over 1,000 feet. This rock does not contain unstable minerals; it provides highly stable foundation conditions and does not constitute a source of potential hazard to the plant.

Soils at the site are, except for the uppermost few feet, glacial in origin and consist of resistant fragments of rock that were transported from the region north of the site. Most were deposited by flowing melt water from the glaciers under torrential flow conditions, and some of the soils probably have been overridden by ice sheets. These glacial soils are noncalcareous and over four-fifths of the rock in them consists of sandstone (Ref. 2.5-5).

The origin and mineralogy of the soils is such that they present no hazardous conditions. Detailed engineering characteristics of soils in regard to slope stability, bearing capacity, stability under dynamic loads and consolidation characteristics under structural loads are discussed in Subsections 2.5.4 and 2.5.5.

All foundations for the Category 1 plant structures that are founded on rock are excavated to or into essentially unweathered material. No significant irregular alteration or weathering, or zones of structural weakness due to weathering, shearing or fracturing were encountered at the bearing elevation for those structures designed to rest on unweathered rock.

2.5.I.2.5.8 Unrelieved Residual Stress on Bedrock

No indications were found during excavation and construction at the site of the presence of significant stress in bedrock. No popping or spalling rock was observed. There were no indications of heave at the base of rock slopes or in the bottom of excavations. Some vertical joints close to and subparallel to vertical excavated slopes opened very slightly, but this was attributed to gravity forces, not in situ stresses. If significant in situ stresses did in fact exist in the rock, evidence of this should have been noted in the excavation; no such indications were noted. For example, a thin mudmat was routinely placed on the rock after evacuation to foundation grade. If significant heave had occurred, it would have been readily detected by cracking of the mudmat. No such cracking was observed.

2.5.I.2.5.9 Conclusions and Summary

Consideration of all engineering geologic factors at the Susquehanna site leads to the conclusion that the site is well suited for the construction and operation of the plant. There is no geologic feature or condition at the site or in the surrounding area which precludes the use of the site for a nuclear generating facility. The bedrock in the construction area is competent and provides satisfactory foundation support for all major structures.

2.5.1.2.6 Site Groundwater Conditions

The groundwater table beneath the site generally occurs within 35 feet of the ground surface. The notable exception is in the deep, easterly sloping, buried bedrock valley present about 1000 feet north of the center of the plant, where a water table depth of as much as 79 feet was recorded. Generally, the lower part of the overburden deposits is saturated, although over portions of the upland area on the site, the groundwater level is found only in the underlying bedrock. Depth to bedrock is variable and ranges from zero to over 100 feet. The groundwater level contours shown in Figure 2.4-31 appear to be controlled to a large extent by the top of bedrock contours (Figure 2.5-17).

Groundwater movement on the site is generally in an easterly direction. With the exception of a few springs on site. Most of the groundwater is believed to discharge ultimately to the Susquehanna River. The average groundwater velocity is estimated to be between 1.5 and 2.0 feet per day along flow paths from the station to the Susquehanna River.

The site is not located in a recharge zone for any aquifer. However, groundwater recharge to the unconsolidated sand and gravel does occur over the site area. The predominantly siltstone strata of the Mahantango Formation beneath the site constitutes a source of limited domestic water supply. Because of its relatively low transmissibility characteristics, the Mahantango cannot be considered to be an aquifer.

From a hydrologic standpoint, there are two general types of aquifers in the region. The first type consists of sandstone and occasional limestone strata which occur within the predominantly shale sections of the Paleozoic age bedrock. The second type is unconsolidated Quaternary deposits which consist for the most part of Pleistocene stratified drift, till, or kames which are often overlain by a thin recent soil cover. From a survey of domestic water supplies within two miles of the station, it was found that nearly all of the wells are completed in shale bedrock.

Detailed information on groundwater conditions and movement on the site and in the region is given in Subsection 2.4.13.

2.5.2 VIBRATORY GROUND MOTION

A discussion and evaluation of the seismotectonic characteristics of the Susquehanna SES site and the surrounding region is presented in this section. The purpose of this investigation is to present the seismic design criteria for major structures at the station in conformance with guidelines as outlined in Standard Format and Content of the Safety Analysis Reports for Nuclear Power Plants, and Appendix A of 10 CFR, Part 100.

A description and results of the field investigation and laboratory testing program, which provided background information for this investigation, is presented in detail in Subsection 2.5.1.

2.5.2.1 Seismicity

The station is situated in a region which has experienced only a minor amount of moderate earthquake activity in historic time. The record of earthquake occurrences in the region dates back to the middle 16th century. Many earthquakes have been reported since that time and minor structural damage has been associated with several of the events; however, none of these earthquakes were considered to be of major or catastrophic proportion. Because this region has been fairly heavily populated since the early 18th century, it is quite certain that any significant earthquake activity (MM Intensity VII or greater as defined by the Modified Mercalli Intensity Scale, 1931, see Table 2.5-1) would have been reported in local newspapers, private journals or diaries. The lack of any such documentation is indicative of the absence of significant major earthquake activity in the region during this period.

Structural damage is the primary rating criterion for larger shocks. The effects of earthquakes on the rather large variety of construction materials used in older structures such as chimneys, rock walls, etc., are highly variable, making intensity evaluations based on such reports imprecise. The rather long period of record, however, and the evenly distributed population provide a reasonable basis for estimates of future activity.

Table 2.5-2 lists all events within 200 miles of the station with magnitudes (Richter) greater than 3.0 or MM intensities greater than III, and all seismic events within 50 miles of the station. Figures 2.5-8A and 2.5-8B displays these events on the regional structure of the area around the station, along with the significant earthquakes (MM Intensity V and greater) which have occurred outside the 200-mile radius.

The largest events to have occurred in the immediate station vicinity were the Wilkes-Barre disturbances of February 21 and 23, 1954 (local dates), with assigned intensities of VII and VI, respectively. These intensities were based on the damage inflicted upon a very small area, about 0.15 square miles of the city. These disturbances occurred only 16 miles from the station; however, they are considered to have resulted from mine collapse as discussed in detail in Appendices 2.5A and 2.5B. Thus, these shocks need not be considered in the analysis of earthquake risk as regards the station.

The largest event to have occurred within 200 miles of the station was the Intensity VII-VIII shock at Attica, New York, on August 12, 1929, some 150 miles northwest of the station. This earthquake imposed some moderate damage at Attica and villages in the immediate vicinity of the epicenter, but was not reported as felt in the Berwick area (Ref. 2.5-71).

Four Intensity VII earthquakes have occurred at a distance of about 100 miles from the station: two (1737 and 1884) were near New York City, one (1927) near Asbury Park, New Jersey and one (1871) near Wilmington, Delaware.

The closest of these Intensity VII events was the shock which occurred in the vicinity of Wilmington, Delaware on October 9, 1871, approximately 100 miles to the south of the station. Based on damage reports and intensities felt, the epicenter has been located near Wilmington, whereas the shock was felt from near Chester, Pennsylvania on the north to Middletown, Delaware, on the south and from Salem, New Jersey on the east to Oxford, Pennsylvania on the west. The initial shock was followed by a much smaller shock just after midnight on October 10. A contemporary newspaper account indicates that the initial shock was felt at Wilmington "with greater distinctness." Buildings were shaken severely and a number of chimneys were damaged in the surrounding towns of Oxford, Pennsylvania, and New Castle and Newport, Delaware. An interesting aspect of this earthquake is the fact that it was accompanied by a very loud sound, as of an explosion. This loud noise, in fact, led to the belief that the shock was caused by an explosion, probably at the powder mill of the E.I. DuPont deNemours Company, near Wilmington. This possibility was carefully investigated at the time and it was concluded that the shock was a legitimate earthquake. Existing reports, however, do not report the shock being felt in the Berwick area.

The two events near New York City, about 118 miles from Berwick, may have been felt at the station, but with an intensity certainly no greater than III. The 1884 shock affected an area extending from Portsmouth, N.H., to Burlington, Vt., southwest to Binghamton, N.Y., Williamsport, Pa., southeast to Baltimore, Md., and Atlantic City, N.J. At Bradford, Pennsylvania, reports were made of panes of glass broken in a large hotel, and other moderate damage was sustained. All hotels in Brooklyn, New York were shaken violently. In 1927, a similar shock listed as Intensity VII was centered near Asbury Park, New Jersey. Several successive waves, described as seeming to travel from west to east, caused homes near Asbury Park to shake and oscillate perceptibly.

Several Intensity VI earthquakes have occurred less than 100 miles from the station. On May 31, 1908, 48 miles from the site, Allentown, Pennsylvania was shaken by what was believed to be a mild earthquake. The shock lasted about a second and was described as a rumbling sound followed quickly by a report which "sounded like the falling of chimneys of a building" (The Lehigh Register June 3, 1908). The Philadelphia Inquirer adds, "The only other place where the shock was felt was Catasauqua, three miles away. At first it was thought that a battery of boilers in some local industry might have exploded, but no such accident was

reported. The quake was accompanied by a low rumbling noise and lasted about two seconds but was only felt over an area of some 50 square miles. On January 7, 1954, an Intensity VI shock occurred approximately 54 miles from the station. This was the first of a series of minor shocks in Berks County. The Reading Times reported on January 8, 1954, that a "minor" earthquake shook Berks County communities, and succeeding tremors of lesser intensity were experienced mainly by residents in West Reading, Wyomissing, West Louwe, West Wyomissing, Wyomissing Hills and Sinking Spring. Continued mild aftershocks shook the downtown area of Sinking Spring which appeared to be the epicentral area for these disturbances. The damage was described (Ref. 2.5-72) as minor: broken chimneys, breaks in brick and plaster walls, and broken dishware. Similar damage, including broken windows, was reported in other communities west of the Schuylkill River. The main shock was recorded by seismograph stations at Fordham, Palisades (Columbia University) and the U.S. Coast & Geodetic Survey at Washington, D.C. Dr. Jack Oliver of Columbia University described the initial tremor as "a typical east coast earthquake."

According to the Philadelphia Inquirer, as of January 8, 1954, the earth tremors which had been recorded in Berks County since 1900 were:

- August 30, 1902
- June 6, 1915
- February 28, 1925
- November 1, 1935
- June 8, 1937
- February 18, 1938
- September 4, 1944

The events of 1902, 1915 and 1938 are not reported in the standard catalogues and are, therefore, considered as very small, localized disturbances for which there is only local record. The shocks of 1925 and 1944 were large events in the St. Lawrence River near Quebec, Canada and Massena, New York which were felt with intensities of less than III at the station.

Shocks on January 24, 1954 and on August 11, 1954 affected Sinking Hole, Pennsylvania, according to the Reading Times (August 11, 1954). These shocks were attributed by the U.S.G.S. to the caving of solution cavities. Similar conclusions about the "sinking of the earth in general" were reached by Penn State University scientists after the event on September 24, 1954 which was assigned an Intensity II at Sinking Spring Borough,5 miles west of Reading, Pennsylvania.

Another shock of Intensity VI occurred 63 miles southeast of the station, near Cornwall, Pennsylvania, on May 12, 1964. Coffman and Von Hake (Ref. 2.5-72) report a cracked wall, fallen plaster, and small landslides. The Lebanon Daily News reported: "...the tremor was so mild that many persons slept right through it." However, Dr. B.F. Howell, Jr., Chairman of the Geophysics Department at Penn State University, reported that the quake was the most intensive to hit the state in l0 years.

On September l, 1895, an event of Intensity VI near Philadelphia, 76 miles from the station, was felt from Sandy Hook, New Jersey to Brooklyn, New York to Darby, Pennsylvania, and Wilmington, Delaware. Another shock of Intensity VI occurred on March 23, 1957 in the same general vicinity, 79 miles southeast of the station. These shocks were not reported felt in the Berwick area.

Five shocks (Intensity VI, III, V, III, and IV) occurred in central and southern New Jersey on August 23, 1938, 116 to 128 miles from the station. The largest shock was felt from northern New Jersey to Wilmington, Delaware.

Although it is indicated that several of the large, distant shocks listed above were probably felt at the station, no damaging effects were experienced. No Pennsylvania earthquakes have been reported as felt in the area of the Susquehanna SES.

In summary, there are no reports from the Berwick area of Pennsylvania which would indicate that ground motions from any historical earthquake in the east have exceeded (or even equalled) an intensity as great as IV on the competent rock on which the station is located.

2.5.2.2 Geologic Structures and Tectonic Activity

2.5.2.2.1 Tectonic Provinces

The area within a 200 mile radius of the Susquehanna SES includes parts of six tectonic provinces (Figures 2.5-8A and 2.5-8B). The provinces are, from west to east, Stable Interior, Fold and Thrust Belt, Blue Ridge-Highlands, Conestoga Valley, Inner Piedmont, and Coastal Plain.

The tectonic province concept used to define these provinces is based on an evolutionary model of the Appalachian orogen (Ref. 2.5-73) and derived from early studies in the region (Ref 2.5-74). This concept was used in this study to provide the province boundaries of significance to the station.

2.5.2.2.2 Tectonic Differentiation of the Appalachian Orogen

The outline and discussion presented below summarizes the relationships and derivations of tectonic provinces of the Appalachian orogen, as displayed in Figures 2.5-8A and 2.5-8B any parts of which occur within 200 miles of the station.

1. Craton
    a. Eastern Belt
        (1) Blue Ridge-Highlands
    b. Western Basin
        (1) Stable Interior
        (2) Fold and Thrust Belt
2. Mobile Belt
    a. Eastern Cratonic Margin
        (1) Conestoga Valley
        (2) Inner Piedmont
        (3) Coastal Plain

Considering the tectonic evolution of the Appalachian orogen, it is subdivided as above into two fundamental areas; the part affected only by convergent diastrophism (craton), and the part affected by initial divergent, convergent, translational, and final divergent diastrophisms (mobile belt). The mobile belt, as defined in this report, is situated east of the great anticlinoria cored by Grenvillian rocks; i.e., east of the Long Range (Nova Scotia), the Green Mountains, the Berkshire Highlands, the Hudson-New Jersey Highlands-Reading Prong, and the Blue Ridge Mountains. The mobile belt thus corresponds to the Appalachian eugeosyncline and includes the quasi-cratonic margins. The western edge of the mobile belt parallels and lies to the west of what was originally the eastern edge of the North American continent during Cambro-Ordovician time as defined by Rodgers (Ref. 2.5-74).

2.5.2.2.3 Tectonic Differentiation of the Craton

The cratonic portion of the Appalachian Highlands is underlain by continental crust composed of 1000 million-year-old crystalline rocks which were deformed during the Grenvillian orogenic cycle. On the eastern edge of the craton, these rocks crop out at the surface as great anticlinoria. West of these elevated anticlinoria lies an elongated, downwarped segment of the continental crust forming the asymmetric Appalachian basin. The floor of this basin is formed of Grenvillian rocks greatly depressed in the east (up to 40,000 feet below sea level) and gradually rising toward the west. The basin is filled with largely unmetamorphosed sedimentary rocks (both clastic and carbonate) ranging in age from Early Cambrian to Carboniferous. These rocks form a sedimentary wedge, thickening to the southeast, reflecting the asymmetry of the basin floor.

Blue Ridge - Highlands Tectonic Province

The eastern portion, termed here the Blue Ridge - Highlands, constitutes a tectonic province and is characterized by Grenvillian rocks deformed during the Paleozoic convergent stage.

Characteristically, the terrain is mountainous and exhibits exposure of some of the oldest rocks in the eastern U.S. (1000-1100 million years). Earthquakes no greater than Intensity VI are characteristic of this tectonic regime, and none have been related to specific structures.

Stable Interior Tectonic Province

The Stable Interior Tectonic Province of the western basin is characterized by the absence of intense deformation and the presence of shelf-delta type Paleozoic sediments.

The rocks display gentle folding as opposed to the intensely folded and faulted rocks of the Fold and Thrust Belt immediately to the southeast. The largest significant earthquake to have occurred in this province (within the regional scope of this study) was the 1929 Attica, New York, event (initially cataloged as Intensity VII-VIII) approximately 168 miles from the station. This shock and an accompanying concentration of lesser events has been spatially related to the Clarendon-Linden Fault, an anomalous structure in the essentially untectonized rocks making up this portion of the Stable Interior. A small concentration of activity, apparently related to doming of the Adirondak massif, occurs 150 to 200 miles northeast of the station. With the exception of these moderately active areas, the province is virtually aseismic.

Fold and Thrust Belt Tectonic Province

The Fold and Thrust Belt tectonic province is characterized by tightly folded and thrust-faulted Paleozoic sedimentary strata deposited as flysch or molasse. The northwestern boundary of this province generally marks a transition between gently folded strata on the northwest (Stable Interior) and intensely folded and faulted strata on the southeast, thus marking the western limit of Paleozoic thrusting (Ref. 2.5-75).

The largest earthquake which has been recorded in the Fold and Thrust Belt tectonic provinces was the Giles County, Virginia, Intensity VIII shock of 1897, over 350 miles from the station, and in the southerly division of the Fold and Thrust Belt. Other earthquakes in this province are widely scattered with only two events as large as Intensity VI occurring within 200 miles of the station.

2.5.2.2.4 Tectonic Differentiation of the Mobile Belt

The mobile portion of the northern Appalachian orogen within the region of interest for this study includes the eastern cratonic margin, which is underlain by continental crust of predominantly Grenvillian age (Inner Piedmont and Conestoga Valley tectonic provinces).

The eastern cratonic margin is bounded on its western side by the Blue Ridge - Highlands tectonic province and on its eastern side by the eastern most extent of Grenvillian basement.

This eastern boundary is interpreted principally from a line of gneiss domes of one billion year-old continental crust including the Pine Mountain belt, the Sauratown Mountains anticlinorium, the Baltimore Gneiss domes, and, possibly, the Chester dome of Vermont. This boundary corresponds to the eastern limit of the ancient continental margin of North America (Ref. 2.5-76 and 2.5-77). It also coincides with several significant geological and geophysical changes (Ref. 2.5-76). First, it parallels the main gravity high of the Appalachians (Ref. 2.5-78). Second, it is marked by contrasting seismic refraction profiles that reflect deep crustal contrast. Finally, it is a zone of faulting, contrasting structural style and contrasting metamorphic facies.

This cratonic margin is divided into two tectonic provinces north of Virginia, the Conestoga Valley province and the Inner Piedmont province. The boundary between these provinces corresponds to the Martic Line in Pennsylvania (Ref. 2.5-79) and the southward extension of Cameron's Line in western Connecticut.

Conestoga Valley Tectonic Province

The Conestoga Valley tectonic province is characterized by a miogeosynclinal assemblage overlapping an older clastic assemblage. Triassic Basins of the Newark Group are characteristic of the Conestoga Valley province (and, to a lesser extent, the Inner Piedmont and Coastal Plain) and are found in the area between Massachusetts and North Carolina. Triassic rocks have been encountered in borings at Bowling Green and Edgehill, Virginia and near Brandywine, Maryland.

These basins were formed during Triassic time as downfaulted and folded elongategraben structures. Non-marine arkosic sediments and intercalated lava flows filled these basins as they were down-faulted and tilted. At the close of the period, the processes of erosion continued to

modify the topography of the eastern section to form the base forde position of Coastal Plain sediments. In the Triassic Basins and associated down-faulting, intrusions of Trio-Jurassic age are cut and displaced, indicating a post-Trio-Jurassic age for some of the faulting.

Similar intrusions in the Inner Piedmont are not disrupted or offset in this manner. Earthquakes no larger than Intensity VI have been noted in the Conestoga Valley province, although some small events, up to Intensity VI, are reported and have been tentatively associated with Triassic basin border faults.

<u>Inner Piedmont Tectonic Province</u>

The Inner Piedmont Tectonic Province is characterized by a eugeosynclinal assemblage over an older clastic assemblage, which is characterized in this region by a northeast-southwest trending belt of Precambrian to early Paleozoic schists, gneisses, slate, metaconglomerates and some igneous intrusions. These rocks are interrelated in a complex manner by faulting and folding.

Earthquakes ascribed to the Inner Piedmont should include the boundary (Inner Piedmont-Coastal Plain) Intensity VII events at Wilmington, Delaware (1871) and Asbury Park, New Jersey (1927) as well as several Connecticut valley events of Intensity VII which occurred over 200 miles from the station, albeit, in the Inner Piedmont Province (Ref. 2.5-73). No larger events have been recorded in this province and none of the historical shocks can be satisfactorily related to specific structures. The Inner Piedmont is, in general, apparently the most seismically active portion of the area within 200 miles of the station. Concentrations of moderate events are apparent in the New York City area in and the Central Virginia seismic zone near Charlottesville as described by Bollinger (Ref. 2.5-80). Both of these zones are characterized by low to moderate seismic activity. Seismicity elsewhere in the province is relatively rare and apparently random.

<u>Coastal Plain Tectonic Province</u>

The Coastal Plain tectonic province is characterized by the development of a miogeosynclinal wedge during the advanced phases of the final crustal divergence. In the region south and east of the station, this province is characterized by a stratigraphic sequence of interbedded sands, gravels, clays and silty sands of both marine and continental origin. These materials were deposited on the downwarped basement complex from Early Cretaceous to Quaternary time. The strata wedge out at the Fall Zone to form a wedge-shaped mass that thickens to the southeast. The average dip of these strata varies from 75 feet per mile within the Cretaceous sediments to approximately 10 feet per mile in the upper Tertiary formations.

Few geologic structures are known in the Coastal Plain Province. The Salisbury Embayment is a structural low in the basement rocks between Newport News, Virginia and Atlantic City, New Jersey. The Embayment is marked by a deep accumulation of Mesozoic and Cenozoic sediments, which approach a thickness of 3,500 to 7,500 feet at the Maryland coastline. The feature is fairly prominent in the basement rocks but loess form in the younger sedimentary sequences, suggesting that it is predominately a pre-Tertiary feature. The Coastal Plain underwent regional epeirogenic movements from Pliocene to Quaternary time, which lifted a portion of the continental terrace above sea level. The significant seismic activity in the Coastal Plain includes the Intensity X event at Charleston, South Carolina, and, for the sake of conservatism, the Wilmington, Delaware event of Intensity VII.

2.5.2.3 Correlation of Earthquake Activity with Geologic Structures or Tectonic Provinces

Only a few of the historical earthquakes in the northeastern United States can be satisfactorily related to specific structures at this time. Therefore, a consideration of the significant events which could influence the seismic design for the Susquehanna SES will rely, for the most part, on an approach based on the tectonic settings discussed above. To augment the tectonic province approach, the concept of the seismic zones within the provinces as discussed by Bollinger (Ref. 2.5-80) and Hadley and Devine (Ref. 2.5-81) will be addressed.

Those events which constitute the largest earthquakes of record in the Eastern United States and which embrace all significant considerations for the Safe Shutdown Earthquake for the station, are listed below:

1) The large events (maximum Intensity IX) such as those in the St. Lawrence Valley and Ottawa-Bonnechere Graben area

2) The large events such as those (maximum MM Intensity VIII) which occurred in the Cape Ann Massachusetts area

3) The (originally categorized as Intensity VII-VIII) Attica shock (1929) in western New York State

4) The Intensity (IX to X) Charleston, South Carolina earthquake in the Coastal Plain

5) The Intensity VIII Giles Co., Virginia earthquake of 1897

6) The Intensity VII events such as those shocks which have been recorded in and around New York City, Wilmington, Delaware, Asbury Park, New Jersey, and Lake George, New York, and

7) The Intensity VI events which occur only infrequently in the general region

St. Lawrence Valley

The St. Lawrence Valley and the Ottawa-Bonnechere Graben area are contained in the Ottawa Basin tectonic province (Ref. 2.5-73). Earthquakes as large as Intensity IX are reported in this region. The structural interpretation shows that this area is the extension of a transverse trough and mobile zone into the stable interior (Ref. 2.5-73).

Because of the obvious historical confinement of seismic activity to this region marked by an intraplate weakness, recurrence of such large shocks are expected to remain in the area and are thus not translatable to the station.

Boston-Cape Ann

The large (Maximum Intensity VIII) events in the Boston-Cape Ann area were formerly historically associated with the Boston-Ottawa trend of earthquake activity (Ref. 2.5-82) which included the Ottawa-Bonnechere Graben area. However, a recent re-evaluation (Ref. 2.5-73) has resulted in the identification of tectonic regimes which separate the former "Boston-Ottawa

trend" into specific tectonic provinces. On the basis of this, the Cape Ann Intensity VIII event, being the largest event to have occurred in the Avalon Platform province (Ref. 2.5-73) would be restricted to a distance from the site of no less than 250 miles. Moreover, according to Ballard and Uchupi (Ref. 2.5-83), it is possible that the significant Boston-Cape Ann seismic activity is associated with the faulted northwestern boundary of the Avalon Platform.

For these reasons it is not deemed necessary to translate this activity (Maximum VIII) out of the Avalon Platform.

Western New York

The shock of 1929 near Attica, New York is anomalous with respect to the exceedingly sparse seismicity of this portion of the Stable Interior. It does mark, however, a noted concentration of earthquakes which are spatially related to the well-recognized feature of the immediate area, the Clarendon-Linden Structure (Ref. 2.5-114). It is generally accepted that any recurrence of a similar event would be confined to the Attica area. Therefore, the postulation of a recurrence of this shock at the closest approach of Stable interior to the station is not warranted. A recurrence of the largest event at any location along the Clarendon-Linden Structure could result in only minimal ground motion at the station (less than Intensity IV).

Charleston, South Carolina

The largest events to occur in the eastern United States were the events of approximately Intensity X at Charleston, South Carolina in 1886.

The concentration of seismic activity (over 400 events) in the immediate vicinity of Charleston is unique to the Atlantic Coastal Plain; moreover, such a confined density of epicenters is unmatched anywhere in the central and eastern United States, with the possible exception of the New Madrid, Missouri region. On the strength of this areal distribution alone, it would be concluded that a specific tectonic anomaly is responsible for this localized activity. Independent lines of investigation have recently suggested a structural regime which may be responsible for the observed seismicity. On the basis of seismic reflection profiles parallel to the coast of South Carolina, Dillon (Ref. 2.5-84) has reported evidence of northwest-trending faults in the continental shelf along the South Carolina coast, and states that this would seem to be the only zone of active faulting in the United States south of Cape Hatteras and east of the Appalachians. Possible evidence of faulting is noted in the basement rocks offshore and in the Tertiary rocks of the continental margin. This possible faulting aligns with the northwest-trending seismic zone (Ref. 2.5-80) and has been postulated to be the extension of an active oceanic fracture zone into the continental block (Ref. 2.5-84, and 2.5-82).

More locally, a mild, breached fold in the shallow sediments several miles west-southwest of Charleston has been identified by Colquhoun and Comer (Ref. 2.5-85) as the Stono Arch. The axis of this arch trends west-northwest and has possible associated faulting. The trend of this structure is aligned with, and grossly parallel to, the seismic zone and the offshore structure discussed above, and represents the only known deformation in the immediate vicinity of Charleston. Thus, it may be a near-surface expression of the more regional (and deeper) anomaly suggested by offshore reflection surveys and magnetic anomaly trends (Ref. 2.5-86).

Transverse to the strike of these structural features are the northeast-trending axes of two structural highs which are identified along the coast, from Savannah, Georgia to just south of

Charleston, as the Beaufort-Burton High and the Yamacraw Ridge (Ref. 2.5-84). According to Dillon et al. (2.5-84), the Beaufort-Burton High may be a shallow expression of the deeper lying Yamacraw Ridge. The intersection of these structures with the suggested northwest trends in the vicinity of Charleston may, at least, be an expression of deeper basement complexity in the area, and lends support to a definition of structure responsible for the well-defined cluster of seismic activity in the Charleston area. No other structural anomalies of significance are known in this area of the Coastal Plain. Therefore, the unique density of earthquake activity in the Charleston area is considered to be associated with localized structure, the character and extent of which are only grossly suggested at the present time. In this respect, an earthquake similar to the largest Charleston shock would be expected to recur in the same locale, and would not be subject to translation throughout the Atlantic Coastal Plain tectonic province.

Giles County, Virginia

The Giles County, Virginia earthquake of 1897 is the largest shock to have occurred in the southern Appalachian region. It is listed (Ref. 2.5-72) as Intensity VIII, and occurred in the Southern Appalachian Seismic Zone near its intersection with the Central Virginia Seismic Zone (Ref. 2.5-80), more than 350 miles from the station. This intersection is marked by a definite break in the continuity of the activity of the northeast-trending Southern Appalachian Seismic Zone and lies well to the south of an area of apparent differentiation of the system of tectonic stresses along the Appalachians called the Central Appalachian Salient in southern Pennsylvania.

This salient was probably initiated during early crustal divergence in late Precambrian time (Ref. 2.5-73 and 2.5-87) resulting in a profound difference between the northern and southern portions of the Appalachian orogen as evidenced by three stages of the orogen's development:

1. In late Precambrian the initial rifting stage developed with a bend, offsetting the northern and southern portions of the continental margin.

2. During the end of the convergent stage (middle to late Paleozoic), the Alleghenian orogeny was pronounced only in the south and translation was restricted to the northern Appalachians.

3. In the Jurassic during the final rifting stage, different stress regimes prevailed in the northern and southern portions (Ref. 2.5-88).

The Central Appalachian Salient occurs where the NNE to NE trends, common to the Appalachians, change to EW in the vicinity of 40°N latitude. The EW trend has been interpreted to be a major crustal structure by several authors (Ref. 2.5-89 and 2.5-90) based largely on circumstantial evidence of interpreted offsets of geophysical anomalies, isopach contours, and geologic map patterns. Drake and Woodward (Ref. 2.5-90) have suggested that 80-90 miles of dextral offset has occurred on this feature and that no evidence of post-Cretaceous movement has been found. Figures 2.5-8A and 2.5-8B shows the approximate location of this structure as defined by Woodward (Ref. 2.5-89).

Even though its surficial expression cannot be well-defined, the Central Appalachian Salient clearly divides the northern and southern portions of the orogen. This is borne out by inspection of the historical seismicity shown on Figures 2.5-8A and 2.5-8B, wherein consistent changes in the seismicity within the described provinces are noted from north to south. The virtually

aseismic character of that portion of the Fold and Thrust province containing the station has been noted by the Nuclear Regulatory Commission (Ref. 2.5-91).

Thus, in consideration of the tectonic development, the inferred geological and geophysical evidence, and seismicity, the existence of a fundamental boundary between the northern and southern orogen is herein considered and illustrated as a zone on Figures 2.5-8A and 2.5-8B.

This change of seismotectonic style is further corroborated by Hadley and Devine (Ref. 2.5-81, Sheet 3) who have shown a seismotectonic province generally recognizing the earthquake activity of Bollinger's (Ref. 2.5-80) Southern Appalachian Seismic Zone within the Fold and Thrust Belt. At its northern extent, their boundary stops at the southern Pennsylvania border about 125 miles from the station. They describe the zone as an area where epicentral distribution or relation to known structure indicated a limiting structural factor, and where at least one earthquake of Intensity VII or VIII (Giles County event, 1897) has been recorded.

Because of (1) the notable change in tectonic style in the Fold and Thrust Belt in Province South of the Pennsylvania border, (2) the reduction in seismicity north of the Central Appalachian Seismic Zone, (3) the assignment of a different seismotectonic character to the Fold and Thrust Belt south of Pennsylvania, and (4) the historical record which shows a sparsity of earthquakes in Pennsylvania, we consider that a translation of an Intensity VIII event (Giles County recurrence) closer than 100 miles to the Susquehanna SES is not warranted.

Intensity VII Events

Consideration must be given to the likelihood of Intensity VII events which are known to occur occasionally in this region of the northeast. Within 200 miles of the station, nine shocks of Intensity VII have been documented. Five of these are early reports (1568, 1574, 1584, 1592, and 1791 - See Table 2.5-2) of concentrated activity in the Connecticut Basin about 200 miles from the site. This area lies within the Inner Piedmont Tectonic Province. The other four occurred within the Piedmont province, or at its (eastern) boundary with the Coastal Plain near New York City and northern Delaware. The closest approach to the station of the tectonic province containing these events would be 50 miles. Such an event would attenuate to about Intensity V even on unconsolidated materials at the station, according to conservative central U.S. attenuation characteristics (Ref. 2.5-92), and would be less on competent rock.

The Intensity VII event at Lake George in northern New York, although over 200 miles from the station, is spatially associated with a general concentration of smaller earthquakes. Hadley and Devine (Ref. 2.5-81, Plate C) confine this Lake George event to (1) a tectonic province whose nearest approach to the Susquehanna SES is greater than 150 miles, and (2) a bounded area of seismic activity "in which known faults are associated with epicentral alignments or distribution in such a way as to indicate that movements on the known faults or closely related faults have been the source of recorded earthquakes." This seismic area boundary approaches no closer than 150 miles to the station.

It is seen, then, that Intensity VII events can be confined to approaches of tectonic provinces, seismic zones, and/or structure which are no closer than 50 miles to the Susquehanna SES.

Intensity VI Events

Within 200 miles of the station, a few scattered Intensity VI events are noted (Figures 2.5-8A and 2.5-8B). The two closest events occur about 48-60 miles due south at the closest approach of their tectonic province and are, at least spatially, related to Triassic border faults (Ref. 2.5-81). Several others are concentrated in the immediate vicinity of the Clarendon-Linden structure in northwestern New York State. It should be noted that in the station province (Fold and Thrust Belt) no Intensity VI events occur north of the Central Appalachian Salient in southern Pennsylvania, a distance of over 100 miles from the station. In the Stable Interior, an Intensity VI event in northern New York, 180 miles north of the station, is not related to known structure, and could conservatively be translated to the closest approach of its province to the station, a distance of about 40 miles.

2.5.2.4 Maximum Earthquake Potential

The previous section defined the maximum potential earthquake in terms of the closest postulated approach of maximum historical events to the Susquehanna SES. Consideration was given in each case to a conservative utilization of tectonic province models, recognized seismic zones, and/or any associated tectonic structure. The resulting candidates for the Safe Shutdown Earthquake (SSE) are:

Intensity (MM)	Closest Approach to Site	Maximum Site Intensity
X	450 miles	V
IX	300 miles	V
VIII	100 miles	V-VI
VII	50 miles	V-VI
VI	40 miles	IV-V

In deriving the maximum Intensity to be felt at the station from the above candidates, the attenuation curves developed for the central United States (Ref. 2.5-92) were used. These curves are the most conservative available for the United States in that both western (California) and eastern (Canada, New York, Charleston, S.C.) data show a greater attenuation of Intensity with distance than does the central United States experience. It should be noted, also, that such attenuation relations are based on isoseismal maps which tend to record the maximum Intensity felt in a given locale, usually on poor soil conditions. It is likely, then, that the Intensities (damage potential) actually experienced on solid foundation material of the Susquehanna SES would be somewhat less than those levels specified in the foregoing table which are used in the Safe Shutdown Earthquake derivation in Subsection 2.5.2.6 below.

From inspection of the above candidates, a station intensity of less than VI is the maximum consistent with the tectonic model seismic zones, and/or associated structure. This is entirely in keeping with the historical earthquake record which shows that the area of the station is virtually aseismic. Moreover, there are no known faults which appear capable of generating other than minor disturbances well below damaging levels of ground motion.

2.5.2.5 Seismic Wave Transmission Characteristics

The static and dynamic properties of the subsurface materials at the station are presented in Subsection 2.5.4.2. The analyses presented in this referenced section are based on characteristic ground motion and significant frequencies generated by the maximum potential earthquake described above and quantified below.

2.5.2.6 Safe Shutdown Earthquake (SSE)

As a result of the derivations discussed above, an SSE of less than Intensity VI is the maximum earthquake consistent with tectonic models and historical evidence presented for the site. However, an SSE generating a horizontal ground acceleration of l0 percent of gravity (g) has been selected in compliance with the minimum design requirement of the regulatory agencies.

To justify the conservative nature of this design level as an anchor for design response spectra, the acceleration/intensity correlations which have been developed for an Intensity of VI are discussed, although this Intensity is not expected to be felt at the station on the basis of the discussions above.

Recent correlations between Intensity and peak horizontal ground acceleration have made use of currently available data for the western United States (Ref. 2.5-113) and worldwide (Ref. 2.5-93). The results of these current studies do not differ greatly from prior parallel studies, but are generally more conservative. Therefore, these investigations can be used as a general guide for an expected value of acceleration (from an Intensity VI event) on which to anchor design response spectra. These studies show an expected acceleration level of about 6 to 7 percent of gravity as a result of a Intensity VI event. On the basis of these relationships, the design acceleration for the Susquehanna SES for structures founded on rock is conservatively selected as the required minimum of 10 percent q in accordance with 10CFR100, Appendix A. This level is used to anchor the design response spectra shown on Figure 2.5-27. For structures founded on soil, the NRC required that the SSE be increased 50 percent or to 0.15 g in order to accommodate any amplification of ground motion in the soil overlying the bedrock. It should be noted, however, that the maximum earthquake for the station is less than Intensity VI, which correlates with an acceleration of no more than 0.06 g. If this value were increased by 50% to accommodate amplification due to soils, the resulting SSE for structures founded on soil would not be more than 0.10 g. Thus, the selected value of 0.15 g provides a large margin of safety. The 0.15g value is applied at foundation level.

The duration of strong motion from the SSE is not expected to exceed 5 seconds (Ref. 2.5-95 and 2.5-96) and in all probability, would be considerably less at frequencies critical to design. Duration of motion from a larger, more distant event such as the Charleston, South Carolina event (X) would be relatively longer than that from the design event, but the low accelerations which are characteristics of long period motion from distant large events will be adequately enveloped by response spectra anchored at the minimum level of l0 percent g.

2.5.2.7 Operating Basis Earthquake (OBE)

On the basis of the historical seismicity described above wherein a maximum Intensity felt at the station from historical earthquakes was no larger than IV, an Operating Basis Earthquake (OBE) which would, during the life of the facility, generate a ground acceleration at the station no

higher than 5 percent g (l/2 SSE) has been selected. This level of acceleration will not be exceeded by the occurrence of even an Intensity V shock adjacent to the station or a recurrence of large, regional events at a distance. According to Trifunac and Brady (1975), a felt Intensity of V will result in acceleration levels below 4 percent g. Return periods for such ground motion at the station are of an extremely low order of probability, as evidenced by the fact that no local (Pennsylvania) shocks have been reported as felt at the station. Figure 2.5-28 is the design spectra anchored at the OBE level of 5 percent g. For structures founded on soil, an OBE of 0.08 g was used for design.

2.5.3 SURFACE FAULTING

Based on the data contained in Subsections 2.5.1 and 2.5.2 and the interpretations and conclusions therein, there is no capable fault (Appendix A, 10 CFR, Part 100) within at least five miles of the Susquehanna Steam Electric Station.

A detailed description of the lithologic, stratigraphic and structural conditions at the site and the surrounding region is contained in Subsection 2.5.1. All historical reported earthquakes within 50 miles of the site, and all earthquakes within 200 miles of the site with magnitudes (Richter) greater than 3.0 or MM intensities greater than III are detailed in Subsection 2.5.2.

The above referenced information clearly indicates that surface faulting is not of significance to the Susquehanna Steam Electric Station.

2.5.4 STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

2.5.4.1 Geologic Features

General. The upper bedrock at the site area includes the Middle Devonian Mahantango Formation. The upper part of the Mahantango is a dark gray siltstone, with bedding generally delineated by thin, consistent, light gray, fine-grained sandstone stringers. Beneath the upper member, the Mahantango is comprised of 120 to 150 ft of dark gray, hard calcareous siltstone, typically having bedding obscure to absent and displaying cleavage. This member which supports the power block structures is harder, more massive, and more resistant to erosion than the upper member.

The irregular bedrock surface underlying the site is the result of a combination of preglacial weathering and stream erosion, glacial scour, later erosion by glacial melt waters, and the varying resistance of the rock units to erosion. The bedrock is blanketed by till and glacial outwash which grades upward from a gravelly boulder zone to a surface layer of silty fine sands and sandy silt. The surface layer is believed to be reworked loess. The maximum thickness of overburden is around 40 ft in the southern half of the site, with bedrock occasionally cropping out at the surface. North of the east-west bedrock ridge situated just north of the reactors, the glacial deposits fill a valley eroded into bedrock to a depth exceeding 100 ft.

Structurally, the site is situated on the north limb of the Berwick Anticlinorium; its axis passes just south of the site. The anticlinorium trends east-northeast and plunges gently to the northeast. As with the regional picture, folding is the most characteristic feature of the site area. Minor faulting in the form of small bedding-plane slips and intraformational shear zones occur,

but they are of no significance to the site. They apparently developed during the Paleozoic (more than 200 million years ago) during the Appalachian orogeny. The zones are typically healed with calcite and quartz (Additional description of site geologic conditions is presented in Subsection 2.5.1.2).

All Seismic Category I plant structures except the spray pond, the Engineered Safeguard Service Water (ESSW) pumphouse and pipeline are founded on bedrock. The ESSW pipeline trench is excavated partly in soil and partly in rock. Most of the other major plant structures, including the cooling towers, are also founded on bedrock.

Site geologic and foundation conditions are entirely suitable for the construction and operation of the Susquehanna SES.

2.5.4.1.1 Areas of Potential Subsidence, Uplift, or Collapse

The potential for significant uplift or subsidence at the site, due to man's activities or geologic conditions such as regional warping, is negligible.

The shallowest carbonate rock that may be present beneath the site is the Onondaga Formation, above which occurs more than 2,000 ft of Middle Devonian shales and siltstones (Figure 2.5-14). At that depth the Onondaga Formation, if present, would not be expected to have a significant potential for subsidence or collapse even if it contained solution cavities. No coal beds are present beneath the site; the nearest coal measures are about 3-1/2 miles north of the site near Shickshinny. Rocks in the site area have no known potential for oil or gas production. The nearest oil or gas field is located 25 miles northeast of the site. Precise leveling surveys and other data in the literature provide no indication that the Site is in an area experiencing any abnormal regional warping, uplift, or subsidence. More detailed discussion of the potential for uplift or subsidence at the site is presented in Subsection 2.5.1.2.5.3.

2.5.4.1.2 Previous Loading History of the Foundation Materials

Bedrock at the site had been buried and deformed during the Appalachian orogeny (over 200 million years ago) with sufficient intensity to impose secondary cleavage in places and to mobilize calcite and to some extent quartz, resulting in a hard, indurated rock lacking the bedding-plane fissility normally associated with less well indurated silty shales and shaly siltstones. During Quaternary time, at least two ice lobes advanced over the site; the only direct effect this additional load might have on the bedrock at the site would be a tendency to scour loose or weathered rock from the rock-soil interface. Any pre-existing surficial deposits not removed by the glaciers would have been preconsolidated and thereby strengthened by ice loading.

Surficial material at the site consists of glacial drift largely, if not wholly, deposited by the Olean advance of the Wisconsin ice sheet. These deposits, which are described in Subsection 2.5.4.1, would be expected to have varying consolidation or preloading characteristics depending on local depositional history. Soils in the spray pond excavation (including the ESSW pumphouse excavation) and in the pipeline excavations leading to the spray pond are of particular interest because they support Seismic Category I facilities in these areas. Here geologic mapping shows that these soils consist of well stratified outwash sands and gravels,

together with poorly stratified to unstratified kame-like gravels (refer to geologic maps, Figures 2.5-15 and 2.5-18). They evidently were deposited during or subsequent stagnation of the final ice advance in the site area, since they are not overlain by a till blanket, nor do the strata show structural evidence of having been overridden by ice. Geologic evidence, therefore, indicates that the stratified surficial materials exposed in the excavations for the soil-supported Seismic Category I facilities are likely to be normally consolidated and not preloaded by ice during or subsequent to deposition.

As the overburden soils in the area were not preloaded, no correction is necessary for the influence of preloading on the SPT or on the relative density shown in Figure 2.5-32, and the cyclic shear stress ratio at failure as shown on Table 2.5-14. The lateral earth pressures shown on Figure 2.5-39 refer to pressures due to compacted backfill and are not influenced by the previous loading history of the site soils.

2.5.4.1.3 Structures and Zones of Weathering, Disturbance, or Weakness in Foundation Materials

Foundation materials consist of two basic types; namely, glacial till and outwash in the spray pond area, and siltstone or indurated slaty shale in the remainder of the principal plant foundations.

A geologic map of the foundation rock for the principal plant structures is presented on Figure 2.5-18. It shows joints, shears, attitudes of bedding and other features. All foundations shown were excavated so that structures are founded on firm bedrock, the Devonian Mahantango Formation. Geologic sections through the foundations are shown on Figure 2.5-19. A geologic map for the spray pond is presented on Figure 2.5-15.

The southwestern tip of the spray pond is cut into bedrock while the remainder is excavated in glacial materials. The thickness of the glacial deposits beneath the bottom of the spray pond ranges from zero at the rock contact to 93 ft at the eastern end of the pond. The foundation for the pumphouse structure, located at the southeastern corner of the pond, is underlain by 35 to 60 ft of glacial material. The water circulation pipelines, between the pumphouse and the plant, intersect bedrock at an elevation of 668 ft, approximately 260 ft southeast of the pumphouse.

The vicinity of the spray pond is situated over a glacial or preglacial, east-west trending bedrock valley as outlined by contours on top of bedrock (Figure 2.5-17). Total relief of the bedrock surface is about 130 ft. The valley is filled with dense, permeable gravelly and sandy glacial outwash and till deposits that attain a maximum thickness of about 110 ft in the spray pond area. They were deposited during the Olean substage (early Wisconsinan) of the Wisconsin glaciation which occurred at least 50,000 years ago, and there is a possibility that at least some of the bedrock erosion and overlying glacial deposits are the result of an earlier Illinoisan glaciation (refer to Subsection 2.5.1.2). In general, the deposits consist of a sequence of sand, gravel, and boulders overlain by sand and gravel, overlain in turn by silty sand. The entire sequence is highly variable in grain size distribution and sorting, and contains discontinuous pockets of similar materials. As a rule, grain size decreases and sorting increases toward the top of the sequence. The glacial materials in the deposit are noncalcareous; most of the rock particles consist of indurated sandstones. The origin and composition of the deposit are such that it is not susceptible to significant weathering or alteration.

In the power block and cooling tower foundations, the principal structural feature is a minor anticline, the axis of which trends about N85° E and was exposed in the radwaste and Unit 1 cooling tower foundations (Figure 2.5-18). South of this feature, bedding generally dips gently south with minor undulations; to the north, beds dip more steeply north. Bedding, which generally strikes N70 to 85°E, is obscure in the foundations; the foundation rock is quite massive and is not characterized by weak zones developed along bedding or cleavage planes. Where observed, the bedding planes as a rule are smooth and uncontorted with only minor undulations. In the turbine and reactor building foundations, the bedding dips 5 to 10 degrees to the south, while north of the circulating water pumphouse the dips are in places up to 5 to 8 degrees north to northeast, reflecting this minor undulatory variability of bedding. Small-scale folds a few feet in dimension occur but are not prevalent in the site area; one such small anticlinal fold was recognized at the north edge of the circulating water pumphouse excavation. Cleavage is variably developed, strikes generally parallel to the strike of bedding, and dips steeply south. Jointing in the rock excavated for foundations is fairly well developed. Figure 2.5-18 shows the principal joints encountered at foundation grade, which is at sufficient depth below the top of the rock to be in essentially unweathered material. Here joints are tight and either uncoated, or coated with calcite or a mixture of quartz and calcite. Few joints at foundation level contained significant iron staining; some iron-stained joints are mapped in the radwaste foundation area. Toward the surface these joints generally become more heavily iron-stained with greater degree of weathering, and calcite coatings tend to be leached out, resulting in open joints, in joints partly coated with quartz, or in clay-filled joints in the zone of weathering. The major joint set strikes east-northeast (N60-85°E), and dips vertically (within 15° of vertical). Other steeply dipping to vertical joint sets strike northwest to north-northwest, and north-south. Less steeply dipping joints generally have an east-northeast strike; one group dips gently northward at 10-18°, and another, in the southern part of the excavation, dips 50-60°SE. Some of the joint surfaces, particularly the low-angle joints, are slickensided. In addition to these principal joints, high-angle, discontinuous, white calcite and quartz-calcite veinlets are typically exposed locally throughout principal plant foundations.

A few minor shear planes, originally recognized in the cores obtained during the early phase of site exploration, were exposed during foundation excavation and are mapped on Figure 2.5-18. One shear plane, traced from the northeast corner of the Unit 1 reactor foundation northward to the Unit 1 cooling tower, was found to be oriented parallel to bedding and is denoted "bedding plane shear A" on Figure 2.5-18. The surfaces of the bedding plane shear are healed with 1/4 to 3/4 in. thick laminae of calcite, siltstone and some quartz. The calcite laminae are approximately 1/16 in. thick, alternating with thinner siltstone laminae. The entire exposed area of this bedding plane contains prominent slickensides trending N30° to 40°W, with a 6° to 7°SE plunge. Updip and closer to the top of the rock, the bedding plane contains a 1/2 to 1 in. wide, iron-stained zone, and it also shows extensive leaching of the minerals filling the shear. In places the adjacent rock is weathered to a granular sandy soil. The calcite which fills the bedding plane shows no sign of crushing. It should be emphasized that the weathering and staining on the bedding plane shear occurs only near the top of the rock where surface water and groundwater could penetrate along the plane; at foundation grade which is well below the weathered zone, the unweathered laminae have the properties of firm rock. In places the bedding plane shear is apparently not a prominent feature in the unweathered rock. For example, it was identified only as a slickensided surface with associated jointing in boring 105 and as horizontal jointing planes in boring 351 (shown in profile on Figure 2.5-19).

Foundation mapping reveals that the bedding plane shear is warped in conformance to the folding of bedding. The shear can be traced northward to the excavation for the Unit 1 cooling

tower (Figure 2.5-18). Measured attitudes of bedding show that the axis of the minor anticline described above occurs near the foundations for pedestals 6 and 7 of the cooling tower. South of pedestal 7, the bedding plane shear dips gently south; north of pedestal 6, the bedding plane shear dips gently north. Additional subsurface data from bore holes farther south and north strongly suggest that the configuration of the bedding plane closely follows the undulations in bedding (Figure 2.5-19). No evidence was found at the site to indicate that the shear plane substantially deviates from bedding planes.

During excavation, this bedding plane shear was traced updip to its intersection with the top of rock at a steep, glacially eroded contact. The eroded rock surface was continuous across the trace of the bedding plane, without displacement or offset (Figures 2.5-20a through 2.5-20g). Since the erosion of the rock surface would necessarily have occurred prior to the deposition of the overlying glacial deposits, which have been established as being more than 50,000 years old (refer to Subsection 2.5.1.2.2.1), this relationship shows that any displacement along bedding plane shear A occurred more than 50,000 years ago. In reality, the most probable age of the shearing is pre-Triassic or over 200 million years ago. This is indicated by regional relationships plus the fact that the shear plane is folded (A detailed presentation and analysis of the relationship between site and regional structure is presented at the end of Subsection 2.5.1.2.3.2).

A second bedding plane shear (Shear B), a few feet below and parallel to the first bedding plane shear, was exposed near the northwest corner of the Unit 1 turbine building foundation. It is similar in appearance but apparently more restricted in a real extent than the first. Two vertical, calcite-filled joints cut across this second bedding plane (Figures 2.5-20a through 2.5-20g). The calcite in these vertical joints is continuous across the bedding plane with no offset, showing that the joints were formed and the calcite was deposited in the joints, subsequent to development of the slickensides on the bedding plane.

The conclusion stated in the PSAR (p. 2.7-2) regarding the significance of these shear planes is still appropriate and deserves restatement: "Minor bedding plane slips at depth have been observed in the site area, both north and south of the interior ridge. Those slips have not experienced movements in more than 200 million years. A minor slip of this nature could be exposed in any large excavation anywhere in the area; however, it would not affect the structural design of the facilities."

All deformational features observed in rock at the site are geologically old and are not significant to plant structures. In unweathered rock, minor shears that do occur are tightly healed with calcite and quartz mineralization, and joints are likewise tight and unweathered. All foundations for plant structures designed to rest on sound rock were excavated to, or into, unweathered bedrock. No structurally weak zones were encountered in these foundations (Refer to Subsection 2.5.1.2.5.5).

Further description of depth of weathering and geologic structures at the site and in the foundations is presented in Subsections 2.5.1.2.3.2 and 2.5.1.2.5.6.

2.5.4.1.4 Unrelieved Residual Stresses in Bedrock

No indications were found during excavation and construction at the site of the presence of any significant stress in bedrock (refer to Subsection 2.5.1.2.5.8 for additional discussion).

2.5.4.1.5 Potential for Unstable or Hazardous Rock or Soil Conditions

Foundation rock at the site is a hard, indurated, unweathered siltstone, a member of the Middle Devonian Mahantango Formation. Similar materials underlie the site to a depth of at least 1,000 ft. This rock contains no unstable minerals and provides highly stable foundation conditions.

Soils at the site are glacial in origin, deposited mostly by flowing glacial meltwater, much under torrential conditions. The soil is noncalcareous. Most of the rock fragments consist of indurated sandstones. The origin and mineralogy of these soils is such that they present no hazardous conditions (refer to Subsection 2.5.1.2.5.7).

2.5.4.2 Properties of Subsurface Materials

A few of the safety-related principal plant structures are founded on soil. These structures consist of the Engineered Safeguard Service Water (ESSW) pumphouse, the spray pond, and portions of the Seismic Category I pipeline linking the reactor building to the spray pond and the diesel generator 'E' fuel tank. Most other plant structures are founded on rock. The location of these structures is shown on Figure 2.5-24; soil and rock foundations are identified on Figure 2.5-17A.

The static and dynamic engineering properties of the site bedrock and overburden soils were determined by field investigation and laboratory testing. The results of laboratory testing of the materials sampled from the project site are covered in two reports (Ref. 2.5-97 and 2.5-98).

A detailed study of the soil properties at the site of the spray pond and ESSW pumphouse is given in Subsection 2.5.5.

2.5.4.2.1 Properties of Foundation Rock

The Category I reactor buildings and diesel generator buildings, as well as the non-Category I turbine and radwaste buildings (see Figure 2.5-24) are founded on unweathered siltstone bedrock. The siltstone, a member of the Mahantango Formation of Devonian age, is hard and indurated, and in the foundations area is lithologically homogeneous with bedding generally not well defined, and lacking the bedding plane fissility usually associated with less well indurated shaly siltstones and silty shales. In places the rock exhibits cleavage, further evidence of its indurated nature.

In the area of the principal plant structures, bedrock bedding where observed generally dips gently (less than 10°) south; locally, such as north of the circulating water pumphouse, beds dip slightly north. At the north end of the radwaste building and the north side of the Unit 1 cooling tower, bedding dips more steeply north. The cleavage is steeply inclined to the south. Minor slickensided bedding plane shears and joint planes occur in the foundations as described in Subsections 2.5.4.1 and 2.5.1.2.3. All such shears beneath the principal plant foundations are fully healed with unweathered calcite and quartz mineralization and do not adversely affect the strength and competence of the foundation rock. Further evidence of the healed nature of these shears is furnished by the RQD values and core recovery rates in borings that penetrated bedding plane shear A (refer to Figure 2.5-18 and discussion in Subsection 2.5.4.1) at elevations below the bottom of the foundation of the principal plant structures, such as in

borings 302, 309, and 314. In all cases RQD values are above 35 percent through the shear plane; in most cases, RQD values exceed 80 or 90 percent and core recovery was close to 100 percent (Further information on foundation geologic conditions is presented in Subsection 2.5.4.1).

Typical values of unconfined compressive strength of unweathered siltstone underlying the principal plant foundations range from 3,650 to 16,000 psi (see Table 2.5-3). The modulus of deformation determined from these laboratory tests on core samples ranges from 3.1×10^6 to 9.4×10^6 psi. These values indicate strong, competent rock.

P-wave measurements were made by Dames and Moore in the laboratory on individual core specimens. The cores were from borings 303, 314, and 315 which are located, respectively, near the Unit 1 turbine building condensate pump pit at the center of the Unit 1 reactor, and at the center of the Unit 2 reactor. The average seismic P-wave velocity determined for 10 samples at or below foundation grade beneath power block structures is 13,236 fps. For three samples from boring 303 in the Unit 1 turbine building, the average Vp value is 14,272 fps, or approximately 14,000 fps. These determinations are listed in Tables 2.5-4 and 2.5-5.

Rock quality designation (RQD) measurements made by Dames and Moore on rock cores from below the foundation elevations in the reactor, turbine, radwaste, diesel generators, and circulating water pumphouse foundations exceed 80 percent (refer to boring logs, Ref. 2.5-97 and 2.5-121).

In the reactor area, cross-hole and down-hole measurements of in situ seismic velocities show high values. The measurements were made by Weston Geophysical Engineers, Inc., June 8 - August 6, 1971 using boreholes 105, 303, 307, 314, 315, and 316 (refer to Figure 2.5-29). Values obtained from the cross-hole array for the elevation interval 550-640 ft MSL are 16,000 fps for the P-wave velocity and 7500 fps for the S-wave velocity in the reactor area (design elevation of bottom of reactor foundations, 639 ft MSL). The results of the down-hole measurements yield values that are slightly lower, by a factor of about 15 percent; that is, a Vp value of about 14,000 fps and Vs of about 6,200 fps. These in site results are in good agreement with the laboratory determinations. Additional cross-hole and up-hole in situ seismic velocity measurements were made in the spray pond area (Ref. 2.5-99). Results of the cross-hole explorations at the site are further discussed in Subsections 2.5.4.2.2 and 2.5.4.4.

Plate load tests were carried out on sound rock near the center of the Units 1 and 2 reactor building excavation in the vicinity of boring 105 (refer to Figure 2.5-18). Plates 24, 13.5, and 8 in. in diameter were subjected to successively increasing total loadings of 7, 22, and 60 tons per square foot (tsf), respectively. A total deflection of .062 in. occurred when the 24 in. plate was loaded to a maximum of 7 tsf. An additional deflection of 0.036 in. was recorded on subsequent loading to 22 tsf, and another 0.036 in. of deflection on application of the 60 sf maximum load, producing a total settlement of 0.134 in. for the three-stage loading to 60 tsf. Recovery of the rock by elastic rebound upon release of these loads was substantial: 68, 75, and 80 percent repeatable elastic recovery of the total deflections were recorded after release of the 7, 22, and 60 tsf loadings, respectively. Additional deflections due to cyclic loading were small. Application of 14 cycles of load at 7, 15, and 30 tsf resulted in additional settlements of only 0.012, 0.003, and 0.002 in., respectively, over the corresponding single loadings. These results are consistent with the high modulus values and seismic velocities of the foundation rock, and indicate structurally strong, competent material for foundations in unweathered rock.

It is concluded from the engineering properties of the unweathered bedrock of the Mahantango Formation that the rock provides adequate support for the major plant structures under both static and dynamic conditions. Settlement of structures under static loading is insignificant. It consists of pseudo-elastic compression of the underlying rocks and occurs essentially upon load application. Moreover, the bedrock will undergo no loss of strength and will experience negligible additional settlement under earthquake loading.

A summary of the properties of the foundation rock is compiled in Table 2.5-5.

2.5.4.2.2 Properties of Foundation Soils

The results of detailed exploration of the soils in the spray pond area are given in Subsection 2.5.5. Only information on the properties of the pumphouse and diesel generator 'E' fuel tank foundation soils is given in this subsection.

The natural soils at the pumphouse and diesel generator 'E' fuel tank sites are normally consolidated and consist predominantly of sand, gravel, cobbles, and boulders. The soils are poorly stratified, starting as sand or sandy gravel at the surface and grading to mostly cobbles and boulders near bedrock. The depth of the soil deposit below foundation grade ranges from about 35 ft at the south end of the pumphouse to about 60 ft at the north end. About eight (8) feet and twenty (20) feet of sand, gravel and boulders are below the foundation grade of diesel generator 'E' fuel tank at north end and south end respectively. A subsurface cross-section through the pumphouse site is shown on Figure 2.5-30, cross-section D-D. The soils below the foundation level are predominantly sandy gravels with large amounts of cobbles and boulders. The properties of these sandy and gravelly soils are as follows:

a) Grain Size Distribution

Grain size distribution tests were made on most of the split spoon samples for classification purposes. Sieve and hydrometer analyses were performed according to ASTM Procedure D-422. The range of grain size curves is shown on Figure 2.5-31. The mean grain size (D50) of the gravelly soils, which are the predominant material below the pumphouse and diesel generator 'E' fuel tank was found to be in the range of 4.5 to 25.0 mm. Wherever the sand is present below the pumphouse, the D50 size is in the range of 0.14 to 3.0 mm.

b) Relative Density

Relative density data were derived from standard penetration test results using the Gibbs and Holtz procedure (Ref. 2.5-100). This procedure is valid for normally consolidated sands.

Values of relative density obtained in this way are summarized on Figure 2.5-32. A direct comparison of relative density from 'N' values given in Figure 2.5-32 and from undisturbed samples and/or in site density tests cannot be made because no relative density tests were made. The soil deposits are glacial in nature. The deposits are quite variable in particle size and sorting and contain discontinuous sand pockets and gravel pockets. Grain size in general increases with depth. At

the foundation level of the pumphouse and diesel generator 'E' fuel tank, the maximum sizes of the particles are in the range of 3 to 12 inches. Undisturbed tube samples could not be obtained in the gravelly soils. The gravel also will influence the results of in site density tests so that they may not represent the in site condition as a whole. The Standard Penetration resistance versus elevation is given on Figure 2.5-33. The 'N' values will be influenced by gravel. Because of this the higher blowcounts were not considered representative of site conditions. A value of N = 40 was selected for design. Of the 49 standard penetration tests made beneath the foundation level at the ESSW Pumphouse and 2 standard penetration tests beneath the diesel generator 'E' fuel tank, 45 exceeded 40 blows per foot. Of the 6 values that were less than 40 blows per foot only one was less than 30 blows per foot.

c) Static and Dynamic Shear Strength

Undisturbed sampling of gravelly soils was not possible. Therefore, shear strength testing was conducted only on the sands. The shear strength of the gravelly soils was then conservatively assumed to be equal to that of the sands.

The details of the testing procedures and selection of design strengths are given in Subsection 2.5.5. The effective angle of internal friction was selected from the test data to be 35° (Figure 2.5-34). The cyclic shear stress ratios at the two effective consolidation pressures 1.0 ksf and 6.0 ksf were determined to be 0.320 and 0.260, respectively, for 5 loading cycles (Figure 2.5-35, Subsection 2.5.5). A linear relationship was assumed in computing cyclic shear stress ratios at other effective consolidation pressures.

d) Shear Wave Velocity and Shear Moduli

Cross-hole shear wave velocity measurements were performed by Weston Geophysical Engineers, Inc. (Ref. 2.5-99). Compressional and shear wave velocities obtained from the measurements are given on Figure 2.5-36.

Shear moduli were computed from the values of shear wave velocity:

$$G = \frac{\gamma}{g} \quad V_s^2$$

Where:

G	=	shear modulus, psf
γ	=	unit weight, pcf
g	=	gravitational acceleration, ft/sec^2
V	=	shear wave velocity, fps

A discussion on how the shear modulus is influenced by the confining pressure, the strain amplitude, and the relative density is given in Subsection 2.5.5.2.

2.5.4.3 Exploration

The location of all field explorations is shown on the plot plan, Figure 2.5-22.

A total of approximately 250 exploratory borings was made in soil and rock at the site. Borings were logged in detail; boring logs are contained in Ref.s. 2.5-97, 2.5-98 and 2.5-99 and Appendix 2.5C. The soils were classified in accordance with the Unified Soil Classification System. Rock logs include RQD (rock quality designation) values. Coring in rock was performed using NX double-tubed coring equipment.

Drilling was conducted in late 1970 (100 and 200 series borings) to establish general geologic relationships over the site area and to determine general soil and rock conditions at the site. A more intensive program (300 series borings) was conducted in the Spring of 1971 to define foundation conditions in the principal plant structures area. Four 45-degree angle holes were drilled in the reactor area. Additional exploration drilling was necessary to locate the site for the Susquehanna River intake and discharge structures (700-800 series borings), to define soil and rock conditions at the spray pond and ESSW pumphouse site (1100 series and some 400 series borings), and to investigate foundation conditions for the cooling towers (borings B1 to B10) and the railroad spur and bridge over State Highway 11 (borings 417 to 455 and 929 to 940). An investigation program (borings 1 through 7) was conducted in 1983 to determine soil and rock conditions in the area of the deisel generator 'E' building. Boring logs are contained in Appendix 2.5C. Because of the safety-related (Category I) function of the spray pond and ESSW pumphouse, the exploration program for these facilities was comprehensive and included split spoon and undisturbed samples, laboratory testing, hydrologic surveys, permeability tests, and seismic cross-hole and up-hole surveys. Split spoon sample laboratory testing, hydrologic surveys, and permeability tests were also performed in the area of the diesel generator 'E' fuel tank. After completion of geologic borings, static water levels were measured in some of the borings drilled on the site. Perforated plastic pipes were installed in a number of the borings to allow collection of future water level data. These borings are denoted on the plot plan, Figure 2.5-22.

Forty-seven test pits were excavated by backhoe at selected locations to observe soil and rock conditions. Two north-south trenches totalling over 700 ft in length were excavated to obtain information on physical properties, structure, and variability of the near-surface materials at the site. Logs of the test pits and trenches are compiled in Appendix 2.5C.

A geologic map of the Category I and other principal plant foundations is presented on Figure 2.5-18. A geologic map of the excavation for the spray pond is shown on Figure 2.5-15. Geologic profiles are identified on Figures 2.5-18, 2.5-22, 2.5-30 and shown on Figures 2.5-19, 2.5-21A, 2.5-21B, 2.5-30, 2.5-40 and 2.5-56.

Photographs depicting significant features in the principal plant foundation excavations are shown on Figures 2.5-20a through 2.5-20g.

2.5.4.4 Geophysical Surveys

Seismic refraction profiles and cross-hole, up-hole and down-hole measurements were conducted at the site during the Fall of 1970, Summer of 1971, and Summer of 1974. The

seismic refraction lines totalled over 40,000 1f of coverage. They are identified on Figure 2.5-29. The refraction profiles collected in Appendix 2.5C.

As interpreted from refraction measurements, overburden at the site consists of a surficial layer of unconsolidated, unsaturated material up to 15 ft thick, constituting at least in part the soil horizon, underlain by more consolidated, partly or fully saturated till and compact outwash, which extend to the bedrock surface. Compressional (P-wave) velocity of the surficial material is typically about 1500 fps. Velocities in the lower till and outwash material generally range between 3,000 and 4,500 fps, although in some places velocities attain 6,000 fps (north-south baseline at boring 107).

The refraction survey obtained a persistent P-wave value of 12,000 to 14,000 fps for unweathered bedrock, which in many places is coincident with the top of rock. Frequently, however, lower velocities were recorded in a zone 0 to 20 ft thick near the top of rock. These lower compressional velocities are in the range of 6,000 to 9,000 fps, and are indicative of the zone of surficial weathering near the top of rock. At the site, material having a P-wave velocity of 4,000 to 6,000 fps may represent either dense soil or more thoroughly weathered or fractured bedrock; construction experience at the site indicates that here such material is generally correlative with dense soil.

Seismic cross-hole velocity measurements were performed in the reactor and spray pond areas, the principal sites of the Category I structures. Two arrays were employed in the spray pond area; namely, a north-south array across the location of the ESSW pumphouse, and an east-west array over the approximate location of lowest top of rock elevations in the spray pond. Both latter arrays provided data from which were calculated values for the dynamic moduli of the soil materials. In addition, down-hole measurements were made in the reactor area and up-hole measurements in the spray pond area. Figure 2.5-29 shows the borings that were used for the cross-hole arrays.

In the spray pond area, the seismic characteristics of the subsurface materials as measured in each array are similar. The material overlying bedrock has a P-wave velocity ranging from 4,200 to 4,800 fps and an S-wave velocity ranging from 1,600 to 1,900 fps. It is overlain by lower velocity material at about elevation 658 at the ESSW pumphouse location and at about elevation 643 farther west in the spray pond. At the ESSW pumphouse, this upper material has P-wave and S-wave velocity ranges of 2,300 to 2,400 fps and 1,300 to 1,350 fps, respectively, while farther west beneath the pond the materials between approximate elevations 643 and 673 have P-wave and S-wave velocity ranges of 3,000 to 3,300 fps and 1,450 to 1,500 fps, respectively. Table 2.5-6 summarizes the results of the seismic velocity measurements in the spray pond area and lists dynamic moduli computed from these data.

In the reactor area, cross-hole measurements were made on material above and below foundation grade. Above foundation grade the bedrock was weathered to a depth of about 10 ft below original top of rock. P-wave velocity of this weathered material was 7,600 fps and S-wave velocity, 3,600 fps. A P-wave velocity of 14,800 fps for the interval 640 to 660 ft MSL indicates the top of unweathered rock is at about 660 ft. At and below foundation grade in the reactor area high seismic velocities were recorded (Vp = 16,000 fps, Vs = 7,500 fps) indicating the presence of strong, competent foundation rock. Table 2.5-7 lists the results of the in situ cross-hole velocity measurements made in the reactor area; Table 2.5-5 lists the moduli values. Further discussion of the properties of the underlying soils and bedrock are given in Subsections 2.5.4.2 and 2.5.5.

2.5.4.5 Excavations and Backfill

2.5.4.5.1 Extent of Seismic Category I Excavations, Fills, and Slopes

Figure 2.5-37 shows the location and limits of excavations, fills, and backfills associated with Seismic Category I structures at the site. Typical foundation sections for seismic Category I structures are shown.

2.5.4.5.2 Excavation Methods and Dewatering

2.5.4.5.2.1 Excavations in Rock

All Seismic Category I rock foundations were carried to or well below unweathered bedrock. Rock foundations for the turbine and radwaste buildings, although they are not Seismic Category I structures, were prepared according to the same general procedures and criteria used in preparing the Seismic Category I rock foundations.

Excavation of rock proceeded by initial ripping of any weathered surficial rock material followed where necessary by line blasting and presplitting in holes drilled to provide slopes of 1 horizontal to 4 vertical. Essentially vertical slopes in unweathered rock proved stable throughout the duration of construction and no special protective measures were required. Weathered rock was cut on slopes of 1 horizontal to 2 vertical. In a few places, wire mesh was used for protection of higher weathered rock slopes that were exposed for extended periods.

The surface of the excavated foundation rock was scaled to remove loose debris and jetted with water or air to remove loose fragments and to prepare the surface for concrete. Before placement of structural concrete or concrete backfill to design elevation, all Seismic Category I foundations were inspected by an engineering geologist to verify the suitability of the rock and its proper surface preparation to receive concrete. All foundation rock bearing a Seismic Category I structure was geologically mapped (see Figure 2.5-18) or geologically cross sectioned (see Figures 2.5-19 and 2.5-30A).

Foundations for each of the cooling towers (nonseismic-Category I structures) consist of 40 individual pedestals supporting the columns and extended to bedrock. Excavation proceeded by cutting a ring trench and preparing for each pedestal a suitable surface in unweathered or partly weathered bedrock by ripping or blasting as necessary, followed by scaling and jetting.

During construction of principal plant structures founded on rock, excavations extended below the water table and some dewatering was required. Due to the low permeability of the rock, groundwater inflow was small. Dewatering was accomplished by surface drains and sumps.

2.5.4.5.2.2 Excavations in Soil

The excavation for the spray pond, ESSW Pumphouse and diesel generator 'E' fuel tank was predominantly in soils. Excavation proceeded initially by using large earth moving equipment, then finished by using more refined procedures. On completion of excavation, the surface layer of the natural soil formation was recompacted as follows:

a) For soils having not more than 12 percent passing the No. 200 sieve size, 80 percent relative density as determined by ASTM D2049

b) For all other soils, 95 percent of maximum dry density as determined by ASTM D1557

Test Results are included in Appendix 2.5C. The location of test specimens with respect to the spray pond is shown on Figure 2.5-59. A statistical analysis of the test results was made and is summarized on Figure 2.5-60. The required compaction was met or exceeded.

A protective concrete mat was immediately placed over the compacted soil under the ESSW Pumphouse and a minimum of 5 in. thick reinforced concrete liner placed over the entire spray pond area.

All temporary slopes in soil were formed at a maximum slope of 1 1/2 horizontal to 1 vertical. The temporary slopes in the vicinity of the ESSW Pumphouse were protected with a 3 in. layer of concrete to maintain the natural soil formation intact. All permanent slopes in soil were formed at a slope of 3 horizontal to 1 vertical.

The excavation for the Seismic Category 1 pipelines in soil was carried out similarly. All slopes were cut at a maximum of 1 1/2 horizontal to 1 vertical. The minimum clearances were 1 ft beneath the pipe and 2 ft to the sides.

The excavation for diesel generator 'E' building was carried to unweathered bedrock by using soldier beams and laggings. All timber laggings were treated with preservative by pressure process. The soldier beams and laggings were left in place. The disturbed soils adjacent to the soldier beams and laggings were densified by compaction grouting. The results of compaction grouting were verified by standard penetration tests. The results of standard penetration tests indicate that the blow count numbers are equal to or exceed those of original soils.

The excavation for diesel generator 'E' fuel tank was carried out to open cut. All slopes were cut at a maximum of 1 1/2 horizontal to 1 vertical.

2.5.4.5.3 Backfill and Compaction

Generally, the excavated area, for a minimum distance of 10-ft surrounding the major structures, was backfilled with a non-corrosive lean mix concrete known as sand-cement-flyash backfill. A minimal amount of backfilling has taken place using granular backfill, with the exception of the spray pond and vicinity addressed later in this section.

The excavated area for the diesel generator 'E' fuel tank was backfilled with sand-cement-flyash to two (2) feet below finished grade.

The Seismic Category I pipelines were generally backfilled with the sand-cement-flyash; otherwise granular material was used.

All category I structures and a portion of the ESSW pumphouse (below approximately elevation 676 feet) was backfilled with sand-cement-flyash. The backfill was placed a minimum of 10 feet

in width all around the building structures. There is no concern with differential settlement at the interface between structure-supported and ground-supported parts of pipelines and conduits.

Buried Seismic Category I electrical ductbanks are composed of reinforced concrete encasements around plastic or metal ducting; the concrete encasement being cast directly against the excavated grade. Granular or sand-cement-flyash backfill was used the same as for buried pipes. The properties of these respective backfills were as follows:

a) Sand-Cement-Flyash

Weight	-	110 lb/cu ft minimum
Slump	-	3 in. minimum
Slump	-	6 in. maximum
Strength	-	40 psi minimum at 28 days

Sand-cement-flyash used on the Susquehanna SES project has been obtained from two sources, namely as follows:

a) Metropolitan Edison, Portland Plant, Reading, Pennsylvania
b) Michigan Ash Sales, Essexville, Michigan.

The sand-cement-flyash backfill that was mixed on-site used flyash furnished by Michigan Ash Sales. The sand-cement-flyash backfill that was furnished by an off-site supplier (Galli Ready Mixed Concrete) used flyash from Metropolitan Edison.

During January 1979, the pH value of each source of flyash and the pH value of the fresh sand-cement-flyash mix was investigated. The results are as follows:

a) Metropolitan Edison Flyash

pH flyash = 4.36 to 4.42
pH sand-cement-flyash mixture = 11.7 to 12.8

b) Michigan Ash Sales Flyash

pH flyash = 4.2 to 8.2
pH sand-cement-flyash mixture = 12.2

The above tests adequately demonstrated that the sand-cement-flyash backfill used on the Susquehanna SES project is not corrosive.

b) Granular

Granular backfill was well-graded, sound, dense, and durable material. It consisted of sand, gravel or crushed rock and did not contain any topsoil, humus, brush, roots, peat, sod, cinders, shale, rubbish or other perishable materials, or portions of clay, waste concrete, trash, or frozen material. No more than five percent by weight passed the No. 200 sieve. The maximum size of the material was 4 in. in confined areas where hand tamping was required and 6 in. in other areas.

The placement specification of these respective backfills was as follows:

a) Sand-Cement-Flyash

Sand-cement-flyash backfill was either mixed at the batch plant or obtained from an offsite source, conveyed to the point of placement by truck, and placed in lifts not exceeding 30 in. in height. The maximum rate of pour did not exceed 4 ft/hr. It was vibrated in place with approved equipment. It was protected from freezing temperatures for a minimum of 3 days.

b) Granular

Granular backfill was placed in maximum 8 in. loose horizontal layers, moisture conditioned, and compacted to at least 80 percent relative density as determined by ASTM D2049.

Backfill material within 2 ft of structures and in areas where large construction equipment could not be used or where there was a danger of damage to structures was compacted to the specified density by hand operated equipment.

Small areas resulting from dental excavation beneath the spray pond concrete liner received a shallow leveling course. The material and placement specification for this type of fill (arbitrarily designated Fill Type A) was as follows:

Fill Type A, Material

The maximum size of this material was 4 inches and no more than 5 percent by dry weight passed the No. 200 sieve.

Fill Type A, Placement

Fill Type A was placed in maximum 6 inch uncompacted layers, moisture conditioned, and compacted to at least 80 percent relative density as determined by ASTM D2049.

The area to the south and south-east of the spray pond was filled in a controlled manner. The material and placement specification for this type of fill (arbitrarily designated Fill Type 'B') was as follows:

Fill Type B, Material

The maximum size of this material was 12 inches and no more than 35 percent by dry weight passed the No. 200 sieve.

Fill Type B, Placement

Fill Type B was placed in a 15 inch maximum uncompacted layer thickness, moisture conditioned, and compacted to satisfy both of the following requirements:

a) At least 80% relative density as determined by ASTM D2049 for material having not more than 12% passing the No. 200 sieve or 90% of maximum dry density as determined by ASTM D1557 for all other material.

b) Irrespective of the compacting effort required to satisfy part a) above, the fill was compacted in one of the following manners as a minimum effort:

   i) Using a crawler tractor having a weight at least equal to that of a D8 Caterpillar tractor with bulldozer blade. Each track overlapped the preceding track by not less than four inches. When the tractor has made one entire coverage of an area in this manner, it was considered to have made one pass. Each fill lift was compacted with four passes.

   ii) Using a vibratory roller of minimum weight 20,000 pounds having a rolled width of approximately 60 minches. The roller had a vibrator frequency range of between 1100 and 1600 vibrations per minute and had a minimum vibratory dynamic force of 40,000 pounds. The roller speed did not exceed 3 mph and each track overlapped the preceding one by a least 4 inches. When the roller had made one entire coverage of an area in this manner, it was considered to have made one pass. Each fill lift was compacted with four complete passes.

   iii) Using a hand controlled vibratory compactor in locations inaccessible by tractor or vibratory compactors was on the basis of the demonstrated ability of the compactor to compact the material to the same density as the continuous backfill.

Test results are included in Appendix 2.5C. The location of test specimens with respect to the spray pond is shown on Figure 2.5-59. A statistical analysis of the test results was made and is summarized on Figure 2.5-60. The required compaction was met or exceeded.

To compute the lateral pressures acting on subterranean walls, all backfill was conservatively assumed to be granular. The static and dynamic engineering properties of this granular backfill was assumed as follows:

Bulk unit weight,	=	135 pcf
Saturated unit weight,	=	140 pcf
Coefficient active earth pressure, K_A	=	0.30
Coefficient earth pressure "at-rest", K_o	=	0.70

The computation of static and dynamic lateral soil pressures acting on subterranean walls is addressed in Subsection 2.5.4.10.2.

A statistical analysis for the sand-cement-flyash backfill is shown in Fig. 2.5-61.

Figure 2.5-61 shows the distribution for the majority of test reports falls within the specification requirement of 40 PSI minimum at 28 days and 100 psi maximum at 90 days. There are

however a sufficient quantity of tests that fall outside these limits and therefore justification is required. The reasons for acceptability are as follows:

1) Slump - The slump requirements of 3 inch minimum and 6 inch maximum were for reasons of workability during field pouring operations and as one of the measurements to ensure that a sufficient compressive strength would be obtained. Therefore a deviation from the specification requirement does not constitute unacceptability, since the only basis would be an altered compressive strength and these items have their own acceptability limits.

2) 90 day compressive strength - The requirement for the 100 psi (150 psi maximum strength for the diesel generator 'E' facility) maximum strength was a convenience item only. The purpose for this upper limit was to facilitate ease of excavation in an area previously backfilled with sand-cement-flyash material. A deviation from the specification requirement in this area in no way limits the intended use of the material.

3) 28 day compressive strength - the requirement for minimum compressive strength was chosen such that the compressive strength would be in the same range as the bearing value of compacted granular backfill. The 40 psi strength converts to 5,760 lb./ft^2 whereas compacted granular backfill has bearing values on the order of 3,000 to 6,000 lb./ft^2. However, on SSES the sand-cement-flyash material has not been used for fill beneath any structure. Therefore, the higher bearing value is not required. The sand-cement-flyash backfill has been used only as general backfill for pipes, conduits, and along the sides of buildings. It has therefore been determined that the 20 psi (2,880 lb/ft^2) minimum value that may be found in some areas is completely adequate to satisfy the conditions for which the fill is being used.

   Strength and Slump tests were conducted every 100 cu. yds, placed or every placement, whichever occurred first.

   Slump and Compressive strength samples of sand-cement flyash were taken at the Batch Plant, or in the case of samples taken from an offsite source, from the concrete mixer agitator truck.

2.5.4.5.4 Bedding Material for Seismic Category I Pipes and Electrical Duct Banks

The bedding material was sand-cement-flyash as defined in Section 2.5.4.5.3 of the FSAR.

The excavation was made to original ground or in sand-cement-flyash backfill to required bedding subgrade. The bedding subgrade was inspected and verified to be sound and dense meeting visual requirements for backfill adequate for support of bedding material, thus meeting specification intent. The subgrade was also inspected for unsuitable material such as water, frozen, organic or deleterious material. Such material, when found, was removed.

The sand-cement-flyash bedding material was either mixed at the batch plant or obtained from an approved offsite source. The sand-cement-flyash was then placed in lifts not exceeding 30 inches in height nor 4 feet per hour. For pipes the pour was brought to the pipe spring line and

was allowed to set. For duct banks the bedding was not placed until the duct bank concrete reached the required strength. Sand-cement-flyash was then poured to the top of the duct bank and allowed to set.

Analysis of the relevant field tests for bedding material is included in the summary given in Figure 2.5-61.

2.5.4.6 Groundwater Conditions

Special measures for control of groundwater levels beneath Seismic Category I plant structures founded on rock are not required. However, control of groundwater levels and seepage is needed at the spray pond; discussion of design criteria for stability of the spray pond is presented in Subsection 2.5.5.

Periodic water level readings were obtained in the vicinity of the principal plant (power block) structures between December 1970 and August 1972. Groundwater fluctuations ranged from 1.5 ft in drill holes 209, 311, to 6.2 ft in drill hole 213.

The maximum groundwater level measured in the plant structures area during this preconstruction period ranged from approximately 690 ft at the west edge of the site of the turbine building, to about 655 ft at the east edge of the site of the reactor buildings (refer to Figure 2.5-55). These levels were obviously influenced by the topographic high of 749 ft just west of the site of the power block structures. However, subsequent excavation and grading in these areas preclude water levels from rising to this height in the future.

During construction, the area just west of the power block structures was graded to elevation 710 ft or less. Excavations for the foundations of the principal plant structures extended below the water table and some minor dewatering was required. Due to the low permeability of the rock, groundwater inflow was small and was confined to seepage from fractures. Dewatering was accomplished by pumping from low areas and sumps. Where seeps were noted issuing from fractures in the rock, holes were drilled into the fractures and pipes caulked in the holes to control water while the mudmat was placed. In the foundation for the reactor building (elevation 639 ft) and in the turbine condensate pump pit (at elevation 635 ft), hydrostatic pressure caused lifting of small areas of the 3 inch thick concrete mudmat that had been placed over the impervious membrane. Approximately 20 relief wells drilled through the mudmat released the pressure and allowed the mat to settle back to its original position. The weight of the structural concrete slab subsequently placed on this mudmat was more than sufficient to resist any uplift pressures.

The highest seeps noted in the foundation rock during construction were at elevation 642 ft in the radwaste building excavation and at about the same elevation in the pipe trench in the southern part of the Unit 2 turbine building. Some seeps were also noted in the foundation rock for the reactor buildings at elevation 639 ft and in sumps below this. To the west of the turbine building in the circulating water pumphouse excavation, water was noted to enter the excavation to an elevation of approximately 660 ft. Hydrostatic lifting (described above) of the impervious membrane did not occur at foundation elevations above 640 ft.

Excavation for diesel generator 'E' building extended below the water table and some minor dewatering was required. The groundwater which seeped into the excavation area was diverted to a sump at a low point and was removed by pumping.

Additional information with regard to groundwater monitoring and water table fluctuations in the principal plant structures area is provided in Subsection 2.4.13 and Tables 2.4-31 and 2.4-32.

At the spray pond, water level information taken between July 29, 1974 and August 4, 1975, and from January through March 1977, indicate a minimum water level fluctuation of 4.0 ft recorded at observation wells 1111 and 1113, and a maximum fluctuation of 7.0 ft in 1115. Additional discussion of groundwater fluctuations in the spray pond area can be found in Subsection 2.5.5. Because groundwater levels at the pond will be higher than the maximum projected flood elevation (refer to Figure 2.5-38 and Subsection 2.4.3, respectively), flooding conditions will not significantly affect the groundwater levels.

Local wells within two miles of the plant site were inventoried and the information is given in Table 2.4-22.

Groundwater flows away from the principal plant structures area to the north, east, and south. However, the predominant direction of flow is to the east and southeast at gradients of 0.05 and 0.06, respectively. The flow rate in bedrock is estimated to be less than 1 ft per day as discussed in Subsection 2.4.13. Groundwater contours at the site are shown on Figure 2.5-38.

Permeability of the intact bedrock at the site is less than 1 ft/year. The average permeability of the glacial materials at the spray pond is 2,000 ft/year; however, this value has been considerably exceeded in some tests. For a complete description of permeability at the spray pond and plant structures areas, consult Subsections 2.5.5 and 2.4.13, respectively. Measured permeability values may be found in Tables 2.4-33 and 2.4-34.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

2.5.4.7.1 Response of Rock to Dynamic Loading

Rock at the site would be unaffected by dynamic loading from earthquakes. During historical time, no Pennsylvania earthquakes have been felt at the site. Approximately 14 earthquakes originating outside Pennsylvania could have been felt at the site, but with a probable maximum intensity of only IV on the Modified Mercalli Scale. Ground motion at this intensity would have had no effect on the site.

The compressional and shear wave velocities of sound, unweathered foundation rock in the reactor area (V_p = 14,000 to 16,000 fps; V = 6,200 to 7,500 fps) indicate that the rock possesses a high rigidity and provides effective resistance against dynamic loads for all structures founded upon it (refer to Table 2.5-5). Such rock will not be subject to any loss of strength under earthquake loadings.

2.5.4.7.2 Response of Soil to Dynamic Loading

The analysis of earthquake-induced soil strain and settlement of the spray pond and ESSW pumphouse are given in Subsection 2.5.5. Evaluation of potential soil settlement under the diesel generator 'E' fuel oil storage tank is provided in Reference 2.5-121. If the sands at the site behave like dry sand during an earthquake, the settlement will be less than 0.05 in. If the sand deposits are saturated and excess pore pressures develop, they will reconsolidate following the earthquake and settlements up to 1.2 in. at the east end of the pond and up to 1.0 in. at the ESSW pumphouse may be expected. Settlement under the diesel generator 'E' fuel tank will be minor and will take place as soon as the loads are applied. There will be no long term settlement.

The bearing capacity of the pumphouse mat footing was evaluated by the following equation (Ref. 2.5-115):

$$q_d' = 1/2\, B\gamma N\gamma + D_f\left(N_q - 1\right)$$

where:

q_d'	=	ultimate bearing capacity
B	=	width of the footing
γ	=	unit weight of the soil
D_f	=	depth of surcharge
$N\gamma$, Nq	=	bearing capacity factors

This equation was derived for the static condition; however, a conservative evaluation of the bearing capacity for the dynamic condition can be made by assuming that, during dynamic loading, the footing has an effective width equal to 1/3 of the actual footing (Ref. 2.5-115). Substituting all values given in Subsection 2.5.4.10.2 into the equation but using B=21.3 ft instead of 64 ft, the ultimate bearing capacity was calculated to be 52 kips/sq ft. The corresponding factor of safety against bearing failure is 17.

2.5.4.7.3 Soil Structure Interaction

Soil structure interaction has been addressed in Subsection 3.7.2.4. The analysis and design of buried pipelines has been addressed in Subsection 3.7.3.12.

2.5.4.8 Liquefaction Potential

For the soil supported spray pond, ESSW pumphouse, diesel generator 'E' fuel oil storage tank and Seismic Category I pipelines, the liquefaction potential was evaluated. The soil underneath these structures is predominantly sand, gravel, cobbles, and boulders.

The liquefaction potential of the soils beneath the spray pond and the ESSW pumphouse is discussed in detail in Subsection 2.5.5. The minimum factor of safety against liquefaction for these structures was found to be 1.26, which is larger than the minimum acceptable factor of safety of 1.20.

The soil supported diesel generator 'E' fuel oil storage tank and Seismic Category I pipelines are underlain by the same glacial deposits as the spray pond area and the maximum predicted water level below the pipelines is lower than that under the pond. Hence, liquefaction potential of the soils beneath the diesel generator 'E' fuel oil storage tank and seismic Category I pipelines is no greater than that of soils beneath the spray pond.

2.5.4.9 Earthquake Design Bases

The design bases for the SSE and OBE are addressed in Subsections 2.5.2.6 and 2.5.2.7.

2.5.4.10 Static Stability

2.5.4.10.1 Static Stability of Safety-Related Structures Supported on Rock

The reactor buildings, control structure, and the diesel generator buildings, all of which are Seismic Category I structures, are founded on sound, unweathered siltstone bedrock. The Seismic Category I pipelines linking the reactor buildings with the spray pond are trenched partly in soil and partly in bedrock.

The strength of the unweathered bedrock amply accommodates the loads of the plant providing highly stable foundation conditions. As measured in the Seismic Category I reactor area, compressional velocities are in the range of 14,000 to 16,000 fps; shear wave velocity ranges between 6,200 and 7,500 fps. Static deformational moduli as measured on rock cores vary between 3.1 to 9.4×10^6 psi (refer to Table 2.5-3). Measurements of unconfined compressive strength of unweathered foundation rock from the vicinity of the principal plant structures were between 3,650 and 16,000 psi (Table 2.5-3). Static properties of the foundation rock are summarized in Table 2.5-5. Loads induced by the plant structures are less than the allowable bearing pressure of the rock and far below the ultimate bearing capacity. The structural loads will produce no significant total or differential settlement of the foundations.

Safety-related structures founded on rock were designed for a hydrostatic groundwater loading caused by a maximum groundwater level of 665 ft. This is higher than the expected maximum water level, as discussed in Subsection 2.4.13.

2.5.4.10.2 Static Stability of Safety-Related Structures Supported on Soil

The mat footing of the ESSW pumphouse is 112 ft long, 64 ft wide, and 3 ft thick. The total dead and live loads are 20,000 kips and 2,100 kips, respectively. The corresponding unit pressures are 2.80 ksf and 0.30 ksf, respectively. The bottom of the mat is at elevation 657 ft.

The ultimate bearing capacity of the mat foundation was found to be 158 kips/sq ft. The factor of safety was computed to be 51, which indicates no danger in overstressing the supporting

granular soil. Therefore, the allowable bearing pressure and settlement of the mat footing were evaluated by the method of limiting settlements suggested by Peck, Hanson, and Thornburn (Ref. 2.5-116). The allowable bearing pressure for a maximum settlement not to exceed 2 in. was computed by the formula:

$$q_a = 0.22\, C_n\, C_w\, N$$

Where:

q_a = allowable bearing pressures, tsf

N = number of blows per foot in the standard penetration test

C_n ,C_w = correction factors for "N", for the effects of overburden pressure and location of groundwater surface

A conservative N value of 40 was selected to represent the soils below the mat foundation (Elevation 657 ft, Figure 2.5-38). The Standard Penetration Tests below the foundation level were made at an average overburden pressure of about 6,000 psf (Figure 2.5-39); the corresponding correction factor C was obtained from Figure 19.6 of Ref. 2.5-115 to be 0.63. Assuming that the groundwater surface is at 7 ft below the mat and no surcharge, the correction factor C_w was computed to be 0.55 by equation 19.4 of Ref. 2.5-115.

The allowable bearing pressure was computed to be 6.0 kips/sq ft based on the values of N, C_n, and C_w given above. At this bearing pressure, the settlement of the mat foundation should be less than 2 in. and the differential settlement should be less than 3/4 in. Therefore, by proportion, for a design total pressure of 3.1 kips/sq ft, the corresponding maximum and differential settlements would be less than 1 in. and ½ in., respectively. Settlement in sand and gravel deposits occurs almost simultaneously with the application of load. Since more than 80 percent of the total load is dead load, then less than 0.2 in. of settlement is expected after the completion of the construction.

The same equations and procedures can be applied to compute the ultimate bearing capacity of the foundation soils and the allowable bearing pressure for a maximum settlement not to exceed 2 inches.

The foundation mat for the diesel generator 'E' fuel tank is 17 feet wide, 57 feet long and 5 feet thick. The total dead and live loads are 111.4 kips and 684.8 kips respectively. The corresponding unit pressures are 0.12 ksf and 0.71 ksf respectively. The bottom of the foundation mat is at elevation 645.0 feet.

The ultimate bearing capacity of the foundation soils was found to be 42.0 ksf. The factor of safety against shear failure was computed to be 50 which indicates that there is no danger of shear failure.

The allowable bearing pressure was found to be 12.0 ksf for a maximum settlement not to exceed 2 inches. By proportion, the maximum and differential settlement corresponding to a design total pressure of 0.83 ksf would be less than 1/8 in. and 1/16 in. respectively.

The structural stability of the ESSW pumphouse is discussed in Subsection 3.8.4 and 3.8.5.

The sustained load from the spray pond is less than the weight of overburden removed; therefore, there is an adequate factor of safety against overstressing the underlying soil. Soil rebound during excavation in granular soils of the type found at the spray pond is insignificant.

The maximum predicted elevation of the water table is below the base of the spray pond and ESSW pumphouse; therefore, hydrostatic water loadings were not considered in the design of these structures. A full discussion of the water table in this vicinity is in Subsection 2.5.5.

The lateral earth pressure acting on subterranean walls of Seismic Category I structures was computed assuming granular backfill having the properties stated in Subsection 2.5.4.5.3. The coefficient of earth pressure "at-rest" was used. Additionally, the walls were designed for surcharge loadings and dynamic soil pressures as appropriate. The typical pressure diagrams and combinations are shown on Figure 2.5-39.

Water levels in the spray pond area are discussed in Subsection 2.5.5.1.2. Contours of the groundwater table in the spray pond area are shown on Figure 2.5-38. Profiles of measured and projected profiles of the groundwater table beneath the spray pond are shown on Figure 2.5-40.

2.5.4.11 Design Criteria

2.5.4.11.1 Design Criteria of Safety-Related Structures on Rock

The plant structures founded on rock are designed for a maximum acceleration of 0.10g from an occurrence of the SSE event. From consideration of its engineering properties, it is evident that the foundation rock will not be measurably affected by seismic loadings, and negligible additional foundation settlement will accompany these maximum potential dynamic loads. The maximum contemplated total static and dynamic loads of 40 tsf are only a fraction of the bearing capacity of the rock, thus ensuring an ample margin of safety.

2.5.4.11.2 Design Criteria of Safety-Related Structures on Soil

The spray pond slopes are designed for a maximum acceleration of 0.15g from an occurrence of the SSE event at the site. The spray pond riser pipe columns, the Seismic Category 1 buried pipes, and the ESSW pumphouse are designed for a maximum acceleration of 0.15g from an occurrence of the SSE event at the site.

The allowable bearing pressure under both static and dynamic conditions satisfies these conditions:

a) Sustained dead load plus live load with a minimum factor of safety of 3
b) Sustained dead load plus maximum live load with a minimum factor of safety of 2
c) Sustained dead load plus live load keeping settlement within tolerable limits.

At the spray pond, a liner has been designed to restrict the seepage rate from the pond in order to limit buildup of a groundwater mound in the glacial materials underlying the pond. The pond has been designed for a maximum groundwater elevation of 665 ft. Detailed description of

design criteria for control of groundwater levels and seepage at the spray pond and the stability of the pond are in Subsection 2.5.5.

2.5.4.12 Techniques to Improve Subsurface Conditions

2.5.4.12.1 Foundations in Rock

No special treatment was required to improve foundation conditions beneath the Seismic Category I structures bearing on rock. During construction, high loads were carried by the gantry crane rails, one of which was adjacent to the top of the temporary vertical slope on the east side of the reactor building excavation. As a precautionary measure to ensure stability of this slope during construction, tensioned rock bolts were installed in the slope. One large pothole was encountered in the Unit 1 turbine building area, necessitating over excavation of some 23 ft below design base elevation. The resulting hole, which was in fresh, unweathered rock, was backfilled with 574 cubic yards of concrete (f^1_c = 2,000 psi) to foundation grade.

2.5.4.12.2 Foundations in Soil

No improvement of the natural soil formation at this site was required.

2.5.4.13 Subsurface Instrumentation

2.5.4.13.1 Instrumentation for Rock Foundations

Since settlements are negligible for the safety-related facilities founded on rock (refer to Subsections 2.5.4.7 and 2.5.4.10), no instrumentation to monitor such settlements is necessary.

2.5.4.13.2 Instrumentation for Soil Foundations

The foundation design for the ESSW pumphouse was based on measured soil parameters obtained by field and laboratory testing. The actual settlement should not exceed tolerable limits for the structure and its piping connections. A systematic monitoring program was therefore instituted to study the settlement performance of the structure. The following instrumentation program was carried out:

a) Permanent Bench Marks: Two permanent bench marks were installed as reference points for measurements.

b) Settlement Pins: A total of six settlement pins were cast into the structural mat and 5 settlement pins were installed in the pumphouse floor at Elev. 685'-6". Details are shown on Figure 2.5-41 and Figure 2.5-62. A survey reading was taken on each pin at approximately monthly intervals. The total settlement and differential settlement of the mat foundation was therefore deduced.

   Survey readings will be taken on the five pins located at ESSW pumphouse floor elevation 685'-6". These readings will be recorded until 1983. This will give

recordings of at least 4 years from the pumphouse completion. In addition to the a survey of these pins shall be conducted after any of the following events:

1) Earthquake
2) 100 year storm
3) Major leakage or break in a water pipe in the pumphouse fill area.

Initial results are shown on Table 2.5-8. Subsequent results are retained in the appropriate SSES records file. All results are within the projections described in FSAR Section 2.5.4.10.2.

2.5.4.14 Construction Notes

During the construction phase excavated material was temporarily stored at the spray pond area which was used for approximately two years as a laydown area for construction materials. This material was then excavated under specification C36 and since the material was unable to meet the requirements of fill type "A" or "B" as outlined in Specification C36 it was removed from the spray pond area. Such removal was ensured as the excavation line for the pond was below the original ground contour.

During construction of the spray pond liner, cracking was observed in several areas, the most extensive being the area along the southwest edge of the spray pond. The remainder of the cracking was distributed between areas just north and south of the spillway and two small areas located along the north and south central portions. The cracks along the southwest and spillway area were approximately 50 feet in length while the cracks along the central area averaged 7 to 10 feet in length. The cracks in all areas ranged from 1/2" to 1 1/2" in depth. The cracks located above elevation 676'-6" and the cracks wider than 1/16" below elevation 676'-6' were "V" grooved to a depth of 1/2" and sealed with Horn Flex L sealant A manufactured by W. R. Grace Co. Cracks below elevation 676'-6" and having widths smaller than 1/16" were left as is.

The hairline cracking which is predominant in the southwest section of the pond is coincident with the concrete liner being placed directly on bedrock. Since the liner in contact with the bedrock is more restrained during the initial concrete curing and shrinkage period it has been determined that these shrinkage forces were the major cause for cracks in this area. In addition two slabs in this area were displaced by hydrostatic uplifting forces causing some additional cracking. This uplift occurred during the construction phase when the pond was empty of water. The hydrostatic uplift pressure was relieved by means of 2-inch diameter core drills through the liner. These relief holes were then filled with grout just prior to filling the pond with water.

A slab located south of the spray pond spillway was displaced by means of frost heave and resulted in cracking. This action also took place during the construction phase when the pond was empty of water. The displaced section was removed and repaired in accordance with section 7.14 of specification C36. The cracks were repaired as described above.

Uplift due to hydrostatic pressure up to design elevation and frost heave are of no design concern when the pond is filled with water as required during plant operation.

In areas where the liner was placed on soil very little hairline cracking has occurred. As a result there has been no indication of cracks being caused by soil settlement.

2.5.5 STABILITY OF SLOPES

Natural slopes at the site are depicted in the site topographic map, Figure 2.4-1. Final plant grades are shown on Figure 2.5-24.

Few rock slopes are present at the site that need to be considered with respect to possible adverse effects on the safety-related operation of the plant. Within the area impounded by the spray pond, bedrock forms a portion of the southwest slope, cut on a gradient of 3 horizontal to 1 vertical. North of the spray pond, a natural slope formed on Trimmers Rock sandstone rises at a maximum gradient of 2 horizontal to 1 vertical to a height of approximately 380 ft above the bottom of the pond (refer to Figure 2.5-56). As discussed in Subsection 2.5.5.2.3.1, such rock slopes would present no significant hazard to safety related plant structures.

The soil slopes to be considered are those forming and surrounding the spray pond.

2.5.5.1 Slope Characteristics

The slopes analyzed include the cut slopes of the spray pond and the slopes of the railroad embankment adjacent to the spray pond. The failure of either slope could affect the normal operation of the spray pond. The stability of these slopes is also dependent upon the stability of the spray pond itself. Therefore, the safety analyses of the slopes and the stability of the spray pond are investigated and discussed together in this section.

The cut slopes of the spray pond consist of two portions separated by a 20 ft service road; both were made at 3 horizontal to 1 vertical (Figures 2.5-42 and Dwg. C-63, Sh. 1). The lower portion is a 17.5 ft slope between the service road (Elevation 685.5) and the pond bottom (Elevation 668). The upper portion extended from the service road to daylight, the height of the slopes varies from 0 ft at the east end to about 40 ft at the west end of the pond. Except for a few cut slopes that are made in bedrock, the majority of the slopes are made of granular material.

The slopes of the railroad embankment adjacent to the spray pond were made of shot-rock. The slopes are at 3 horizontal to 1 vertical with a maximum height of 30 ft.

2.5.5.1.1 Geologic Conditions

The vicinity of the spray pond is situated over a glacial, or preglacial, east-west trending bedrock valley as outlined by contours on top of bedrock (Figure 2.5-17). These contours indicate that the bedrock surface of the valley was eroded about 100 ft below the average elevation of bedrock to the south and considerably more than that below bedrock elevations to the north. Total relief of the bedrock surface is about 130 ft. The valley is filled with dense gravelly and sandy glacial outwash and till deposits which attain a maximum thickness of about 110 ft in the spray pond area. They were deposited during the Olean substage (early Wisconsinan) of the Wisconsin glaciation, which occurred approximately 50,000 years ago, and there is a possibility that some of the bedrock erosion and overlying glacial deposits are the result of an earlier Illinoisan glaciation known to have occurred here (refer to Subsection 2.5.1.2). In general, the

deposits are normally consolidated and consist of a sequence of sand, gravel, and boulders overlain by sand and gravel, overlain in turn by silty sand. The entire sequence is highly variable in grain size distribution and sorting, and contains discontinuous pockets of similar materials. As a rule, grain size decreases and sorting increases toward the top of the sequence. Topsoil of variable thickness, consisting of brown sandy silt and organic matter, overlies the glacial drift.

Bedrock beneath the spray pond is correlated with the uppermost strata of the Middle Devonian Mahantango Formation. Strata of the overlying Trimmers Rock Formation crop out along the ridge north of the spray pond; the contact between these two formations is buried by glacial material, but has been inferred from drill hole data to occur immediately north of the spray pond along the buried south-facing bedrock slope (refer to Subsection 2.5.1.2.2.2 and Figures 2.5-40 and 2.5-56). The strata, which consist of dark gray, noncalcareous siltstone with fine sandstone stringers in the upper Mahantango grading to more sandy material in the Trimmers Rock Formation, strike N75°E and dip 15° to 40° north.

The southwestern tip of the spray pond is cut into bedrock while the remainder is excavated in glacial materials. The thickness of the glacial deposits beneath the bottom of the spray pond range from zero at the rock contact to 93 ft at the eastern end of the pond. The ESSW pumphouse structure located at the southeastern corner of the pond is underlain by 40 to 80 ft of glacial material. The water circulation pipelines between the pumphouse and the plant overlie glacial material having a maximum depth of 65 ft. They intersect bedrock at an elevation of 668 ft, approximately 260 ft southeast of the pumphouse (refer to Figure 2.5-17A).

North of the spray pond, the Trimmers Rock Formation forms a steep ridge rising approximately 380 ft above the spray pond. The south-facing slope of this ridge is essentially a rock slope underlain by resistant sandstone thinly mantled with soil and rock fragments. The sandstone is massive to flaggy and exposures exhibit well developed joint systems. The lower portions of the Trimmers Rock are less sandy and occur beneath the surface from the base of this high ridge southward to the northern part of the spray pond area (Figure 2.5-56).

Geologic conditions elsewhere at the site are reviewed in Subsection 2.5.4.1.

2.5.5.1.2 Groundwater Conditions

The groundwater table elevations and contours shown on Figure 2.5-38 are based on water level measurements made June 30, 1971 in the vicinity of the major plant structures, and on measurements made August 6, 1974 in the spray pond area. Water level measurements in the plant structures area were discontinued before the observation wells in the spray pond area were installed, and the wells were destroyed during construction of the plant. The water level data show that the groundwater table is in bedrock beneath the major plant structures, whereas beneath most of the spray pond it is in the glacial drift. Modification (lowering) of the water table by excavation in the major plant structures area is described in Subsection 2.4.13.5. However, some movement of groundwater from the plant structures area toward the spray pond to the north can still be expected, even though the major direction of movement is toward the Susquehanna River to the east. The direction of groundwater movement from the spray pond is also easterly toward the Susquehanna River. The undisturbed groundwater table elevation beneath the southwest end of the spray pond is about 670 ft where it is in bedrock. At the east end of the pond, it is in soil at an elevation of 615 ft.

The observation wells installed in the spray pond area have not been monitored long enough to allow a close determination of a maximum high water level. Monitoring of the observation points was discontinued in August 1975 and was resumed in January 1977. The recorded measurements suggest that, in some cases, up to 11 ft of fluctuation has occurred. However, the measurements taken during October 1974 are considered to be incorrect; therefore, they are not included in the evaluation. Eliminating those measurements, the maximum fluctuation is 7 ft (Table 2.5-9). Intermittent measurements of water levels at observation wells in the area of the principal plant structures were taken by Dames & Moore over a period of 11 months (1970 through 1971). These data indicate fluctuation of less than 10 ft. Using these limited data, it is estimated that the maximum rise of groundwater levels beneath the spray pond will not be greater than 10 ft above those on August 6, 1974.

2.5.5.1.3 Field Sampling and Testing

The field exploration for the spray pond was carried out from June 27, 1974 through August 15, 1974. The drilling subcontractor was American Drilling and Boring Company of Providence, Rhode Island. The boring locations are shown on Figure 2.5-44.

At the time of the investigation, the spray pond area had been used as a spoil area for excavation from the plant site. As much as 33 ft of soil and rock was dumped above natural ground. The majority of this was in the east half of the spray pond area. At the west end of the spray pond, a railroad fill consisting in large part of shot rock skirted the spray pond. The railroad fill was 30 ft deep at Boring 1120. The area between Borings 1110 and 1107 was the only area without any spoil.

Underlying the spoil material is glacial drift which in turn overlies siltstone bedrock. The depth of glacial material varies from 0 ft at Borings 1118 and 1121 to 108 ft at Boring 1104. The bedrock surface generally slopes to the east. At the southwest end of the site, bedrock is exposed at ground surface. The natural soils consist predominantly of sand, gravel, cobbles, and boulders. The soils are poorly stratified, starting as sand or sandy gravel at the surface and grading to mostly cobbles and boulders near bedrock. However, cobbles and boulders were encountered at various depths in most of the borings. Some of the sands and gravels were silty. Generalized sections through the pond area are given on Figure 2.5-30.

Twenty-five test borings were drilled. Ten holes were completed for the geophysical survey, ten for permeability and five as groundwater observation wells. Also shown on Figure 2.5-44 are borings in the 300 and 400 series made in 1971 and 1972 (Ref. 2.5-97 and 2.5-98). Information provided by these early borings was used for preparing the generalized sections given on Figure 2.5-30. The details of the drilling and sampling program are included in Subsection 2.5.5.3 along with logs of borings.

Permeability tests, using either packers or driven casing to isolate zones to be tested, were conducted in nine holes in the spray pond site. The method of analysis used is described in US Bureau of Reclamation Earth Manual, Designation E-18. One hole (1124) was constructed for permeability testing using the field permeameter method, as described in the US Bureau of Reclamation Earth Manual, Designation E-19. Locations of these test holes are shown on Figure 2.5-44, and results of the tests are listed in Table 2.5-10.

The tests were conducted primarily to determine permeability characteristics of the glacial drift and the contact zone between the glacial drift and the bedrock (siltstone of the Mahantango formation). Permeability testing of the Mahantango Formation was performed during investigation of the railroad bridge (Table 2.5-11). The siltstone beneath the spray pond is similar to that tested at the railroad bridge, and these data are taken as representative of the intact bedrock beneath the spray pond.

One of the test sections in the spray pond was isolated in the weathered and fractured siltstone (Boring 1117) immediately below the contact with the glacial drift. The calculated average permeability of that test (Table 2.5-10) is markedly higher than any of the tests performed in the intact bedrock, as would be expected. The exploratory holes in the spray pond area penetrated no more than 10 ft of the more permeable weathered bedrock. Three of the tests (Borings 1112, 1113, and 1114) measured permeability of the contact zone (including from 5 to 10 ft of the weathered bedrock with overlying glacial drift in the test section), and the balance of tests in the spray pond measured permeability of different materials in the glacial drift.

The boring logs indicate that the glacial drift is primarily outwash deposits consisting of permeable sands and gravels, with some discontinuous lenses of less permeable silty sands. The materials tend to be coarser and, presumably, more permeable toward the base of the deposits filling the small valley. The tests summarized in Table 2.5-10 indicate that the permeability of these materials varies considerably. Permeability of the predominant sand and gravel deposits is greater than 2,000 ft/yr (Borings 1111 and 1115). The silty sand lenses are much lower in permeability (Boring 1122 through 1125).

These data indicate that the average permeability of the glacial drift is considerably higher than that of the intact bedrock. The range of permeability in the glacial drift is greater, with permeability of some silty sands as low as some of the bedrock.

The maximum measured permeability of intact bedrock is 277 ft/yr, and the median value of the 41 tested intervals (Table 2.5-11) is 81 ft/yr. Assigning an average permeability of 200 ft/yr to the bedrock appears conservative. For purposes of seepage analysis, it can be assumed that bedrock is impermeable and groundwater movement occurs in the glacial drift.

The high permeability of the glacial outwash deposits is indicated by the two tests in which the capacity of the measuring equipment was exceeded. Also, during drilling of eight of the exploratory holes, there was considerable difficulty because of loss of drilling fluid (see Table 2.5-12). Commonly, it was necessary to drive casing to seal off highly permeable zones. The coarse nature of these lost-circulation zones precluded attempts to perform meaningful permeability tests. Further, the permeable nature of the glacial drift is demonstrated by the performance of the two plant site water wells for construction use (Figure 2.5-38). Each of these wells has a capacity of 150 gpm, and at least one is operating continuously. These wells draw from a maximum of 60 ft of saturated glacial drift. From the relationship of specific capacity of a water well to the thickness of the aquifer, the permeability of the aquifer can be estimated (Ref. 2.5-101). This method indicated 4,000 ft/yr as the apparent minimum average permeability at these wells.

An average permeability of 2,000 ft/yr for the glacial drift was used in the seepage analysis. Considering the evidence that highly permeable materials are present, the results of the permeability tests, and the yield from the wells, assumption of an average permeability of 2,000

ft/yr is conservative in relating seepage losses to groundwater levels and safety against liquefaction.

In the seepage analyses, the possible differences of vertical and horizontal permeabilities must be considered. The vertical permeability of glacial outwash deposits can be as small as one-fifth the horizontal permeability. Because groundwater in the saturated zone moves in a predominantly horizontal direction, the effective permeability is the horizontal permeability. In analyzing seepage through the unsaturated zone, however, movement of groundwater may be predominantly vertical; thus, the possibly lower vertical permeabilities were considered. Beneath the spray pond lenses of materials with low permeability are thin and discontinuous and therefore do not appear to cause a significantly lower permeability in the vertical direction. This is confirmed by the fact that no perched water has been detected in the area.

2.5.5.1.4 Laboratory Testing

2.5.5.1.4.1 General

In general, the granular deposits underlying the spray pond consist of silty sand at shallow depth, underlain by sandy gravel with boulders and cobbles. The test program was conducted only on the sands because of difficulties in collecting undisturbed gravel samples. The undisturbed samples were obtained in sand zones which had lower standard penetration blow counts than in the coarser material. The relative locations of soil samples for which the tests were made are shown on the generalized cross sections E and F, on Figure 2.5-45.

The laboratory test results are summarized in Table 2.5-13. For detailed information on test procedures and results, see Ref. 2.5-102.

2.5.5.1.4.2 Grain Size Distribution

Grain size determinations were made on most of the split spoon samples and on Shelby tube samples for classification purposes and to determine the D50 size that can be used as an index for evaluating the potential susceptibility of granular soils to liquefaction.

Sieve and hydrometer analyses were performed according to ASTM Procedure D 422-63, 1972. The range of grain size curves for the granular deposits is shown on Figure 2.5-31. The mean grain sizes (D50) of the samples of sand and gravel were found to be in the range of 0.14 to 3.0 mm and 4.5 to 25.0 mm, respectively.

2.5.5.1.4.3 Unit Weight

Unit weights were obtained for all undisturbed Shelby tube samples on which strength tests were performed. The undisturbed samples were obtained by cutting the Shelby tubes into approximately 7 in. lengths by a tube cutter. The length and weight of each sample section was determined while in the tube for unit weight computations. The unit weight is required to determine the relative density of the site soils.

2.5.5.1.4.4 Maximum-Minimum Densities

Maximum dry density values were obtained using two procedures; namely, impact compaction and vibratory compaction. Both tests were performed on samples obtained by mixing bulk samples from Test Pit No. 1.

The impact compaction tests were performed using ASTM Procedure D 1557-70, method D, modified so that each of the five layers was compacted with 20 blows of a 10 lb hammer dropping 18 in., i.e., a total compaction energy equal to 20,000 ft.lb/ft^3 of soil. The vibratory compaction test was performed according to ASTM Procedure D 2049-69 using a 0.1 cu ft mold and the wet method.

The maximum dry density obtained from these two tests were 106.1 pcf and 108.2 pcf, respectively.

The minimum dry density was also performed on bulk samples obtained from Test Pit No. 1 according to ASTM Procedure D 2049-69. The minimum dry density obtained was 91.5 pcf.

The relative density of the in site soils was determined using the maximum and minimum densities.

2.5.5.1.4.5 Relative Density

Relative density data were obtained from two sources: densities of the undisturbed Shelby tube samples were correlated to the maximum-minimum dry densities, and correlations were made with standard penetration test results obtained during the drilling operations using the Gibbs and Holtz procedure (Ref. 2.5-100).

The relative densities based on maximum-minimum dry densities were determined using the relationship:

$$D_d = \frac{\gamma^{\max}\left(\gamma - \gamma^{\min}\right) x100}{\gamma\left(\gamma^{\max} - \gamma^{\min}\right)}$$

as given in ASTM Procedure D 2049-69, where

D_d	=	relative density, percentage
γ^{max}	=	maximum dry density, pcf
γ^{min}	=	minimum dry density, pcf
γ	=	dry density of undisturbed samples, pcf

There was insufficient data to directly determine the relative density of each of the samples of in site soils. Therefore, the relative density of undisturbed samples was not used in the analyses. The design engineering properties of the site soils were based on tests on undisturbed samples. During drilling operation, a Standard Penetration Test was performed every 3 ft in each of the

drill holes. From these data and the values of effective overburden pressure at the location of each Standard Penetration Test, the relative densities were determined from the correlation between standard penetration resistance, effective overburden pressure, and relative density of granular soils given by Gibbs and Holtz (Ref. 02.5-100). The Gibbs and Holtz procedure is valid for normally consolidated sands. Values of relative density obtained in this way are summarized on Figure 2.5-46.

Relative density determinations of well stratified outwash sands and gravels, together with poorly stratified to unstratified kame-like gravels by currently available test procedures is not meaningful as the maximum and minimum density values will not be representative of the maximum and minimum density values of individual strata or lenses. Therefore no attempt was made to measure relative density directly. Furthermore, for the same reasons it is not possible in deposits such as those at the spray pond site, to select "the most appropriate maximum and minimum density values."

In deposits like these an indication of the relative density can be obtained from a conservative evaluation of the Standard Penetration test data. This was done by estimating the relative density from the lower bound values of the blowcounts by using the Gibbs and Holtz procedure (Ref. 2.5-100). Since the deposit is normally consolidated the Gibbs and Holtz procedure can be used to estimate relative density.

A direct comparison between measured relative density and Standard penetration test results cannot be presented as relative density was not measured directly.

2.5.5.1.4.6 Static Triaxial Shear Test

Eight static consolidated-drained triaxial tests were performed on undisturbed samples. The purpose of the test was to obtain the strength data required to evaluate the static stability of the cut slopes.

The tests were carried out in triaxial cells and the test specimens were saturated by the back pressure method. The saturation was checked by determining the value of Skempton's B coefficient (Ref. 2.5-103). Specimens were considered to be saturated when the B coefficient was 0.95 or higher. The specimens were obtained by cutting the 3 in. Shelby tubes into 7 in. lengths. They were then extruded and trimmed. The specimens were consolidated isotropically under effective consolidation pressures ranging from 0.50 to 6.0 ksf. These confining pressures represent the range of effective overburden pressures at the site. The results of these tests are presented on Figure 2.5-34 which also shows the selected design parameters.

2.5.5.1.4.7 Cyclic Triaxial Shear Tests

Twenty-five cyclic loading triaxial shear tests (CR) were performed to determine the cyclic shear strength of the soils. Sixteen tests were performed on undisturbed samples. Nine tests were performed on remolded samples. Undisturbed specimens were prepared in the same manner as for static triaxial tests. After completion of the tests on selected undisturbed specimens, they were oven dried, broken down, and compacted by vibration to the same dry density as the original undisturbed specimen. Test specimens were saturated as in the case of the static triaxial tests and consolidated under an isotropic pressure equal to either 1.0 ksp or 6.0 ksf.

During the cyclic shear tests, a symmetrical cyclic deviator stress was applied at a constant frequency ranging between 1 cycle per 2 to 3 seconds while measuring axial deformation, axial load, and pore pressure continuously by means of electric transducers and a chart recorder. The results of all CR tests including the number of cycles to reach a total strain of 5 percent are given in Table 2.5-14. The undisturbed specimens were generally found to be more resistant to cyclic loading than the corresponding remolded specimens prepared at the same dry density. The loss of shear resistance may be due to changes in the original soil structure and destruction of slight cementation which exists in the soil in the undisturbed state. The test results on undisturbed samples are shown on Figure 2.5-35.

Also shown on Figure 2.5-35 are the results of four cyclic triaxial tests reported by Dames & Moore (Ref. 2.5-98). In general, the Dames & Moore samples yielded higher cyclic strength. The reason for the difference may be due to the difference in the method of sampling. The undisturbed samples tested by GEI (Ref. 2.5-102) were sampled with a thin-walled Shelby tube sampler which was pushed by hydraulic pressure in accordance with ASTM D1587-67. However, the undisturbed samples tested by Dames & Moore were obtained with the "Dames & Moore" sampler. The area ratio of the "Dames & Moore" sampler is large compared to the thin-walled Shelby tube sampler, and the greater area ratio may result in greater disturbance to the sample. Since the amount of disturbance could not be evaluated and since the GEI samples yielded lower cyclic strength, the Dames & Moore results were not used in the liquefaction analysis for conservatism.

2.5.5.2 Design Criteria and Analyses

2.5.5.2.1 Design Criteria for Spray Pond

The design criteria adopted for the analysis of the spray pond and the slopes surrounding the spray pond include criteria for ground surface acceleration, liquefaction, and slope stability.

2.5.5.2.1.1 Ground Surface Acceleration

The horizontal ground accelerations used for design of the spray pond are 0.15g for the Safe Shutdown Earthquake (SSE) and 0.08g for the Operating Basis Earthquake (OBE).

2.5.5.2.1.2 Liquefaction

For the most adverse water level conditions at the spray pond site, the factor of safety provided against liquefaction should not be less than 1.2 for the SSE condition.

2.5.5.2.1.3 Slope Stability

The slopes in the area of the spray pond must be designed to provide a minimum factor of safety of 1.5 for the static condition and 1.1 when subjected to an SSE event.

2.5.5.2.2 Design Analyses for Spray Pond

The design analyses, including the seepage analysis, liquefaction potential of the spray pond, stability of slopes, and the earthquake induced settlement, are given in the following four Subsections.

2.5.5.2.2.1 Spray Pond Seepage Analysis

The total inventory that determines the spray pond capacity includes sufficient water to compensate for losses that could occur over the 30 day shutdown period. Additionally, seepage losses must be controlled during normal operation so that the groundwater table is not artificially elevated to a level that would aggravate the safety margin against liquefaction. Seepage analyses were made to determine what design parameters are required for the spray pond to meet these restrictions. It was first determined that seepage from an unlined pond does not meet these restrictions, and that a lining of the pond is required. The second case determines the design parameters for a lining that will sufficiently control the quantity of seepage to satisfy liquefaction requirements.

To maintain the groundwater level below the levels necessary to ensure an adequate factor of safety against liquefaction, an unsaturated zone must be maintained beneath the spray pond. A liner must be designed that will sufficiently restrict seepage and prevent groundwater levels from rising above the design levels. Seepage from the pond will increase the total groundwater underflow beneath the pond and develop a groundwater mound. That is: Total Underflow = Pond Seepage and Base Flow (the present underflow).
The groundwater flow path from the pond is eastward along the trough in bedrock which is filled with glacial deposits. The downstream discharge point of the groundwater mound is assumed to be at the surface at elevation 600 near a present spring. The quantity of underflow, Q, beneath the pond may be calculated using Darcy's law:

$$Q = KIA$$

Where:

Q	=	quantity of underflow (ft^3/yr)
K	=	permeability (ft/yr)
I	=	average hydraulic gradient (ratio)
A	=	cross sectional area of flow path (sq ft)

The controlling permeability for this case is the average permeability of the glacial drift, 2,000 ft/yr. The average hydraulic gradient may be taken as the difference in elevation between the elevation of the assumed discharge point, 600 ft, and the elevation of the water table beneath the center of the pond over the distance between the two points, 1,850 ft. Gradients were determined for several assumed elevations beneath the pond. The average cross-sectional area of saturated glacial draft along the flow path was determined for each assumed elevation of the groundwater mound beneath the pond.

An average base underflow of 4.3 X 10^5 ft^3/yr was calculated for the undisturbed groundwater conditions, represented by water levels shown on Figure 2.5-38. The amount of total underflow was then calculated for several assumed groundwater elevations beneath the center of the pond. The seepage which is producing the groundwater mound can be determined by subtracting the base flow of 4.3 X 10^5 ft^3/yr from the total underflow. Then, by using a form of Darcy's Law (Ref. 2.5-105):

$$q = k\,\frac{p+d;}{d}\ \text{and}\ Q = qa$$

Where:

q	=	quantity of seepage through one square foot of liner assumed to be saturated
K	=	effective liner permeability (ft/yr)
p	=	head of water in pond (ft)
d	=	liner thickness (ft)
Q	=	effective seepage losses through the liner (ft^3/yr)
A	=	area of the spray pond (sq ft)

The seepage loss as related to maximum groundwater level beneath the pond can be calculated. Then, the liner thickness and permeability that would restrict the amount of seepage sufficiently to maintain the selected groundwater elevation can be calculated. This provides a relationship between liner parameters and the elevation of the groundwater mound. The groundwater elevations beneath the pond that would be maintained by specific liner parameters are listed in Table 2.5-15 and shown on Figures 2.5-38 and 2.5-40. The relationship between seepage losses and the groundwater elevation beneath the pond is shown on Figure 2.5-47. To maintain the groundwater level below the maximum allowable level determined by the liquefaction analysis, 665 ft, a reinforced concrete liner has been constructed. The concrete liner has expansion, contraction and construction joints at appropriate spacing to control cracking. Both expansion and construction joints have impermeable rubber waterstops incorporated.

The relationship between the thickness of the liner, permeability of the liner and seepage loss is shown on Figure 2.5-57. The permeability of reinforced concrete is conservatively 1 X 10^{-2} feet per year. The minimum thickness of liner provided over the entire pond is 5 inches. Therefore, during normal operation, the seepage loss from the spray pond is estimated to be 5.9 X 10^4 gallons per 30 days. More than 5 times this amount is required to raise the groundwater level to the design value of 665 feet which was used for the liquefaction analysis.

Accident conditions and their possible effect on the integrity of the liner and on seepage losses have also been examined.

Should a tornado-generated missile having a frontal area of 20 square feet puncture the liner, the additional leakage would be

1.6×10^4 gallons per 30 days. This volume of water would not be sufficient to elevate the water table to an unacceptable level.

In the event of an SSE, the soils analysis covering slope stability, discussed in Subsection 2.5.5.2.2.3.2, shows that the cut slopes will remain stable. No credit is taken in the analysis for the presence of the concrete liner. Subsection 2.5.5.2.2.4 discusses the settlement which might result from an earthquake induced motion. The relative settlement across the pond would be very small, less than 1 inch in 500 ft. It is therefore anticipated that the liner will not undergo any significant displacement as a result of an SSE. Some additional cracking could occur. However, since a very conservative approach has been taken in providing a liner with a permeability well below that required to establish liquefication potential, the additional cracking can be tolerated.

2.5.5.2.2.1.1 Spray Pond Seepage Monitoring

A total of six (6) permanent observation wells in soil along the perimeter of the spray pond are used to monitor seepage from the spray pond. These piezometer installations consist of two inch (2") minimum diameter well point casings with base elevations of 645' along the west section of the spray pond to a low elevation of 625' east of the pond. Every six months readings of these wells will be taken to monitor the elevation of the ground water table beneath the concrete liner. A seventh well (piezometer number 2) is located in bedrock and therefore cannot be used to assess the possibility of liquefaction of the soil.

Seepage from the spray pond was documented by measuring pond levels, precipitation and evaporation. Meteorology and evaporation data was collected. Data collected included the following: Pan evaporation, air temperature relative humidity, wind speed, precipitation, solar radiation and cloud cover. Measurements were taken on a daily basis for a period of 33 days from 4/25/81 to 5/27/81 to perform the seepage study of the spray pond liner. Data collected as a result of this study is included in the quality assurance files.

Groundwater levels will be monitored every six months using the six (6) permanent piezometer wells (located in soil) discussed above. When it has been determined that the actual groundwater level has reached EL 663 feet at any one (1) of the six (6) piezometer locations, the following actions will be taken:

(1) NRC will be notified of the high (EL 663') groundwater condition;

(2) Steps will be taken to identify the cause of the rise in the water level;

(3) An assessment of the safety impact of the occurrence will be performed;

(4) Appropriate action(s) will be taken based on the findings of the safety impact analysis.

2.5.5.2.2.2 Liquefaction Potential

2.5.5.2.2.2.1 Method of Analysis

The evaluation of the liquefaction potential of the soils at the site was made by comparing the shear strength of the soils under cyclic loading conditions to the dynamic shear stress induced in the soils by the vibratory motion associated with the SSE. The ratio of shear strength to induced shear stress is termed the factor of safety against liquefaction. Since both the shear strength of the soils and the induced shear stresses are dependent on depth below ground surface, determinations of the factor of safety against liquefaction were made at various depths.

Soil profiles and the corresponding groundwater levels representative of the site conditions were chosen for the study. The SSE was applied at the ground surface and deconvolved downward to bedrock using the SHAKE 3 Computer Program (Ref. 2.5-106).

The soil profiles used in the analyses were conservatively assumed to consist only of sand even though they included gravel, boulders, and cobbles in places as discussed in Subsection 2.5.5.1. Based on limited information available (Ref. 2.5-107), the resistance to liquefaction of gravel, boulders, and cobbles is equal or better than that of sand. For instance Kishida (Ref. 2.5-112) has indicated that soils with D 60 less than 2 mm and with uniformity coefficients less than 10 are most susceptible to liquefaction (Ref. 2.5-98). The saturated unit weight of the sand was taken to be 130 pcf and the buoyant unit weight to be 67.5 pcf. The spray pond was simulated as a material with a low shear modulus value of 1.0 ksf. Because water does not transmit shear waves, the simulation was necessary so that the computer program SHAKE 3 could be used to compute the shear stresses induced by the earthquake. Use of this small modulus has an insignificant influence on the induced shear stresses.

2.5.5.2.2.2.2 Soil Profiles and Positions of Groundwater Table

As disclosed by the field investigation, the thickness of overburden varies at the site of the spray pond. The bedrock contours are shown on Figure 2.5-17. At the southwest end, bedrock was exposed at the ground surface and over 90 ft of granular deposits were encountered at the northeast end. Therefore, to evaluate the liquefaction potential, three soil profiles were chosen to represent three thicknesses of overburden. The depths from the bottom of the pond (Elevation 668 ft) to the bedrock for the three soil profiles were 93 ft (Profile 1 - east end of spray pond), 57 ft (Profile 2 - central section, and pumphouse), and 20 ft (Profile 3 - west end of spray pond). The predicted maximum groundwater levels that will occur beneath a lined pond as discussed in Subsection 2.5.5.2.2.1 were used at each profile.

To evaluate the liquefaction potential at other locations in the spray pond, the same soil profiles were used and the groundwater table was varied in accordance with the predicted maximum water table elevations given on Figure 2.5-40.

Figure 2.5-48 shows the soil profiles and the maximum groundwater levels used at Profiles 1, 2, and 3 in the analyses.

2.5.5.2.2.2.3 Shear Moduli

Cross-hole shear wave velocity measurements were performed during August and September 1974 (Ref. 2.5-99). Compressional and shear wave velocities were measured in situ to depths of about 100 ft by the cross-hole procedure. The average shear wave velocities obtained from the measurement are presented on Figure 2.5-36. As shown on the figure, the shear wave velocity increases linearly with depth. The average shear wave velocities used for soil and rock in the liquefaction analyses are also shown.

The shear moduli of sand were computed from the values of shear wave velocity as follows:

$$G = \frac{\gamma}{g} \quad V_s^2$$

Where:

G	=	shear modulus, psf
γ	=	unit weight, pcf
g	=	gravitational acceleration, ft/sec²
V_s	=	shear wave velocity, fps

The shear modulus is influenced by the confining pressure, the strain amplitude, and the relative density and, in general, these can be related by the equation:

$$G = 1000\, K_s \left(\bar{\sigma}_m\right) 1/2 \qquad \text{(Ref. 2.5-108)}$$

Where:

G	=	shear modulus, psf
k	=	a variable parameter, dependent on relative density, shear wave velocity, and strain amplitude
$\bar{\sigma}_m$	=	mean principal effective stress, psf

In the liquefaction analysis, the shear modulus values at the corresponding effective confining pressures obtained from the above equation, were used as initial values at very small strains. The strain-compatible shear moduli were then determined from the curve of shear modulus-shear strain relationship as given by Seed and Idriss (Ref. 2.5-108).

2.5.5.2.2.2.4 Cyclic Shear Strength

The results of cyclic triaxial shear tests are given in Table 2.5-14 and on Figure 2.5-35. The results are given in terms of the cyclic shear stress ratio (σ_1 - σ_3) cy/2 $\bar{\sigma}_c$ and the number of loading cycles required for the test specimen to reach a total axial strain of 5 percent, where:

$(\sigma_1 - \sigma_3)$ cy = cyclic deviator stress

$\overline{\sigma}_c$ = effective consolidation pressure

The selected design cyclic shear strength is given on Figure 2.5-35. Based on results of the site seismicity study and on the SSE having a magnitude less than 6, the cyclic shear strength at 5 equivalent uniform load cycles was considered appropriate and this was used in evaluating the liquefaction potential at the pond (Ref. 2.5-96). The cyclic tests were performed at two effective consolidation pressures, 1.0 and 6.0 ksf. These pressures were selected to envelope the actual field conditions. However, the 1.0 ksf was selected as a lower limit for the testing pressure. Testing of sand samples at very low pressure may not be relative. From the test results, the cyclic shear stress ratios at these two effective consolidation pressures were determined to be 0.320 and 0.260, respectively, for 5 loading cycles. A linear relationship was assumed in computing cyclic shear stress ratios at other effective consolidation pressures. The cyclic triaxial testing conditions differ from field conditions and to account for these differences, and also to permit the use of effective vertical pressures instead of effective consolidation pressures, a correction factor, C, must be applied to the test results before using them in liquefaction analyses. The correction factor is a function of relative density and values have been published by Seed and Idriss (Ref. 2.5-108). A value of 0.57 was used in the analyses. This corresponds to an average field relative density of 50 percent for normally consolidated sands at the site. Using the above data, the following relationship was established between field cyclic shear strength, and the effective vertical pressure:

Cyclic Shear Strength, $$\tau = 0.57\,\overline{\sigma}\left(0.32 - 0.012\,\overline{\sigma}_N\right)$$

or

$$\tau = \overline{\sigma}\left(0.189 - 0.0068\,\overline{\sigma}\right)$$
$$\left(\tau \text{ and } \overline{\sigma} \text{ in ksf}\right)$$

The above expression permits the calculation of the cyclic shear strength at any depth down to bedrock.

2.5.5.2.2.2.5 Determination of Dynamic Shear Stresses

The vibratory motion of the SSE was applied at the ground surface and deconvolved downward to the bedrock; thus, inducing shear stresses into the soil. The synthetic time history of ground surface acceleration during the SSE was used with a maximum acceleration of 0.15g as discussed in Subsection 2.5.2.6.

The maximum shear stresses developed at various depths within the soil during the SSE were calculated using the computer program SHAKE 3 developed by Schnabel, Lysmer, and Seed (Ref. 2.5-106). In addition to the SSE time history and maximum ground acceleration, the computer program uses the following parameters:

a) Unit weights of the subsurface strata and depth to the groundwater table

b) Damping ratios of the subsurface strata and the variation of these damping ratios with shear strain
c) Shear moduli of the subsurface strata and the variation of these moduli with shear strain

The output of the SHAKE 3 computer program provides values of the peak shear stresses induced in the various strata during an SSE event. However, for the liquefaction potential analysis, the equivalent uniform average shear stress is required. The average shear stress during the SSE has been taken to be equal to 0.65 times the peak shear stress (Ref. 2.5-107).

The results of the average shear stress determinations for the various cases analyzed are compared with the cyclic shear strength.

2.5.5.2.2.2.6 Design Earthquake

The synthetic acceleration time history as discussed in Subsection 3.7b.1.2 was used in the evaluation of liquefaction potential.

Based on the site seismicity studies discussed in Subsection 2.5.5.1.2, the ground acceleration of 0.15g was adopted for the SSE for structures founded on soil.

2.5.5.2.2.2.7 Results of Liquefaction Analyses

2.5.5.2.2.2.7.1 Liquefaction Potential Under the Design SSE

The average shear stresses induced by the SSE of 0.15g and the corresponding shear strengths and factors of safety, for three different profiles and various groundwater levels, are given in Table 2.5-16. The factors of safety are also shown on Figure 2.5-49.

Based on these values, it was possible to obtain and to interpolate the factor of safety at any particular location in the pond for the predicted maximum groundwater elevation as shown on Figure 2.5-38. On Figure 2.5-50 the factors of safety at seven selected locations are shown along with the information on the elevation of maximum predicted water table and bedrock. The minimum factor of safety was found to be 1.26, which is larger than the minimum acceptable factor of safety of 1.20, as given in Subsection 2.5.5.2.1.

As indicated by the results shown on Figure 2.5-50, the factor of safety decreases as the groundwater table rises, and at the same water level the factor of safety decreases as the depth to bedrock increases.

2.5.5.2.2.2.7.2 Variations of Shear Moduli and Damping Ratios for Evaluation of Liquefaction Potential

As mentioned in Subsection 2.5.5.2.2.2.5, the "standard" relationship between the effective strain and the dynamic properties (shear moduli and damping ratios) given by Seed and Idriss (Ref. 2.5-108) was used in the liquefaction analysis for estimating induced cyclic stresses. To

evaluate the effects of possible variations of these relationships, liquefaction studies were made initially in which the values of shear moduli and damping ratios were varied by ±30 percent.

A plot of damping ratio versus shear strain, used in the liquefaction analysis for the sandy deposits, is shown on figure 2.5-58. The plot is similar to the average curve of damping ratio versus strain presented by Seed and Idriss (Reference 2.5-108). In the liquefaction analysis, the curve was digitized and represented by a set of points connected by straight lines. No damping tests were made on samples from the site. However, the Seed and Idriss curves represent a summary of laboratory damping values determined for a wide range of granular materials and the data are given in Reference 2.5-108.

In the liquefaction analyses variation of damping does not have a very significant effect on the magnitude of induced stresses. For example, reducing the damping ratio by 30%, changes the average shear stress from 553 psf to 556 psf which is less than 1% (Table 2.5-17). The effect on Factor of Safety (F.S.) of change in the damping ratio is shown on Figure 2.5-50A. The trend established indicates that if the damping ratio was varied beyond 30%, the Factor of Safety would still be larger than the acceptable value of 1.2. Also shown on Figure 2.5-50A is a line representing the F.S. vs. change of damping ratio if the shear modulus was varied by 50%. This line was projected based on data in Table 2.5-17. As shown by this line, the minimum acceptable F.S. of 1.2 will still be satisfied by the combined variation of ±50% shear modulus and -50% damping ratio.

The most critical soil profile (Profile 2) with the maximum predicted water table at 665 ft was used in the study.

The study included the following cases:

a) Varying shear moduli ±30 percent, damping ratios remain unchanged
b) Varying damping ratios by ±30 percent, shear moduli remain unchanged
c) Change both shear moduli and damping ratios by ±30 percent

The average induced cyclic shear stresses, shear strengths, and factors of safety, along with the results using the standard relationship, are summarized in Table 2.5-17. The maximum change of shear stress is found to be about 3 percent. This reduces the minimum factor of safety to 1.23, but it is still larger than the acceptable value of 1.2 (Subsection 2.5.5.2.1). The results of this study indicate that the effects of variations of moduli and damping are small and do not change the conclusion that there is an adequate factor of safety against liquefaction.

A subsequent review of the results in Table 2.5-17 were made to determine the effect of varying the damping ratio by ±50% and the shear modules by ±50%. The review was made by projecting the results. A plot showing the effect on the factor of safety is given on Figure 2.5-50A.

2.5.5.2.2.2.7.3 Results of Liquefaction Analyses Using Real Earthquake Records

All the results of the liquefaction study, presented in the previous sections, were based on the design SSE of 0.15g at the ground surface. Some real earthquake records obtained at sites with comparable geologic conditions and in the same range of magnitude were available and were used to check the liquefaction potential. Three rock records: Golden Gate (M = 5.3, 1957), Helena (M = 6.0, 1935), and Parkfield (temblor Station, M = 5.6, 1966) were used for this purpose.

Liquefaction studies were made using these records applied at rock outcropping for obtaining cycle shear stresses in the soil.

The resulting factors of safety along with the factor of safety for the design SSE (Bechtel synthetic) are summarized on Table 2.5-18 and Figure 2.5-51. The minimum factors of safety obtained from the real records were larger than the ones obtained from the synthetic earthquake. The stresses induced by both the design SSE and the real earthquakes are also shown on Figure 2.5-52.

2.5.5.2.2.3 Slope Stability Analyses

2.5.5.2.2.3.1 Stability of Rock Slopes

The southwestern tip of the spray pond is cut into bedrock. However, since the cut slope is 3 horizontal to 1 vertical, the slope will obviously be stable, considering the engineering properties of the bedrock as discussed in Subsection 2.5.4. A detailed analysis of the stability of such a slope in rock is therefore not required.

North of the spray pond, the Trimmers Rock Formation forms a steep ridge rising approximately 380 ft above the bottom of the spray pond. The south-facing slope of this ridge is essentially a rock slope underlain by flaggy, resistant sandstone thinly mantled with soil and rock fragments. The closest approach of this slope to the spray pond is along the northern perimeter of the pond; the toe of slope at elevation 710-720 ft is at least 150 ft from the top of the north slope of the pond at elevation 700-727 ft (Refer to Figure 2.5-24 for final site grades in this area). The maximum slope along the ridge is about 2 horizontal to 1 vertical, and an overall slope of 3-1/2 horizontal to 1 vertical, a relatively flat slope for rock. Bedding in the rock dips approximately 30° to the north into the slope; thus, it is favorably oriented for slope stability. Data of McGlade (Ref. 2.5-56, p. 108) indicate that natural slopes eroded on Trimmers Rock strata are "steep and stable". In consideration of the competency of the rock forming the slope and the favorable orientation of rock structure, together with the fact that such gentle rock slopes are normally stable in this region, it is concluded that there is an ample margin of safety against failure of the slope north of the spray pond.

2.5.5.2.2.3.2 Stability of Slopes in Soil

Stability analyses were performed for the spray pond cut slopes and the fill slopes of the railroad embankment that are immediately adjacent to the pond. Both cut and fill slopes are constructed at 3 horizontal to 1 vertical. Except a small portion of cut slopes that are made in bedrock, the majority of cut slopes and all the fill slopes are made of granular materials. The granular materials range from sand and sandy gravel for the cut slope to the shot rocks for the embankment slopes. The shot rocks were obtained from the main plant excavation.

To evaluate the stability of the slopes, the effective angle of internal friction of the sand deposits was found to be 35° from the test data shown on Figure 2.5-34. For the sandy gravel and the shot rock, the effective angle of internal friction was conservatively assumed to be the same as that of the sand.

The pond is lined and the maximum predicted groundwater level is below the bottom of the slopes. Therefore, the infinite slope analysis and the yield acceleration analysis by Seed and Goodman (Ref. 2.5-109) are considered appropriate for evaluating the stability of the slopes.

For static conditions, the infinite slope analysis was used to determine the factor of safety of soil slopes:

$$FS = \frac{\tan\phi}{\tan i}$$

Where:

ϕ = friction angle of sand

i = inclination of slope

Therefore, for ϕ = 35° and i = $\tan^{-1}$ (1/3), the factor of safety under static condition is found to be 2.10.

For the dynamic condition the yield acceleration analysis was used. The yield acceleration is defined as the acceleration at which sliding will begin to occur. The yield acceleration coefficient (k_y) is defined as:

$$k_y g = \tan(\phi - i)g$$

where ϕ and i were defined in the previous paragraph. For ϕ = 35° and i = $\tan^{-1}$ (1/3), k_y is found to be 0.297. Compared to the SSE of 0.15g, the factor of safety for the dynamic condition would be:

$$FS = \frac{0.297}{0.150} = 1.98$$

Consequently, the railroad embankment slopes and the cut slopes will be stable under both the static and dynamic conditions.

2.5.5.2.2.4 Earthquake-Induced Soil Strain and Settlement

Two methods were used for estimating the earthquake induced settlement. Seed and Silver have proposed a method for determining settlement of sands that are sufficiently free draining in the field such that excess pore pressure cannot develop during an earthquake (Ref. 2.5-110). Lee and Albaisa have proposed a method that accounts for the reconsolidation settlement that results from dissipation of excess pore pressure following the earthquake (Ref. 2.5-111).

Following the procedure suggested by Seed and Silver (Ref. 2.5-110), the distributions of average induced shear strain as a function of depth for the soil profiles shown on Figure 2.5-48 were plotted and are shown on Figure 2.5-53.

It was conservatively assumed that the relationship between vertical strain and cyclic shear strain for sand at 60 percent relative density, as shown on Figure 8b of Ref. 2.5-110, was applicable for the sand deposits at the site. The corresponding values of vertical strain were then interpolated based on the values of shear strain as shown on Figure 2.5-53. The settlement of each layer was obtained by multiplying the layer thickness and the vertical strain. The total settlement was then obtained by summing up the settlements of all layers. By this approach, the vertical settlements at three Profiles 1, 2, and 3 shown on Figure 2.5-48 were found to be 0.05, 0.03, and 0.01 in., respectively. The results of these computations are summarized in the first half of Table 2.5-19.

To estimate the vertical settlement resulting from dissipation of excess pore pressure following the earthquake, the procedure proposed by Lee and Albaisa (Ref. 2.5-111) was followed. The results of cyclic triaxial shear tests carried out by GEI (Ref. 2.5-99) and an analysis using the SHAKE 3 computer program were used in addition to the experimental data shown on Figures 6 and 7 of Ref. 2.5-111.

The stress ratio causing liquefaction is related to field conditions by the equation (Ref. 2.5-104):

$$\frac{\sigma dc}{2\sigma a} = \frac{1}{C_r}\frac{(\tau)}{(\sigma'_o)}$$

in which

$\frac{\sigma dc}{2}$ = stress ratio

and C_r = correction factor

τ = shear stress induced

σ'_o = effective overburden pressure

The stress ratios developed at various depths can be computed when the shear stress induced during an earthquake and the effective overburden pressure are known. The stress ratios

developed at various depths were computed based on the values of induced shear stress given in Table 2.5-16, the computed effective overburden pressure, and a correction factor C = 0.57. Entering these stress ratios on Figure 2.5-35, the corresponding number of cycles (N_1) to reach a total axial strain of 5 percent are obtained. A nondimensional cycle ratio (N/N_1) is computed for each depth with N equal to 5.

Figure 2.5-54 was prepared to show the relationship between the peak excess pore pressure and cycle ratio. The correlation was based on the data obtained from the cyclic triaxial shear tests on undisturbed samples (Ref. 2.5-99). This figure is similar to Figure 9 of Ref. 2.5-111. The peak excess pore pressure ratio ($\Delta\mu/\sigma 3c$) that will occur during an earthquake can be estimated from Figure 2.5-54 using the cycle ratio (N/N_1) obtained previously.

The volumetric strains were estimated from Figures 6 and 7 of Ref. 2.5-110 using the peak excess pore pressure data in Figure 2.5-54. Figure 6 of Ref. 2.5-110 was used first to obtain the volumetric strains for sand at 50 percent relative density. These strains were then corrected to correspond to strains in sand at 60 percent relative density by multiplying by a factor of 0.8 obtained from the curve shown on Figure 7 of Ref. 2.5-110.

Lee and Albaisa assumed in their paper (Ref. 2.5-110) that vertical strain is equal to the measured volumetric strain in triaxial tests. However, when lateral movement is restricted as is the case of the soil deposit at the Susquehanna site, the vertical strain is one-half of such volumetric strain. Therefore, the volumetric strains obtained by the procedure of Lee and Albaisa were divided by two to obtain the appropriate vertical strains for the site conditions. The vertical settlement of each layer was then determined by multiplying the thickness of each layer by the vertical strain in the layer. The total settlement was obtained by summing up the settlements of each layer. The results of these computations are summarized in the second part of Table 2.5-19.

The values of total vertical settlement are also summarized below:

	Resulting from Compaction of Dry Soils (Inches)	Resulting from Reconsolidation of Saturated Soils (Inches)
PROFILE 1 (East End of Pond)	0.05	0.1-1.2
PROFILE 2 (Central Section, Pumphouse, etc.)	0.03	01.-1.0
PROFILE 3 (West End of Pond)	0.01	0.01-0.2

Based on the results given above, it is apparent that if the sands at the site behave like dry sand during an earthquake, then the settlement will be less than 0.05 in. However, if the sand deposits are saturated and excess pore pressures develop, they will reconsolidate following the earthquake and settlements up to 1.2 in. at the east end of the pond and up to 1.0 in. at the ESSW pumphouse may be expected.

The settlements given above were based on soil profiles consisting of sand deposits (Figure 2.5-48); the imposed dead and live loads on the mat footing of the pumphouse were not considered. The imposed weight will increase the confining pressure of the soil, resulting in a higher reconsolidation volumetric strain. However, according to Lee and Albaisa (Ref. 2.5-111), the influence of confining pressure is not strong and is only significant for developed excess pore pressure ratios greater than about 0.6. As shown on Table 2.5-19, the only soil that may develop a pore pressure ratio greater than 0.6 is at a depth below 43 ft near the bedrock of Profile 2. The soil at that depth, however, has a higher relative density than the 60 percent used for estimating the settlement. Since the volumetric strain decreases rapidly as the relative density increases (Ref. 2.5-111), the net combined effects of a larger pore pressure ratio developed and a higher relative density would result in a smaller volumetric strain. Therefore, the results given above are still valid for the additional imposed weight at the surface.

2.5.5.3 Logs of Borings

Logs of 25 test borings and two test pits are presented in Appendix 2.5C.

Standard Penetration Tests were made in 16 of these holes at 3 ft intervals. Undisturbed samples were taken in five of the split spoon holes where soil conditions permitted. Ten holes were completed for the geophysical survey, 10 for permeability and five as groundwater observation wells. The locations of borings and test pits are shown on Figure 2.5-44. Holes 1118, 1119, and 1121 were planned but not drilled.

Based on the results of the Standard Penetration Test borings, six borings (1106A, 1107A, 1110A, 1112A, 1113A, and 1115A) were drilled immediately adjacent to six of the Standard Penetration Test borings for undisturbed sampling of strata in which, based on classification of the split spoon samples, it was believed undisturbed samples could be obtained. Two test pits were dug with a Case backhoe to a depth of 12 ft to obtain bulk samples.

Due to the large amounts of oversize material encountered, drilling operations were slow and difficult. Frequent mud losses hampered drilling operations in spite of using additives in the drilling fluid. The additives included walnut shells, sawdust, cotton waste and Quick-gel. In some holes, it was necessary to case the hole in order to continue drilling.

Soil sampling consisted of both split spoon (Standard Penetration Test) and undisturbed sampling. The split spoon sampling was carried out in accordance with ASTM D1586-67. The undisturbed sampling was conducted in accordance with ASTM D1587-67. The undisturbed sampling was carried out using both Shelby tube and pitcher barrel sampler methods. In both cases, the sample tubes were 3 in. outside diameter, 3 ft long and the tubes were of 16 gage steel.

Undisturbed samples were difficult to obtain due to the large amount of gravel, cobbles, and boulders in the glacial drift. The majority of undisturbed samples were obtained from Borings 1106, 1106A, 1107A, 1113A, and 1122. Where possible, every attempt was made to obtain samples below the proposed bottom elevation of the spray pond (Elevation 668 ft).

All undisturbed samples were handled in the same manner. The top end of the tube was cleaned out; a piece of plastic followed by damp paper towels was inserted and a plug was then formed with microcrystalline wax. The bottom end of the tube was trimmed and an expandable

packer was installed. The packers were perforated with a 1/16 in. diameter hole for drainage of free water from the sample. Both ends of the tube were capped and dipped in wax. The samples were stored vertically with the expandable packer on the bottom in the subcontractor's equipment trailer in special boxes supplied for this purpose. The temperatures were well above freezing during the time they were stored so no provisions for heating were necessary.

Soil samples selected for the laboratory testing are indicated on the boring logs. The following symbols were used on the boring logs to indicate the type of laboratory test conducted.

CR - Cyclic Consolidated - Undrained Triaxial Test

S - Consolidated - Drained Triaxial Test

Gs - Specific Gravity Determination

Grain Size - Grain Size Determination

2.5.5.4 Compacted Backfill

Compacted fill is placed at the southeast corner of the spray pond to satisfy freeboard requirements. This fill has been placed with a maximum slope of 3 horizontal to 1 vertical. The material, placement, and testing specifications were as follows:

a) Material

Well graded, sound, dense, durable material. It does not contain any topsoil, roots, brush, logs, trash or waste material, ice, or snow. The maximum size of the material is 12 in. and no more than 35 percent by weight passed the No. 200 sieve.

b) Placement

The material was placed in uniform horizontal layers so that when compacted it was free from lenses, pockets, and layers of material differing substantially in grading from surrounding material. It was not placed on frozen ground. Placement for which moisture conditioning was required was suspended whenever the ambient temperature reached 34°F and falling.

The compaction requirements were specified as follows:

Fill shall be placed in a 15 in. maximum uncompacted layer thickness, moisture conditioned to obtain the required compaction, and compacted to satisfy both of the following requirements:

a) At least 80 percent relative density as determined by ASTM D2049 for material having not more than 12 percent passing the No. 200 sieve or 90 percent of maximum dry density as determined by ASTM D1157 for all other material.

b) Irrespective of the compacting effort to satisfy part a) above the fill shall be compacted in one of the following manners as a minimum effort:

i) Using a crawler tractor having a weight at least equal to that of a D8 Caterpillar tractor with bulldozer blade. Each track shall overlap the preceding track by not less than 4 in. When the tractor has made one entire coverage of an area in this manner, it will be considered to have made one pass. Each fill lift shall be compacted with four passes.

ii) Using a vibratory roller of minimum weight 20,000 lb having a roller width of approximately 78 in. and a diameter of approximately 60 in. The roller shall have a vibrator frequency range of between 1,100 and 1,600 vibrations per minute and have a minimum vibratory dynamic force of 40,000 lb. The roller speed shall not exceed 3 mph and each track shall overlap the preceding one by at least 4 in. When the roller has made one entire coverage of an area in this manner, it shall be considered to have made one pass. Each fill lift shall be compacted with four complete passes.

iii) Using a hand controlled vibratory compactor in locations inaccessible by tractor or vibratory roller. Approval to use hand controlled vibratory compactors will be on the basis of the demonstrated ability of the compactor to compact the material to the same density as the contiguous backfill.

c) Testing

The testing requirements were specified as follows:

The in site density of the fill shall be determined in accordance with ASTM D1556 and performed at a frequency of at least one test per lift and every 10,000 sq ft on plan.

Tests in accordance with ASTM D2049 or ASTM D1557, as appropriate, shall be carried out on the same material extracted for the ASTM D1556 test. The frequency of this testing shall be once in every 10 ASTM D1556 tests.

Gradation tests in accordance with ASTM D422 shall be carried out at least twice in each 8 hours during placing operations.

The railroad embankment to the north of the spray pond was constructed out of rock, obtained from the main plant area excavation.

The material and placement specifications were as follows:

a) Material

Fill shall consist of rock derived from Class B and C excavation having a maximum size of 20 in. Class B and C excavations are defined as follows:

1) Class B Excavation

Rock that cannot be excavated except by systematic ripping.

Ripping shall not be judged necessary when the material can be cut by a bulldozer in the following manner: A fifty-three and one-half (53 1/2) inch high bulldozer blade with standard rock or corner bits mounted on a caterpillar D-8 or equal tractor having 270 net flywheel horsepower moved through forty (40) feet of travel shall fill even with the top with a minimum angle of repose of forty-five (45) degrees, or a volume of ten (10) cubic feet per one linear foot of width of the blade.

2) Class C Excavation

Rock that cannot be excavated except by systematic drilling and blasting.

Blasting shall not be judged necessary if the rock can be ripped by a tractor rated at not less than 385 net flywheel horsepower, equipped with a single shank beam, parallelogram type (72" for deep arrangement), and weighing not less than 40 tons fully equipped; i.e., with dozer blade, ripper and other accessories. Drawbar pull will not be less than the following ratios:

1st gear - 95,000 lbs at 1 mph
2nd gear - 48,000 lbs at 2 mph
3rd gear - 30,000 lbs at 3 mph.

Fill shall be placed in lifts not exceeding 24 in. in uncompacted thickness and in such a manner so as to produce a well graded matrix.

b) Placement

Fill shall be compacted in one of the following manners:

1) Using a crawler tractor having a weight at least equal to that of a D8 Caterpillar tractor with bulldozer blade. Each track shall overlap the preceding track by not less than 4 in. When the tractor has made an entire coverage of an area in this manner, it will be considered to have made one pass. Each fill lift shall be compacted with four passes.

2) Using a vibratory roller of minimum weight 20,000 lb having a roller width of approximately 78 in. and a diameter of approximately 60 in. The roller shall have a vibrator frequency range of between 1,100 and 1,600 vibrations per minute and have a minimum vibratory dynamic force of 40,000 lb. The roller speed shall not exceed 3 mph and each track shall overlap the preceding one by at least 4 in. When the roller has made one entire coverage of an area in this manner, it shall be considered to have

made one pass. Each fill lift shall be compacted with four complete passes.

3) Using a hand controlled vibratory compactor in locations inaccessible by tractor or vibratory roller. Approval to use hand controlled vibratory compactors will be on the basis of the demonstrated ability of the compactor to compact the material to the same density as the contiguous backfill.

2.5.6 REFERENCES

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TABLE 2.5-1

MODIFIED MERCALLI INTENSITY (DAMAGE) SCALE
(Abridged)

I. Not felt except by a very few under especially favorable circumstances. (I Rossi-Forel Scale.)

II. Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing. (I to II Rossi-Forel Scale.)

III. Felt quite noticeably indoors, especially on upper floors of buildings, but 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 rocked noticeably. (IV to V Rossi-Forel Scale.)

V. Felt by nearly everyone, many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbances of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop. (V to VI Rossi-Forel Scale.)

VI. Felt by all, many frightened and 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. Person driving motorcars disturbed. (VIII+ to IX Rossi-Forel Scale.)

TABLE 2.5-1 (Continued)

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 -2

EARTHQUAKE LIST

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN* (MM)	MAGNITUDE*	REF*	DISTANCE (MILES)
1568				41.5	72.5	VII		ANY	193
1574				41.5	72.5	VII		ANY	193
1584				41.5	72.5	VII		ANY	193
1592				41.5	72.5	VII		ANY	193
1698				41.4	73.5	IV		ANY	140
1702				41.4	73.5	IV		ANY	140
1711				41.4	73.5	IV		ANY	140
6 AUG 1729				41.4	73.5	IV		ANY	140
19 DEC 1737	4	0	0.0	40.8	74.0	VII		EQH	115
25 APR 1758	2	30	0.0	38.9	76.5			EQH	151
30 NOV 1783	2	0	-0.0	41.0	74.5			PAG	87
30 NOV 1783	3	50	-0.0	41.0	74.5	VI		PAG	87
30 NOV 1783	7	0	-0.0	41.0	74.5			PAG	87
16 MAY 1791	3	0	0.0	41.5	72.5	VII		WGP	193
29 AUG 1792	3	0	0.0	41.5	72.5	IV		ANY	193
24 OCT 1792	6	0	0.0	41.5	72.5	IV		ANY	193
11 JAN 1793	13	0	0.0	41.5	72.5	IV		ANY	193
6 JUL 1793	11	0	0.0	41.5	72.5	IV		ANY	193
6 MAR 1794	19	0	0.0	41.5	72.5			EQH	193
9 MAR 1794	19	0	0.0	41.5	72.5	IV		EPA	193
9 MAY 1794	19	0	0.0	41.5	72.5	IV		ANY	193
12 AUG 1805				41.5	72.4	IV		ANY	198
30 DEC 1805	11	0	0.0	41.5	72.4	IV		ANY	198
28 DEC 1813	21	0	0.0	41.5	72.4	IV		ANY	198
12 APR 1837				41.7	72.7	V		EPB	185
9 AUG 1840	20	30	0.0	41.5	72.9	V		EQH	172
26 OCT 1845				41.0	73.8	V		WGP	123
9 SEP 1848	4	0	0.0	40.8	74.0	V		NYS	115
12 MAR 1853	7	3	0.0	43.7	75.5	VI		EQH	183
7 FEB 1855	4	30	0.0	42.0	74.0	V		WGP	129
13 MAR 1856	3	0	0.0	41.4	72.6	IV		WGP	187
23 OCT 1857	20	15	0.0	43.2	78.6	VI		EQH	192
1 JUL 1853	3	45	0.0	41.3	73.0	V		WGP	165
3 FEB 1862	1	0	0.0	41.5	72.5	IV		ANY	193
9 OCT 1871	14	40	0.0	39.7	75.5	VII		EQH	101
11 JUL 1872	10	25	0.0	40.9	73.8	V		EQH	124
11 DEC 1874	3	25	0.0	40.9	73.8	V		WGP	124
28 JUL 1875	9	10	0.0	41.8	73.2	V		EQH	161
10 SEP 1877	14	59	0.0	40.3	74.9	V		EQH	85
5 FEB 1878	16	20	0.0	40.7	73.7	V		WGP	131

*See Notes and Reference Key at end of Table.

TABLE 2.5 - 2 (CONT.)
EARTHQUAKE LIST

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN (MM)	MAGNITUDE	REF	DISTANCE (MILES)
4 OCT 1878	7	30	0.0	41.5	74.0	V		EQH	116
25 MAR 1879	24	30	0.0	39.2	75.5	V		EQH	134
9 AUG 1880	20	30	0.0	41.5	72.9	V		ANY	172
11 MAR 1883	23	57	0.0	39.5	76.4	V		EQH	109
12 MAR 1883	6	0	0.0	39.5	76.4	V		EQH	109
31 MAY 1884				40.6	75.5	V		ANY	48
10 AUG 1884	19	7	0.0	40.6	74.0	VII		EQH	118
3 JAN 1885	2	16	0.0	39.2	77.5	V		EQH	147
5 SEP 1886				41.5	72.5	IV		ANY	193
8 MAR 1889	23	40	0.0	40.0	76.0	V		EQH	75
9 MAR 1893	5	30	0.0	40.6	74.0	V		EQH	118
10 APR 1894				41.6	72.5	IV		ANY	194
1 SEP 1895	11	9	0.0	40.7	74.8	VI		EQH	76
5 SEP 1897				41.5	72.5	IV		ANY	193
17 MAY 1899	1	15	0.0	41.5	72.5	V		ANY	193
8 MAY 1906	17	41	0.0	38.7	75.7	V		EQH	166
8 MAY 1906	13	30	0.0	41.5	72.5	IV		ANY	193
10 JAN 1907	9	45	0.0	41.3	77.0	IV		ANY	45
24 JAN 1907	11	30	0.0	42.8	74.0	V		EQH	162
5 FEB 1908	8	20	0.0	41.4	73.2	IV		ANY	156
31 MAY 1908	17	42	0.0	40.6	75.5	VI		EQH	48
2 APR 1909	7	25	0.0	39.4	78.0	VI		EQH	151
23 APR 1910				41.0	73.0	IV		NJS	165
2 FEB 1916	16	26	0.0	42.9	74.0	V		EPB	167
8 JUN 1916	21	15	0.0	41.0	73.8	V		EQH	123
2 NOV 1916	2	32	0.0	43.3	73.7	V		EQH	198
16 FEB 1917	9	0	0.0	41.5	72.5	IV		ANY	193
6 SEP 1919	2	46	0.0	38.8	78.2	VI		EQH	191
19 JAN 1921	10	0	0.0	43.3	73.7	IV		NYS	198
26 JAN 1921	23	40	0.0	40.0	75.0	V		EQH	96
27 JAN 1921				43.3	73.7	IV		NYS	198
29 OCT 1925				41.5	72.5	IV		ANY	193
14 NOV 1925	13	4	0.0	41.5	72.5	V		WGP	193
16 NOV 1925	6	20	0.0	41.8	72.7	IV		ANY	186
26 JAN 1926	23	40	0.0	40.0	75.0	V		ANY	96
12 MAY 1926	3	30	0.0	40.9	73.9	V		EQH	119
30 MAR 1927				41.7	72.8	IV		ANY	180
1 JUN 1927	12	20	0.0	40.3	74.0	VII		EQH	126
12 AUG 1929	11	24	48.0	42.9	78.3	VIII		EPB	168
2 DEC 1929	22	14	0.0	42.8	78.3	IV		EPB	161

TABLE 2.5-2 (CONT.)
EARTHQUAKE LIST

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN (MM)	MAGNITUDE	REF	DISTANCE (MILES)
3 DEC 1929	12	50	0.0	42.8	78.3	IV		NYS	161
1 NOV 1930	6	34	0.0	39.2	76.5			USE	131
22 APR 1931				42.9	78.9	IV	3.6	EPB	188
1 JUL 1931	2	45	0.0	41.6	73.4	IV	3.6	EPB	148
25 JAN 1933	2	0	0.0	40.2	74.7	V	4.3	EPB	98
29 OCT 1933				43.0	74.7	IV		NYS	152
30 JAN 1934	10	30	0.0	41.8	72.6	IV	3.6	EPB	191
19 JUL 1937	3	51	0.0	40.7	73.7	IV		USE	131
15 JUL 1938	22	45	0.0	40.4	78.2	V		USE	119
2 AUG 1938	10	2	0.0	41.1	73.7	IV		USE	128
23 AUG 1938	3	36	34.0	40.2	74.2	V	4.6	EPB	116
23 AUG 1938	5	4	55.0	40.2	74.2	VI	4.8	EPB	116
23 AUG 1938	7	3	29.0	40.2	74.2	V	4.6	EPB	116
23 AUG 1938	11	11	6.0	40.2	74.2	IV	3.7	EPB	120
15 NOV 1939	2	53	48.0	39.6	75.2	V		CGS	114
18 NOV 1939	2	33	0.0	39.5	76.5			USE	110
24 OCT 1942	17	27	3.6	41.0	75.2	IV	3.4	EPB	48
1943				41.1	74.2	V		NJS	102
5 FEB 1944	16	22	.5	40.8	76.2	IV	3.7	EPB	19
14 DEC 1944	3	15	0.0	41.6	72.8	IV	3.6	EPB	178
28 OCT 1946	20	36	6.0	41.5	76.6	IV	3.6	EPB	36
10 NOV 1946	11	41	23.1	42.9	77.4	III	3.1	EPB	139
4 JAN 1947	18	51	4.0	41.0	73.6	V	4.3	EPB	135
20 MAR 1950	22	55	11.5	41.5	75.8	III	3.3	EPB	34
29 MAR 1950	14	43	2.0	41.0	73.6	IV	3.6	EPB	134
26 JAN 1951	3	27	0.0	41.5	72.5	III	3.3	EPB	193
3 SEP 1951	21	26	24.5	41.2	74.2	V	4.4	EPB	100
23 NOV 1951	6	45	36.0	40.6	75.5	IV	3.6	EPB	48
25 AUG 1952	0	7	0.0	43.0	74.5	V	4.3	EPB	157
8 OCT 1952	21	40	0.0	41.7	74.0	V	4.3	EPB	120
27 MAR 1953	8	50	0.0	41.1	73.5	V	4.3	EPB	139
17 AUG 1953	4	22	50.0	41.0	74.0	IV	3.7	EPB	113
7 JAN 1954	7	25	0.0	40.3	76.0	VI	5.0	EPB	54
1 FEB 1954	0	37	50.0	43.0	76.7	IV	3.3	NYS	135
21 FEB 1954*	20	0	0.0	41.2	75.9	VII	5.7	EPB	15
24 FEB 1954*	3	55	0.0	41.2	75.9	VI	5.0	EPB	16
31 MAR 1954	21	25	0.0	40.2	74.0	IV	3.6	EPB	127
11 AUG 1954	3	40	0.0	40.3	76.0	IV	3.6	EPB	55
20 JAN 1955	3	0	0.0	40.3	76.0	IV	3.6	EPB	54
21 JAN 1955	8	40	0.0	43.0	73.8	V	4.3	EPB	179

*See text (mine collapse).

TABLE 2.5-2 (CONT.)

EARTHQUAKE LIST

DATE	H	M (GMT)	S	LAT (NORTH)	LONG (WEST)	INTEN (MM)	MAGNITUDE	REF	DISTANCE (MILES)
16 AUG 1955	7	35	0.0	42.9	78.3	V	4.3	EPB	165
23 MAR 1957	19	2	0.0	40.6	74.8	VI	4.8	ANY	79
6 MAY 1958	19	0	0.0	42.7	73.8	IV	3.6	EPB	162
22 AUG 1958	14	25	5.0	43.0	79.0	IV	3.6	EPB	196
13 APR 1959	21	20	19.0	41.9	73.3	IV	3.4	EPB	161
22 JAN 1960	20	53	22.0	41.5	75.5	IV	3.4	EPB	45
15 SEP 1961	2	16	56.0	40.6	75.4	V		ANY	52
27 DEC 1961	17	6	0.0	40.1	74.8	V		ANY	98
6 MAR 1962				41.1	74.6			NJS	81
11 AUG 1962				41.1	74.6			NJS	81
7 SEP 1962	14	0	45.9	39.7	78.2			CGS	143
3 MAR 1963	1	24	32.0	41.5	75.8	IV*	3.4	ANY	34
19 MAY 1963	19	14	18.0	43.5	75.2	IV	3.5	EPB	173
20 MAY 1963	0	14	18.0	43.5	75.2	IV*	3.5	NYS	174
24 JUN 1963				41.1	74.6			NJS	81
1 JUL 1963	19	59	12.0	42.4	73.8	III	3.3	EPB	153
10 OCT 1963	14	59	52.5	39.8	78.2	IV	3.6	EPB	139
12 MAY 1964	6	45	14.1	40.2	76.5	VI	4.5	CGS	63
17 NOV 1964	17	8	0.0	41.2	73.7	V	4.3	EPB	128
16 JUL 1965	11	6	55.0	43.2	78.5	IV	3.5	EPB	189
28 AUG 1965	1	57	0.0	42.9	78.2	IV		NYS	163
29 SEP 1965	20	57	39.5	41.4	74.4	IV		NYS	94
1 JAN 1966	13	23	38.0	42.9	78.2	VI	4.7	CGS	163
1 JAN 1966	13	23	38.0	42.9	78.2	VI	4.7	EPB	163
21 MAY 1966	7	30	55.0	41.2	74.0		1.3	NYS	113
13 JUN 1967	19	8	54.4	42.9	78.2	VI	3.9	USE	163
22 NOV 1967	22	10	0.0	41.2	73.8	V		NYS	123
3 NOV 1968	8	33	52.5	41.4	72.5	V		ANY	192
10 DEC 1968	9	12	44.9	39.7	74.6	V	2.6PAL	USE	126
13 AUG 1969	2	42	0.0	42.9	78.2	IV		NYS	163
6 OCT 1969				41.0	74.6	IV		NJS	82
8 DEC 1972	3	0	32.6	40.1	76.2	IV		ERL	64
28 FEB 1973	8	21	32.3	39.7	75.4	VI	3.8SLM	ERL	101
27 APR 1974	14	45	39.1	41.0	76.0	III*	3.0GS	GS	12
7 JUN 1974	19	45	36.8	41.6	73.9	III*	3.3PAL	PAL	120
11 MAR 1976	21	7	20.2	41.0	74.4	IV	2.5PAL	D-M	94
13 APR 1976	15	39	13.0	40.8	74.1	V	3.0PAL	PAL	112

NOTES:

1. The magnitude listed on the printout is chosen in the following order: Local ML, CGS MS, Other MS, black if no magnitude information.

2. When maximum intensity is not given, an estimated intensity is calculated by using the relation: M = 1 + 2/3 I. The calculated intensity is indicated by an asterisk. However, a large percentage of our data cards were obtained from - Earth Physics Branch, Dept. of Energy, Mines and Resources, Canada (EPB), which converted the original intensity values, for earthquakes occurring before 1899, into Local Magnitude (ML), by using the above relation. Many magnitudes for earthquakes after 1899 were also converted from intensity values. However, because on the cards received from EPB, no distinction is made between Local Magnitude, ML, determined instrumentally and those estimated from intensity, the calculated intensities in this situation are not indicated by the asterisk.

3. The following abbreviations are used in Table C-1:

ANY	Earthquakes adjacent to New York State (N.Y. State Geological Survey)
CGS	Coast and Geodetic Survey
D-M	Dames & Moore (1969), proposed North Anna Power Statio
EEC	Earthquakes in Eastern Canada, Smith (1962)
EPB	Earth Physics Branch, Dept. of Ener. Mines and Res., Canada
EQH	Earthquake History of the United States, Coffman and Von Hake (1973)
ERL	Environmental Research Laboratories (NOAA)
G-R	Guttenberg and Richter (1953)
GS	U.S. Geological Survey
NJS	New Jersey State Geological Survey, Dombroski (1973)
NOS	National Ocean Survey (NOAA)
NUT	NUTTLI (1974)
NYS	Earthquakes within New York State (N.Y. State Geological Survey)
PAG	Page et al. (1968)
PAL	Lamont Doherty Geological Observatory, Palisades
USE	U.S. Earthquakes, Yearly Publication (NOAA)
WGP	Weston Geophysical

TABLE 2.5-3

UNCONFINED COMPRESSIVE STRENGTHS OF FOUNDATION ROCK (MAHANTANGO FORMATION)

BORING	PLANT STRUCTURE	SAMPLE ELEVATION (FT)	DEPTH (FT)	DEPTH BELOW TOP OF ROCK (FT)	COMPRESSIVE STRENGTH (LBS/SQ. IN.)	YOUNG'S MODULUS (LBS/SQ. IN.)
101	(a)	620	49	23	12,500	-
105	Reactor #1	618	63	61	14,400	-
110	(a)	494	183	73	3,650	-
114	(a)	608	31	19	11,600	-
204	(a)	612	35	29	910(b)	-
210	(a)	652	37	14	12,000	9.4×10^6
303	Turbine #1	657	38	28	16,000	4.45×10^6
307	Reactor #1	632	42	29	14,150	4.12×10^6
311	Reactor #2	629	46	41	14,300	3.1×10^6
320	Clg. Twr. #1	680	28	20	8,240	6.8×10^6
321	Clg. Twr. #2	643	36	14	6,940	5.2×10^6
323	Clg. Twr. #2	648	28	6	8,150	5.4×10^6

Notes:

(a) Boring location not contained within principal plant structure foundation. See Plot Plan Figure for location.

(b) Fractured on recemented joint.

TABLE 2.5-4

LABORATORY MEASUREMENTS OF P-WAVE SEISMIC VELOCITIES ON ROCK CORES TAKEN FROM REACTOR AND TURBINE BUILDING AREAS

Boring No.	Depth (feet)	Elevation (feet above MSL)	P-Wave Velocity, V_p (feet per second)
303	45	650	14,130
303	65	630	13,773
303	95	600	14,912
314	44	638	11,737
314	61	621	12,061
314	90	592	14,754
315	40	640	15,432
315	61	619	12,785
315	68	612	11,111
315	90	590	11,666

TABLE 2.5-5

REPRESENTATIVE ENGINEERING PROPERTIES OF UNWEATHERED FOUNDATION ROCK FOR PRINCIPAL PLANT STRUCTURES

PROPERTY	RANGE	AVERAGE VALUE
Density	-	170 lb/cu. ft.
Unconfined compressive strength	3,650 - 16,000 lb/sq. in.	11,080 lb/sq. in.
Static modulus of deformation	3.1 x 106 - 9.4 x 106 lb/sq. in.	5.5 x 106 lb/sq. in.
Compressional wave velocity (on core)(1)	11,111 - 15,432 ft/sec.	13,236 ft/sec.
Compressional wave velocity (in situ)(2)	14,000 - 16,000 ft/sec.	14,670 ft/sec.
Shear wave velocity (in situ)(2)	6,200 - 7,500 ft/sec.	6,730 ft/sec.

Notes:

(1) From laboratory "Shockscope" measurements on core samples (refer to Table 2.5-4).

(2) From cross-hole and down-hole measurements in reactor area (below elevation 670 ft. MSL).

TABLE 2.5-6

P-WAVE AND S-WAVE VELOCITIES AND COMPUTED DYNAMIC MODULI, [1] SPRAY POND AREA

ELEVATION (FT ABOVE MSL)	AVERAGE "P" WAVE VELOCITY (FT/SEC)	AVERAGE "S" WAVE VELOCITY (FT/SEC)	POISSON'S RATIO	YOUNG'S MODULUS (PSI)	SHEAR MODULUS (PSI)	BULK MODULUS (PSI)
		ESSW PUMPHOUSE ARRAY [2]				
683 to 658	2,350	1,320	.270	1.24×10^5	4.9×10^4	9.0×10^4
658 to 606	4,340	1,710	.408	2.31×10^5	8.2×10^4	4.19×10^5
606 to 603	13,300	6,400	.349	4.06×10^6	1.50 x 105	4.49×10^6
		SPRAY POND ARRAY [3]				
678 to 673±	2,600	1,300	.333	1.26×10^5	4.7×10^4	1.26×10^5
673± to 643±	3,200	1,480	.364	1.68×10^5	6.1×10^4	2.05×10^5
643± to 606	4,775	1,800	.417	2.58×10^5	9.1×10^4	5.19×10^5
606 to 588	12,400	5,850	.357	3.41×10^6	1.26×10^6	3.97×10^6

Notes:

(1) Computations are based on unit weights of 130 lbs/cu. ft. for soil and 170 lbs/cu. ft. for rock.
(2) ESSW array is a north-south alignment utilizing boreholes 1101 to 1105.
(3) Spray pond array is an east-west alignment utilizing boreholes 1106 to 1110.

TABLE 2.5-7

CROSS-HOLE SEISMIC VELOCITIES, REACTOR AREA[1]

ELEVATION (FT. ABOVE MSL) FROM	TO	COMPRESSIONAL WAVE (FT/SEC)	SHEAR WAVE (FT/SEC)
680	670	1,500	700
670	660	7,600	3,600
660	640	14,000	6,500
640	550	16,000	7,500

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(1) Utilizing boreholes 105, 303, 307, 314, 315 and 316 for the seismic array.

TABLE 2.5-8

RESULTS OF SETTLEMENT MEASUREMENTS TAKEN ON ESSW PUMPHOUSE BASEMAT

Date	Elevation (Ft. MSL) Pin #1	Pin #2	Pin #3	Pin #4	Pin #5	Pin #6
8-30-77	659.839	659.833	659.812	659.832	659.848	659.845
9-30-77	659.84	659.83	659.81	659.83	659.85	659.85
10-28-77	659.84	659.83	659.81	659.83	659.85	659.85
12-2-77	659.84	659.83	659.81	659.83	659.85	659.85
12-30-77	659.84	659.83	659.81	659.83	659.85	659.85
2-2-78	659.83	659.83	659.80	659.82	659.84	659.84
3-7-78	659.82	659.82	659.79	659.81	659.83	659.83
4-3-78	659.84	659.83	659.81	659.83	659.85	659.84
4-28-78	659.84	659.83	659.81	659.83	659.85	659.84
5-30-78	659.84	659.83	659.81	659.83	659.85	659.84
7-5-78	Backfilled	659.83	659.81	659.83	659.85	Backfilled
8-3-78	Backfilled	659.83	659.81	659.83	659.85	Backfilled
9-5-78	Backfilled	659.83	659.81	659.83	659.85	Backfilled
10-3-78	Backfilled	659.83	659.80	659.82	659.85	Backfilled
11-1-78	Backfilled	659.83	659.80	659.82	659.85	Backfilled
12-1-78	Backfilled	659.83	659.80	659.82	659.85	Backfilled
1-4-79	Backfilled	Inaccessable Due to Ice & High water				Backfilled
2-6-79	Backfilled	Inaccessable Due to Ice & High water				Backfilled
3-2-79	Backfilled	Inaccessable Due to Ice & High Water				Backfilled

NOTE: Pin Locations are given on Figure 2.5-41

TABLE 2.5-8 (Page 2)

RESULTS OF SETTLEMENT MEASUREMENTS
TAKEN ON ESSW PUMPHOUSE BASEMAT

Date	Pin #7	Pin #8	Pin #9	Pin #10	Pin #11
		Elevation (Ft. MSL)			
6-5-78	685.53	685.54	685.51	685.49	685.49
7-5-78	685.53	685.54	685.51	685.49	685.49
8-3-78	685.53	685.54	685.51	685.49	685.49
9-5-78	685.52	685.53	685.51	685.49	685.49
10-3-78	685.52	685.53	685.51	685.49	685.48
11-1-78	685.52	685.53	685.51	685.49	685.48
12-1-79	685.52	685.53	685.50	685.49	685.48
1-4-79	685.52	685.53	685.50	685.49	685.48
2-6-79	685.51	685.53	685.50	685.48	685.48
3-2-79	685.51	685.53	685.50	685.48	685.48
4-4-79	685.51	685.53	Temp. Inaccess.	685.48	685.48
5-3-79	685.51	685.53	"	685.48	685.47
6-4-79	685.50	685.52	"	685.47	685.46
7-2-79	685.50	685.52	"	685.47	685.47
8-1-79	685.50	685.52	"	685.47	685.47
9-4-79	685.51	685.53	"	685.48	685.48
10-2-79	685.51	685.53	"	685.48	685.47
11-9-79	685.51	685.53	"	685.48	685.48
12-4-79	685.50	685.52	"	685.48	685.47
1-8-80	685.51	685.53	"	685.48	685.47

TABLE 2.5-8 (Page 3)

RESULTS OF SETTLEMENT MEASUREMENTS
TAKEN ON ESSW PUMPHOUSE BASEMAT

Date	Pin #7	Pin #8	Pin #9	Pin #10	Pin #11
	Elevation (Ft. MSL)				
* 4-3-80	685.51	685.53	"	685.48	685.48
6-30-80	685.50	685.52	685.50	685.48	685.47
10-2-80	685.51	685.52	685.50	685.48	685.48

* Time interval for survey measurements revised to 3 months.

SSES - FSAR

TABLE 2.5-9

SPRAY POND
WATER LEVEL ELEVATIONS

OBSERVATION WELL NO.	1111	1112	1113	1114	1115
GROUND SURFACE ELEVATION	687.4	708.2	702.1	711.7	708.0
7-29-74	613.0	626.0	632.5	647.0	640.5
7-30-74	613.0	625.5	633.0	647.0	639.5
7-31-74	611.0	625.0	632.0	648.0	640.5
8-2-74	612.5	626.0	633.0	648.0	640.5
8-5-74	611.5	626.0	632.5	648.0	640.5
8-6-74	612.0	626.0	632.0	649.0	641.5
8-7-74	612.0	626.0	631.0	649.0	640.5
8-9-74	611.5	625.8(1)	632.0	648.0	640.2(1)
8-12-74	611.5	626.0	632.2(1)	645.5	640.5
8-14-74	612.0	626.0	633.0	644.0	640.5
8-16-74	612.0	626.0	633.0	645.0	640.0
8-19-74	612.0	626.0	632.2(1)	645.0	638.7(1)
8-21-74	611.5	626.0	632.5	645.0	640.5
8-22-74	612.0	626.0	633.0	645.0	640.5
8-27-74	611.0	625.0	631.5	644.0	638.5
9-3-74	610.5	625.0	632.0	644.5	638.5
9-4-74	611.0	625.5	632.5	644.5	638.5
9-8-74	611.2(1)	626.0	633.0	645.0	639.0
9-10-74	611.5	626.0	633.0	645.0	639.0
9-12-74	611.0	626.0	632.5	645.0	638.5
9-17-74	611.5	625.5	632.5	645.5	638.5
9-18-74	611.0	630.0	635.0	647.0	637.5
9-23-77	611.0	626.0	632.5	645.0	638.5
9-30-74	-	(2)	-	647.0	637.5
10-23-74	602.0(3)	-	628.1(3)	646.0(3)	630.5(3)
10-25-74	602.0(3)	-	628.1(3)	646.0(3)	630.5(3)
10-30-74	602.0(3)	-	629.1(3)	646.0(3)	630.5(3)
11-6-74	610.0	-	-	648.0	634.5
11-7-74	609.0	-	-	648.0	634.5
11-8-74	611.0	-	629.0	648.0	638.5
11-12-74	609.0	-	633.0	648.0	636.5
11-14-74	609.0	-	631.0	647.0	637.5
11-18-74	610.0	-	632.0	647.0	637.5
11-22-74	610.0	-	632.0	648.0	638.5
11-26-74	610.0	-	632.0	648.0	638.5

SSES - FSAR

TABLE 2.5-9 (CONTINUED)

OBSERVATION WELL NO.	1111	1112	1113	1114	1115
GROUND SURFACE ELEVATION	687.4	708.2	702.1	711.7	708.0
11-29-74	610.0	-	632.0	648.0	638.5
12-2-74	608.0	-	632.0	648.0	638.5
12-6-74	610.2(1)	-	633.0	648.0	638.5
12-10-74	610.0	-	633.0	648.0	638.5
12-12-74	610.0	-	633.0	648.0	637.5
12-18-74	610.0	-	633.0	648.0	637.5
12-24-74	609.0	-	633.0	648.0	637.5
12-31-74	610.0	-	633.0	648.0	637.5
1-10-75	610.0	-	633.0	648.0	637.5
1-17-75	610.0	-	632.0	646.5	637.5
1-22-75	610.0	-	632.0	647.0	637.5
1-30-75	610.0	-	632.0	647.0	638.5
27-75	612.0	-	632.0	648.0	637.5
2-13-75	612.0	-	632.0	648.0	637.5
2-21-75	612.0	-	632.0	648.0	637.5
2-26-75	612.0	-	632.0	648.0	637.5
3-5-75	612.0	-	632.0	648.0	637.5
3-13-75	612.0	-	632.0	648.0	638.5
3-18-75	613.0	-	632.0	648.0	639.5
3-24-75	613.0	-	632.0	648.0	640.5
4-2-75	613.0	-	632.0	648.0	640.5
4-11-75	613.0	-	632.0	648.0	639.5
4-17-75	613.0	-	632.0	647.0	639.5
4-28-75	613.0	-	632.0	648.0	639.5
5-15-75	614.0	-	632.0	648.0	638.5
529-75	614.0	-	632.0	647.0	638.5
6-4-75	614.0	-	631.5	647.0	638.5
6-12-75	613.5	-	631.5	647.0	638.5
6-18-75	613.5	-	631.5	647.0	638.5
6-30-75	614.0	-	631.5	647.0	638.5
7-1-75	613.5	-	631.5	646.2(1)	637.5
7-9-75	613.0	-	631.0	647.0	638.0
7-28-75	612.3(1)	-	632.5	647.6(1)	638.5
8-4-75	611.9(1)	-	632.5	648.0	638.1(1)

TABLE 2.5-9 (CONTINUED)

Notes: Water level measurements made to the nearest half a foot except where noted.

(1) Water level measurements made to the nearest .1 foot.

(2) Well No. 1112 destroyed after 9-23-74.

(3) Readings believed to be anomalous due to use of incorrect reference point by new monitor.

SSEE - FSAR

TABLE 2.5-10

PERMEABILITIES
MEASURED IN SPRAY POND[1]

Hole	Ground Surface Elevation (ft)	Depth Interval Tested (ft)		Material	Average Permeability of Tested Interval (ft/yr)
1111	687.4	89-99	(35-97) (97-99)	glacial drift siltstone	>2900[2]
1112	708.2	73-111(est.)	(96.-100.5) (100.5-106.)	glacial drift siltstone	560
1113	702.1	77-87	(77-83) (83-87)	glacial drift siltstone	1400
1114	711.2	47-74	(47-63) (63-74)	glacial drift siltstone	350
1115	708.0	65-75		glacial drift	>4300[2]
1117	706.8	21-31		siltstone	900
1122	685.0	33-35		glacial drift	30
1123	707.4	53-55		glacial drift	8
1125	705.3	53.5-55		glacial drift	16
1124[1]	705(est.)	36-40		glacial drift	50

1. Constant-head tests in bore holes were performed in accordance with Designation E-18 of the U.S.B.R. Earth Manual for all tested intervals, except for boring 1124. Testing of 1124 was performed in accordance with Designation E-19.

2. These are minimum values. These holes accepted water in quantities greater than could be pumped with available equipment (30 gpm). The minimum value was calculated by assuming the gravity head (H) is equal to three-fourths of the depth to the water table, and using the maximum pump-in rate (Q). Hole 1111 took over 30 gpm and hole 1115 took over 50 gpm.

SSES-FSAR

TABLE 2.5-11

PERMEABILITIES

MEASURED NEAR THE RAILROAD BRIDGE

HOLE	GROUND SURFACE ELEVATION (FT)	INTERVAL TESTED (FT)	MATERIAL	PERMEABILITY (FT/YR)
929	530.1	48 - 53	Rock	-
		38 - 53	Rock	5.7
		33 - 53	Rock	4.6
		28 - 53	Rock	29.2
		23 - 53	Rock	20.6
		18 - 53	Rock	104.6
930	531.2	35 - 40	Rock	127.5
		30 - 40	Rock	81.8
		25 - 40	Rock	67.2
		20 - 40	Rock	52.0
		15 - 40	Rock	175.8
931	521.7	40 - 47	Rock	36.3
		40 - 47	Rock	55.9
		36 - 47	Rock	100.3
		36 - 47	Rock	107.5
		30 - 47	Rock	95.4
		30 - 47	Rock	81.4
932	516.8	38 - 43	Rock	129.0
		32 - 43	Rock	40.4
		27 - 43	Rock	36.2
		22 - 43	Rock	257.0
933	519.9	36 - 41	Rock	48.7
		31 - 41	Rock	253.7
		27 - 41	Rock	276.6
934	519.4	32 - 38	Rock	43.7
		28 - 38	Rock	46.6
		22 - 38	Rock	60.6
935	526.1	36 - 41	Rock	187.9
		31 - 41	Rock	134.1
		26 - 41	Rock	123.4
		21 - 41	Rock	110.8
937	540.4	39 - 44	Rock	-
		30 - 44	Rock	25.7
		25 - 44	Rock	24.2
		20 - 44	Rock	31.2

SSES-FSAR

TABLE 2.5-11

HOLE	GROUND SURFACE ELEVATION (FT)	INTERVAL TESTED (FT)	MATERIAL	PERMEABILITY (FT/YR)
938	538.3	41 - 46	Rock	-
		36 - 46	Rock	33.4
		31 - 46	Rock	96.1
		26 - 46	Rock	140.5
939	514+	35 - 38	Rock	23.7
		30 - 38	Rock	201.9
		25 - 38	Rock	158.5
940	514+	38 - 43	Rock	-
		33 - 43	Rock	109.2
		28 - 43	Rock	194.4

NOTE: These permeability values were obtained assuming the water table is 38 feet below the ground surface in this area and that the gauge was 2.0 feet above the ground surface, unless otherwise specified.

SSES-FSAR

TABLE 2.5-12

CIRCULATION LOSSES IN DRILL HOLES IN SPRAY POND

Hole No.	Depth (ft) Below Surface	Elevation (ft)	Geologic Material
1102	88	613	sand and gravel
1105	62	639	coarse sand and gravel
1110	73	616	coarse-fine sand, coarse-fine gravel, cobbles & boulders
1111	88	599	sand & gravel - boulders
1114	60 66	651 645	fine-coarse silty sand, little gravel, cobbles
1116	43 84	658 617	sand & gravel, boulders sand & gravel, boulders
1123	22	685	silty gravely sand
1127	46	659	sand & gravel, cobbles and boulders

SSES - FSAR
TABLE 2.5 - 13

SOIL TEST RESULTS SUMMARY (continued)

JOB NO 8856 PROJECT Susquehanna Unit 1 & 2 FEATURE Spray Pond DATE Jan. 10, 1975

HOLE, TEST PIT, OR TRENCH NO	SAMPLE NO.	DEPTH FT. FROM	DEPTH FT. TO	LABORATORY CLASSIFI-CATION	MECHANICAL ANALYSIS GRAVEL %	SAND %	FINES %	ATTERBERG LIMITS LL	PL	PI	SPECIFIC GRAVITY G	NATURAL WATER CONTENT %	TOTAL UNIT WEIGHT PCF	DRY UNIT WEIGHT PCF	COMPACTION OPTIMUM WATER %	MAXIMUM DRY DENSITY PCF	SHEAR DATA TEST	INITIAL WATER %	INITIAL DRY DENSITY PCF	c KSF	φ DEG	$\bar{\sigma}_c$ KSF	CYCLE	STRESS RATIO	CONSOL-IDATION TEST	REMARKS
1106A	UD-8A	22.0	23.5	SP	4 1	91 94	5 5					10.2	113.6	103.1			$C\bar{R}$	10.2	101.8			60	2	.456		
	UD-8B	22.0	23.5	SP	1	92	7					23.9	124.0	100.1			$C\bar{R}$	23.9	100.4 101.6			1.0	6 10	.388 .30[illegible]		2nd sample remolded
1107	SS-18	52	-	SW	10	79	11																			
	SS-20	58		SW	3	88	9																			
1107A	UD-7A	18.5	21.0	SP	0 0	96 95	4 5				2.70	17.5	117.9	100.4			$C\bar{R}$	17.5	100.7			6.0	11	.242		
	UD-7B	18.5	21.0	SP	0	94	6					17.1	116.8	99.8			$C\bar{R}$	17.1	96.2			1.0	25	.236		Remolded sample for $C\bar{R}$ test
	UD-8A	21.5	23.8	SP	0 0	94 95	6 5					13.9	112.8	99.0			$C\bar{R}$	13.9	99.0			6.0	48	.14[illegible]		
	UD-8B	21.5	23.8	SP	0	95	5					23.5	125.9	102.0			$C\bar{R}$	23.5	100.9 100.2			1.0	184 34	.21[illegible] .186		2nd sample remolded
	UD-10	51		SM	8																					Grain size curve stop at #10 sieve
	UD-11	55.5		GP	76	16	8																			
	UD-12			GW	63																					Grain size curve stop at #10 sieve
1110	SS-16	47		GP-SM	54																					Grain size curve stop at #10 sieve
	SS-18	53		GW	59	31	10																			
	SS-21	62		GW-SW	55	32	13																			
	SS-22	65		GW	66																					Grain size curve stops at #10 sieve

* VISUAL CLASSIFICATION

SPECIFIC GRAVITY
(a) - MINUS NO.4
(b) + PLUS NO.4

COMPACTION
(1) ASTM D698
(2) ASTM D1557
(3) 20,000 FT.LBS./CU.FT.
(4) MAXIMUM - MINIMUM
(5) OTHER (SEE TEXT)

TRIAXIAL COMPRESSION TESTS

UC UNCONFINED COMPRESSION
UU UNCONSOLIDATED UNDRAINED
CU CONSOLIDATED UNDRAINED
$C\bar{U}$ CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)
CD CONSOLIDATED DRAINED
$C\bar{R}$ CYCLIC CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)

OTHERS
$\bar{\sigma}_c$ = CONFINING PRESS. KSF

SSES - FSAR
TABLE 2.5 - 13

SOIL TEST RESULTS SUMMARY

JOB NO 8856 PROJECT Susquehanna Unit 1 & 2 FEATURE Spray Pond DATE Jan. 9, 1975

HOLE, TEST PIT, OR TRENCH NO	SAMPLE NO.	DEPTH FT. FROM	DEPTH FT. TO	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS GRAVEL %	SAND %	FINES %	ATTERBERG LIMITS LL	PL	PI	SPECIFIC GRAVITY G	NATURAL WATER CONTENT %	TOTAL UNIT WEIGHT PCF	DRY UNIT WEIGHT PCF	COMPACTION OPTIMUM WATER %	MAXIMUM DRY DENSITY PCF	SHEAR DATA TEST	INITIAL WATER %	INITIAL DRY DENSITY PCF	c KSF	φ DEG	$\bar{\sigma}_c$ KSF	CYCLE	STRESS RATIO	CONSOLIDATION TEST	REMARKS
1104	UD-2	41		GW-SM	55	38	7																			
	UD-4	56.5	58.2	SW	38	53	9					8.4	142.8	131.9												Sample disturbed
	UD-5	69		GW	56																					Grain size curve stops at #10 sieve
1106	UD-6B	15.5	17.2	SP	0	95	5					11.4	113.5	101.6			$\bar{CR}$	11.7	99.0			6.0	4	.28[illegible]		
	UD-8A	19.5	21.1	SP	3	93	4				2.71	9.2	108.3	99.2			$\bar{CR}$	9.2	98.3			6.0	2	.32		
	UD-8A	19.5	21.1	SP	0	99	1				2.71	9.2	108.3	99.2			$\bar{CR}$	12.9	97.9 / 98.1			1.0	45 / 16	.21[illegible] / .18[illegible]		2nd sample remolded
	UD-8B	19.5	21.1	SP	1	95	4					12.9	113.7	100.7												
	UD-9A	21.5	23.1	SP	0	94	6					7.8	110.8	102.8												
	UD-9B	21.5	23.1	SP	0	95	5					10.7	112.7	101.8			$\bar{CR}$	10.7	100.2 / 99.9			1.0 / 1.0	108 / 39	.13[illegible] / .13[illegible]		2nd sample remolded
	UD-10A	23.5	26.1	SM	15	76	9				2.70	12.5	123.4	109.7			$\bar{CR}$	12.5	107.7			6.0	23	.15[illegible]		
	UD-10B	23.5	26.1	SW	5	79	16					14.1	127.7	111.9			$\bar{CR}$	14.1	109.5			1.0	40	.21[illegible]		
	UD-11A	26.5	29.0	SM	5	78	17					18.1	127.9	108.3			$\bar{CR}$	18.1	106.3			6.0	20	.175		
	UD-11A	26.5	29.0	SM	3	90	7					18.1	127.9	108.3												
	UD-11B	26.5	29.0	SP	0	89	11					22.1	129.7	106.2			$\bar{CR}$	22.1	105.7 / 106.0			1.0 / 1.0	48 / 12	.26[illegible] / .23[illegible]		Remolded sample for $\bar{CR}$ Test
1106A	UD-2B	9.5	11.4	SP	2	80	18					12.9	109.6	97.1			CD	12.9	97.5	0	43					
	UD-5B	15.0	16.6	SP	0	86	14					11.9	112.0	100.1			CD	11.9	99.2	0	43					
	UD-6B	17.0	18.5	SP	0	95	5					10.9	113.3	102.2			$\bar{CR}$	10.9	102.8			[illegible].0	55	.167		
	UD-7A	18.5	21.0	SP	0	92	8				2.70	12.5	109.5	97.3												
	UD-7B	18.5	21.0	SM	0 / 0	77 / 95	23 / 5					19.2	118.5	99.4			$\bar{CR}$	19.2	97.0 / 99.8			1.0	6 / 10	.28[illegible] / .21[illegible]		2nd sample remolded

* VISUAL CLASSIFICATION

SPECIFIC GRAVITY
(a) = MINUS NO. 4
(b) = PLUS NO. 4

COMPACTION
(1) = ASTM D698
(2) = ASTM D1557
(3) = 20,000 FT.LBS./CU.FT.
(4) = MAXIMUM - MINIMUM
(5) = OTHER (SEE TEXT)

TRIAXIAL COMPRESSION TESTS
UC UNCONFINED COMPRESSION
UU UNCONSOLIDATED UNDRAINED
CU CONSOLIDATED UNDRAINED
$\bar{CU}$ CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)
CD CONSOLIDATED DRAINED
$\bar{CR}$ CYCLIC CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)

OTHERS
$\bar{\sigma}_c$ = CONFINING PRESS. KSF

BSES - FSAR
TABLE 2.5 - 13

SOIL TEST RESULTS SUMMARY (Continued)

BECHTEL

JOB NO. 8856 PROJECT Susquehanna Unit 1 & 2 FEATURE Spray Pond DATE Jan. 10, 1975

HOLE, TEST PIT, OR TRENCH NO	SAMPLE NO.	DEPTH FT.		LABORATORY CLASSIFI-CATION	MECHANICAL ANALYSIS			ATTERBERG LIMITS			SPECIFIC GRAVITY	NATURAL			COMPACTION		SHEAR DATA								CONSOL-IDATION TEST	REMARKS
		FROM	TO		GRAVEL %	SAND %	FINES %	LL	PL	PI	G	WATER CONTENT %	TOTAL UNIT WEIGHT PCF	DRY UNIT WEIGHT PCF	OPTIMUM WATER %	MAXIMUM DRY DENSITY PCF	TEST	INITIAL WATER %	INITIAL DRY DENSITY PCF	c KSF	φ DEG	$\bar{\sigma}_c$ KSF	CYCLE	STRESS RATIO		
1111	SS-13	39		SW	45																					Grain size curve stops at #10 sieve
	SS-14	42		SW	65	26	9																			
	SS-22	66		SW	37																					Grain size curve stops at #10 sieve
	SS-23	69		SW-GW	42	46	12																			
1112	SS-14	57		GW	70	22	8																			
	SS-15	60		SW	20																					Grain size curve stops at #10 sieve
	SS-25	90		SW	15	66	19																			
	SS-26	93		SM	11																					Grain size curve stops at #10 sieve
1112A	UD-5	26.0	28.6	SW-GW	44	50	6					17.6														
1113	SS-13	41		SW-GW	47	47	6																			
	SS-19	58		SW	35	57	8																			
1113A	UD-3B	25.0	27.5	SW	10	80	10					3.9	110.8	106.7			CD	3.9	110.0	0	36					
	UD-4A	28.0	30.5	SW	13	82	5					10.6	117.7	106.4												Sample disturbed
	UD-4B	28.0	30.5	SW	13	79	8					9.1	121.1	111.0												Sample disturbed
	UD-6A	33.5	35.4	SP	7	87	6					9.8	125.6	114.4			CD	9.8	117.3	0	46					
1115	SS-12	45		GM-SW	61	32	7																			
	SS-15	55		GM-SW	50	37	13																			

* VISUAL CLASSIFICATION

SPECIFIC GRAVITY
(a) = MINUS NO.4
(b) = PLUS NO.4

COMPACTION
(1) ASTM D698
(2) ASTM D1557
(3) 20,000 FT.LBS./CU.FT
(4) MAXIMUM - MINIMUM
(5) OTHER (SEE TEXT)

TRIAXIAL COMPRESSION TESTS
UC UNCONFINED COMPRESSION
UU UNCONSOLIDATED UNDRAINED
CU CONSOLIDATED UNDRAINED
C̄U CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)
CD CONSOLIDATED DRAINED
C̄R CYCLIC CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)

OTHERS
$\bar{\sigma}_c$ = CONFINING PRESS, KSF

SSES - FSAR
TABLE 2.5 - 13

SOIL TEST RESULTS SUMMARY (Continued)

JOB NO. 8856 PROJECT Susquehanna Unit 1 & 2 FEATURE Spray Pond DATE Jan. 10, 1975

HOLE, TEST PIT, OR TRENCH NO.	SAMPLE NO.	DEPTH FT. FROM	DEPTH FT. TO	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS GRAVEL %	SAND %	FINES %	ATTERBERG LIMITS LL	PL	PI	SPECIFIC GRAVITY G	NATURAL WATER CONTENT %	TOTAL UNIT WEIGHT PCF	DRY UNIT WEIGHT PCF	COMPACTION OPTIMUM WATER %	MAXIMUM DRY DENSITY PCF	SHEAR DATA TEST	INITIAL WATER %	INITIAL DRY DENSITY PCF	c KSF	φ DEG	$\overline{\sigma_c}$ KSF	CYCLE	STRESS RATIO	CONSOLIDATION TEST	REMARKS
1115A	UD-2	35		GW-SW	52	38	10																			
1116	SS-16	57		SW	23	59	18																			
	SS-19	66		SW	29	58	13																			
1120	UD-4A	52.0	53.5	SM	0	90	10					13.1	114.2	101.1			CD	13.1	102.4	0	36					
	UD-4B	52	53.5	SM	0	87	13					15.6	116.9	101.2												
	SS-10	69		GW-SW	47	42	11																			
	SS-14	83		GW	53	36	11																			
1122	UD-2A	8.0	9.8	SP	0	92	8					10.1	112.8	102.5			CD	10.1	105.4	0	43					
	UD-2B	8.0	9.8	SP	0	96	4					11.4	117.6	105.6												
	UD-4B	16.0	17.5	SM	2	68	30					8.7	108.7	100.0			CD	8.7	99.4	0	40					
1128	UD-4B	26.0	28.0	SW	8	84	8					14.1	139.4	122.2			CD	14.1	119.9	0	50					
	SS-12	50		SW	35	53	12																			
	SS-14	60		GW-SW	48	42	10																			
Test Pit	TP-1			SP	0	90	10																			

* VISUAL CLASSIFICATION

SPECIFIC GRAVITY
(a) = MINUS NO. 4
(b) = PLUS NO. 4

COMPACTION
(1) = ASTM D698
(2) = ASTM D1557
(3) = 20,000 FT. LBS./CU. FT.
(4) = MAXIMUM - MINIMUM
(5) = OTHER (SEE TEXT)

TRIAXIAL COMPRESSION TESTS
UC UNCONFINED COMPRESSION
UU UNCONSOLIDATED UNDRAINED
CU CONSOLIDATED UNDRAINED
$\overline{CU}$ CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)
CD CONSOLIDATED DRAINED
$\overline{C\bar{R}}$ CYCLIC CONSOLIDATED UNDRAINED (PORE PRESSURE MEASUREMENTS)

OTHERS
$\overline{\sigma_c}$ = CONFINING PRESS. KSF

TABLE 2.5-14

SPRAY POND

SUMMARY OF CYCLIC

CONSOLIDATED-UNDRAINED TRIAXIAL ($\overline{CR}$) TESTS

Test† Number	Sample* Condition	Dry Unit Weight, pcf	Relative ** Density, % (ASTM D-2049)	Consolidation Pressure, $\bar{\sigma}_c$ ksf	Number of Cycles To 5% Strain	Stress Ratio $(\sigma_1-\sigma_3)_{cy}/2\bar{\sigma}_c$
$\overline{CR}$-1	U.D.	100.4	57.4	1.0	6	0.338
$\overline{CR}$-2	U.D.	101.8	65.6	6.0	2	0.456
$\overline{CR}$-3	U.D.	97.9	42.4	1.0	45	0.210
$\overline{CR}$-4	U.D.	100.2	56.3	1.0	108	0.136
$\overline{CR}$-5	U.D.	98.3	44.8	6.0	2	0.327
$\overline{CR}$-6	U.D.	109.5	100.0	1.0	40	0.214
$\overline{CR}$-7	U.D.	107.7	97.5	6.0	23	0.156
$\overline{CR}$-8	R.	105.7	87.0	1.0	48	0.267
$\overline{CR}$-9	U.D.	106.3	90.2	6.0	20	0.175
$\overline{CR}$-10	U.D.	97.0	36.7	1.0	6	0.280
$\overline{CR}$-11	U.D.	96.6	34.2	1.0	14	0.240
$\overline{CR}$-12	U.D.	100.7	59.2	6.0	11	0.242
$\overline{CR}$-13	U.D.	100.9	59.1	1.0	184	0.219
$\overline{CR}$-14	U.D.	99.0	49.1	6.0	48	0.145

TABLE 2.5-14 (Continued)

Test† Number	Sample* Condition	Dry Unit Weight, pcf	Relative ** Density, % (ASTM D-2049)	Consolidation Pressure, $\bar{\sigma}_c$ ksf	Number of Cycles To 5% Strain	Stress Ratio $(\sigma_1-\sigma_3)_{cy}/2\bar{\sigma}_c$
$C\bar{R}$-15	R.	99.6	52.7	1.0	10	0.215
$C\bar{R}$-16	U.D.	106.0	88.2	1.0	12	0.231
$C\bar{R}$-17	R.	97.0	36.7	1.0	2	0.229
$C\bar{R}$-18	R.	99.9	54.5	1.0	39	0.125
$C\bar{R}$-19	R.	101.2	62.1	1.0	34	0.186
$C\bar{R}$-20	R.	98.1	43.6	1.0	11	0.183
$C\bar{R}$-21	U.D.	102.8	71.2	6.0	55	0.167
$C\bar{R}$-22	R.	108.6	100.0	1.0	10	0.198
$C\bar{R}$-23	U.D.	99.0	49.1	6.0	4	0.288
$C\bar{R}$-24	R.	96.2	31.7	1.0	25	0.236
$C\bar{R}$-25	R.	101.6	64.4	1.0	10	0.302

*Note: U.D. = Undisturbed Sample
R. = Remolded Sample

**Relative Density Determined by

$$D_d = \frac{\gamma \max (\gamma - \gamma \min)}{(\gamma \max - \gamma \min)} \times 100, \quad \gamma \max = 108.2 \text{ pcf}, \quad \gamma \min = 91.5 \text{ pcf}$$

† The boring numbers and sample depth for each CR test is given in Table V of Reference 2.5-102

SSES-FSAR

TABLE 2.5-15

DESIGN PARAMETERS FOR SPRAY POND LINER

Liner		Total Seepage Loss*		Elevation of Ground Water Mound Beneath
Thickness (ft)	Permeability (ft/yr)	(ft3/yr)	Gal/30Days	Center of Pond
1.0	0.20	8.4 x 105	5.2 x 105	670
1.0	0.12	4.8 x 105	3.0 x 105	660
1.0	0.04	1.9 x 105	1.2 x 105	650

*Liquefaction design requirements require a seepage loss rate of less than 6.7 x 105 cubic feet per year (4.1 x 105 gal/30 days). The liner is designed for a seepage loss of no more than 3.0 x 105 gal/30 days.

TABLE 2.5-16

SPRAY POND
SUMMARY OF LIQUEFACTION ANALYSES

Profile 1 (West End of Pond)

Depth Ft.	Avg. Induced Cyclic Stress psf		Cyclic Shear Strength psf		Factor of Safety	
	GWT 650'	GWT 640'	GWT 650'	GWT 640'	GWT 650'	GWT 640'
1	12	12	149	149	12.4	12.4
3.5	43	43	206	206	4.8	4.8
6.5	80	80	273	273	3.4	3.4
10.5	128	128	359	359	2.80	2.80
15.5	186	186	460	460	2.47	2.47
23	267	267	558	601	2.09	2.25
33	370	371	651	732	1.76	1.97
44	478	480	746	820	1.56	1.71
56	590	594	840	908	1.42	1.53
69.5	705	710	938	997	1.33	1.40
85	806	816	1034	1085	1.28	1.33

TABLE 2.5-16 (Continued)

Profile 2 (Central Section, Pumphouse, etc.)

Depth	Avg. Induced Cyclic Stress psf			Cyclic Shear Strength psf			Factor of Safety		
Ft.	GWT 665'	GWT 660'	GWT 650'	GWT 665'	GWT 660'	GWT 650'	GWT 665'	GWT 660'	GWT 650'
1	18	12	12	162	149	149	9.0	12.4	12.4
3.5	49	43	43	207	206	206	4.2	4.8	4.8
6.5	80	80	80	239	273	273	3.0	3.4	3.4
10.5	128	128	128	282	334	359	2.20	2.61	2.80
15.5	185	185	186	337	388	460	1.82	2.10	2.47
23	265	266	267	418	465	558	1.58	1.75	2.09
33	367	368	370	519	565	651	1.41	1.54	1.76
43	464	466	470	615	657	738	1.33	1.41	1.57
52.5	553	555	560	700	739	815	1.26	1.33	1.46

TABLE 2.5-16 (Continued)

Profile 3 (East End of Pond)

Depth Ft.	Avg. Induced Cyclic Stress psf GWT 665'	Cyclic Shear Strength psf GWT 665'	Factor of Safety GWT 665'
1.5	18	162	9.0
4	49	207	4.2
6.5	80	239	3.0
10.5	128	282	2.20
15.5	185	337	1.82
19	223	375	1.68

Note:

1. Avg. Induced Cyclic Stresses = 0.65 Maximum Induced Cyclic Stresses (Ref. Seed and Idriss, 1971)

2. GWT = Ground Water Table, MSL.

3. Factor of Safety = Cyclic Shear Strength/Avg. Induced Cyclic Stress.

TABLE 2.5-17

EFFECT OF VARYING STANDARD RELATIONSHIP* OF EFFECTIVE STRAIN WITH DAMPING AND MODULI FOR PROFILE 2, GWT-665'.

Average Induced Cyclic Shear Stress, psf.

Depth Ft.	Std. Relation	30% Increase in Modulus: 30% Decrease In Damping	30% Increase in Modulus: No Change In Damping	30% Increase in Modulus: 30% Increase In Damping	No Change in Modulus: 30% Decrease In Damping	No Change in Modulus: 30% Increase In Damping	30% Decrease in Modulus: 30% Decrease In Damping	30% Decrease in Modulus: No Change In Damping	30% Decrease in Modulus: 30% Increase In Damping
1.5	18	18	18	18	18	18	18	18	18
4	49	50	50	49	50	49	49	49	49
6.5	80	80	80	80	80	80	80	80	79
10.5	128	129	129	128	128	127	127	126	125
15.5	185	188	187	186	185	184	182	181	179
23	265	271	270	268	266	263	259	258	258
33	367	376	374	372	368	368	357	355	355
43	464	476	474	475	466	461	448	445	441
52.5	553	570	567	564	556	549	522	519	516

Factor of Safety Against Liquefaction

Depth Ft.	Shear Strength psf	Std. Relation	30% Increase in Modulus: 30% Decrease In Damping	30% Increase in Modulus: No Change In Damping	30% Increase in Modulus: 30% Increase In Damping	No Change in Modulus: 30% Decrease In Damping	No Change in Modulus: 30% Increase In Damping	30% Decrease in Modulus: 30% Decrease In Damping	30% Decrease in Modulus: No Change In Damping	30% Decrease in Modulus: 30% Increase In Damping
1.5	162	9.0	9.0	9.0	9.0	9.0	9.0	9.0	9.0	9.0
4	207	4.2	4.1	4.1	4.2	4.1	4.2	4.2	4.2	4.2
6.5	239	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
10.5	282	2.20	2.19	2.19	2.20	2.20	2.22	2.22	2.24	2.26
15.5	337	1.82	1.79	1.80	1.81	1.82	1.83	1.85	1.86	1.88
23	418	1.58	1.54	1.55	1.56	1.57	1.59	1.61	1.62	1.62
33	519	1.41	1.38	1.39	1.40	1.41	1.41	1.45	1.46	1.46
43	615	1.33	1.29	1.30	1.29	1.32	1.33	1.37	1.38	1.39
52.5	700	1.26	1.23	1.23	1.24	1.26	1.28	1.34	1.35	1.36

*Seed, H. B. and Idriss, I. M., "Soil Moduli and Damping Factors for Dynamic Response Analysis," EERC Report No. 70-10, U. C. Berkeley, 1970.

GWT = Ground Water Table, MSL.

Factor of Safety = Shear Strength/Average Induced Cyclic Stress.

TABLE 2.5-18

SPRAY POND

FACTOR OF SAFETY AS OBTAINED USING DIFFERENT EARTHQUAKES
(ALL 3 PROFILES AND AT DIFFERENT GROUND WATER LEVELS)

Profile 1 (West End of Pond, 93' Overburden Below Bottom of Pond)*

Factor of Safety at Corresponding Depth

Depth Below Pond Bottom Ft.	EARTHQUAKE: Bechtel Synthetic		Golden Gate		Helena		Parkfield	
	Ground Water Elevation, Ft. (MSL)							
	650**	640	650**	640	650**	640	650**	640
1	12.4	12.4	11.5	11.5	12.4	12.4	11.4	11.4
3.5	4.8	4.8	4.5	4.5	4.9	4.9	4.6	4.6
6.5	3.4	3.4	3.3	3.3	3.5	3.5	3.2	3.2
10.5	2.80	2.80	2.74	2.69	3.0	3.0	2.65	2.63
15.5	2.47	2.47	2.44	2.39	2.95	2.95	2.30	2.30
23	2.09	2.25	2.08	2.15	2.89	3.0	1.90	2.04
33	1.76	1.97	1.79	1.90	2.37	2.79	1.58	1.78
44	1.56	1.71	1.67	1.71	2.17	2.41	1.43	1.58
56	1.42	1.53	1.63	1.64	2.19	2.36	1.39	1.50
69.5	1.33	1.40	1.68	1.64	1.98	2.22	1.42	1.50
85	1.28	1.33	1.79	1.72	1.91	2.05	1.48	1.52

TABLE 2.5-18 (Continued)

Profile 2 (Central Section, Pumphouse, etc., 57' Overburden below Bottom of Pond)*

Factor of Safety at Corresponding Depth

EARTHQUAKE

Depth Below Pond Bottom Ft.	Bechtel Synthetic			Golden Gate			Helena			Parkfield		
	Ground Water Elevation, Ft. (MSL)											
	665**	660	650	665**	660	650	665**	660	650	665**	660	650
1	9.0	12.0	12.4	8.1	12.4	11.5	8.1	12.4	11.5	8.1	11.5	11.5
3.5	4.2	4.8	4.8	4.0	4.5	4.5	4.1	4.7	4.8	4.0	4.6	4.6
6.5	3.0	3.4	3.4	2.85	3.4	3.4	2.95	3.4	3.5	2.92	3.3	3.3
10.5	2.20	2.61	2.80	2.16	2.57	2.78	2.27	2.63	2.80	2.13	2.53	2.72
15.5	1.82	2.10	2.47	1.79	2.09	2.46	1.96	2.19	2.54	1.75	2.04	2.39
23	1.58	1.75	2.09	1.59	1.79	2.12	1.84	1.96	2.26	1.48	1.69	2.02
33	1.41	1.54	1.76	1.51	1.64	1.87	1.80	1.96	2.04	1.35	1.49	1.74
43	1.33	1.41	1.57	1.52	1.62	1.79	1.81	1.96	2.03	1.33	1.47	1.61
52.5	1.26	1.33	1.46	1.59	1.67	1.79	1.91	2.01	2.05	1.37	1.44	1.55

TABLE 2.5-18 (Continued)

Profile 3 (East End of Pond, 20' Overburden below Bottom of Pond)*

Factor of Safety at Corresponding Depth

EARTHQUAKE

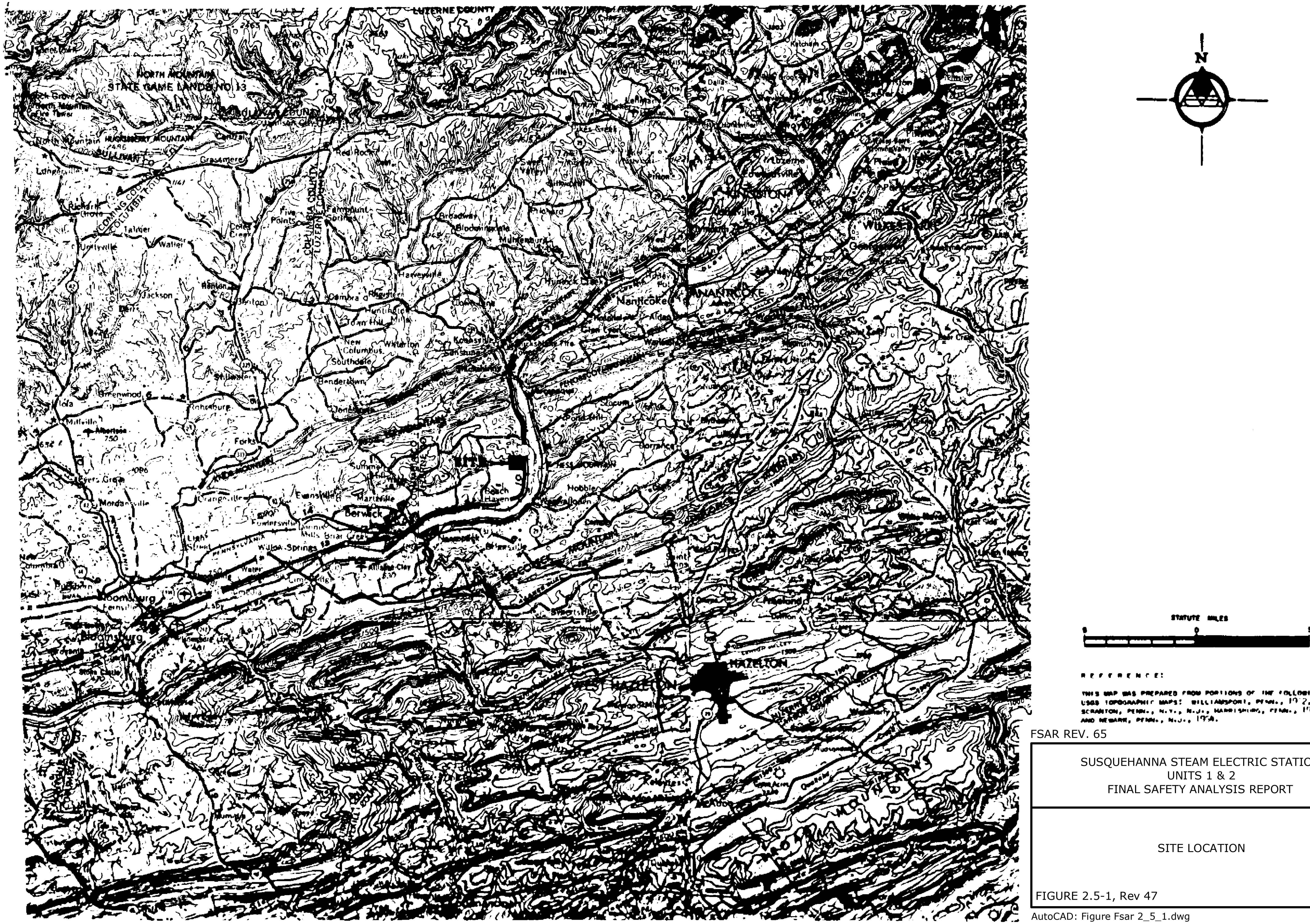
Depth Below Pond Bottom Ft.	Bechtel Synthetic	Golden Gate	Helena	Parkfield
Ground Water Elevation = 665 Ft. (MSL)**				
1.5	9.0	8.5	8.1	9.0
4	4.2	4.1	4.1	4.4
6.5	3.0	3.0	2.95	3.0
10.5	2.20	2.26	2.31	2.27
15.5	1.82	1.92	2.10	1.87
19	1.68	1.84	2.13	1.71

*See Figure 2.5-48 for Soil Profile
**Maximum Water Table

TABLE 2.5 - 19

PROBABLE SETTLEMENT DURING EARTHQUAKE

		RESULTED FROM COMPACTION OF DRY SOILS			RESULTED FROM RECONSOLIDATION OF SATURATED SOILS						
Depth Ft.	Thickness Ft.	Ave. Induced Cyclic Strain %	Probable Vert. Strain %	Probable Vert. Settlement 10^{-2}, ft.	Stress Ratio Developed	Cycle Ratio. N/N_1	Pore Pressure Ratio. $\Delta u/\sigma_{3c}$	Volumetric Strain (Dr=50%), %	Vol. Strain Corrected to Dr=60%, %	Vertical Strain %	Probable Vert. Settlement 10^{-2}, ft.
PROFILE 1	(East End of Pond, Ground Water Table = 650')										
1	2	.0003	.0001	.0002	.026	.005	0 - .01	0	0	0	0
3.5	3	.0009	.0008	.0024	.066	.005	0 - .01	0	0	0	0
6.5	3	.0016	.0011	.0033	.092	.005	0 - .01	0	0	0	0
10.5	5	.0024	.0016	.0080	.110	.005	0 - .01	0	0	0	0
15.5	5	.0031	.0025	.0125	.121	.005	0 - .01	0	0	0	0
23	10	.0043	.0035	.0350	.139	.025	.02 - .19	0 - .12	0 - .096	0 - .048	0 - 0.48
33	10	.0056	.0045	.0450	.161	.17	.15 - .41	.02 - .32	.016 - .256	.008 - .128	.080 - 1.28
44	12	.0065	.0055	.0660	.175	.18	.16 - .42	.02 - .33	.016 - .264	.008 - .132	.096 - 1.58
56	12	.0072	.0062	.0744	.185	.25	.22 - .49	.04 - .40	.032 - .320	.016 - .160	.192 - 1.92
69.5	15	.0073	.0063	.0945	.190	.25	.22 - .49	.04 - .40	.032 - .320	.016 - .160	.240 - 2.40
85	16	.0071	.0061	.0976	.187	.25	.22 - .49	.04 - .40	.032 - .320	.016 - .160	.256 - 2.56
Total	33 ft.			.004389 ft (0.05 in)							.009 - 0.10 ft (0.1 - 1.2 in)
PROFILE 2	(Central Section, Pump House, etc., Ground Water Table = 665')										
1.5	3	.0004	.0001	.0003	.036	.005	0 - .01	0	0	0	0
4	2	.0011	.0009	.0018	.075	.005	0 - .01	0	0	0	0
6.5	3	.0018	.0011	.0033	.107	.005	0 - .01	0	0	0	0
10.5	5	.0027	.0020	.0100	.142	.062	.06 - .23	.01 - .17	.008 - .136	.004 - .068	.020 - 0.34
15.5	5	.0033	.0028	.0140	.170	.17	.15 - .41	.02 - .32	.016 - .256	.008 - .128	.040 - 0.64
23	10	.0050	.0038	.0380	.191	.26	.23 - .51	.04 - .41	.032 - .328	.016 - .164	.160 - 1.64
33	10	.0062	.0049	.0490	.207	.33	.30 - .57	.05 - .48	.040 - .384	.020 - .192	.200 - 1.92
43	10	.0074	.0064	.0640	.214	.36	.32 - .60	.05 - .52	.040 - .416	.020 - .208	.200 - 2.08
52.5	9	.0076	.0065	.0585	.221	.42	.33 - .65	.06 - .53	.048 - .424	.024 - .232	.216 - 2.08
Total	57 ft.			.002389 ft (0.03 in)							.008 - .087 ft (0.1 - 1.0 in)
PROFILE 3	(West End of Pond, Ground Water Table = 665')										
1.5	3	.0004	.0001	.0003	.036	.005	0 - .01	0	0	0	0
4	2	.0011	.0009	.0018	.075	.005	0 - .01	0	0	0	0
5.5	3	.0018	.0011	.0033	.107	.005	0 - .01	0	0	0	0
10.5	5	.0027	.0020	.0100	.142	.062	.06 - .28	.01 - .17	.008 - .136	.004 - .068	.020 - 0.34
15.5	5	.0033	.0028	.0140	.170	.17	.15 - .41	.02 - .32	.016 - .256	.008 - .128	.040 - 0.64
19	2	.0045	.0036	.0072	.181	.23	.20 - .47	.04 - .39	.032 - .312	.016 - .156	.032 - 0.31
Total	20 ft.			.000366 ft (0.01 in)							.0009 - .013 ft (0.01 - 0.2 in)



REFERENCE:

THIS MAP WAS PREPARED FROM PORTIONS OF THE FOLLOWING USGS TOPOGRAPHIC MAPS: WILLIAMSPORT, PENN., 1972, SCRANTON, PENN., N.Y., N.J., HARRISBURG, PENN., 1972 AND NEWARK, PENN., N.J., 1974.

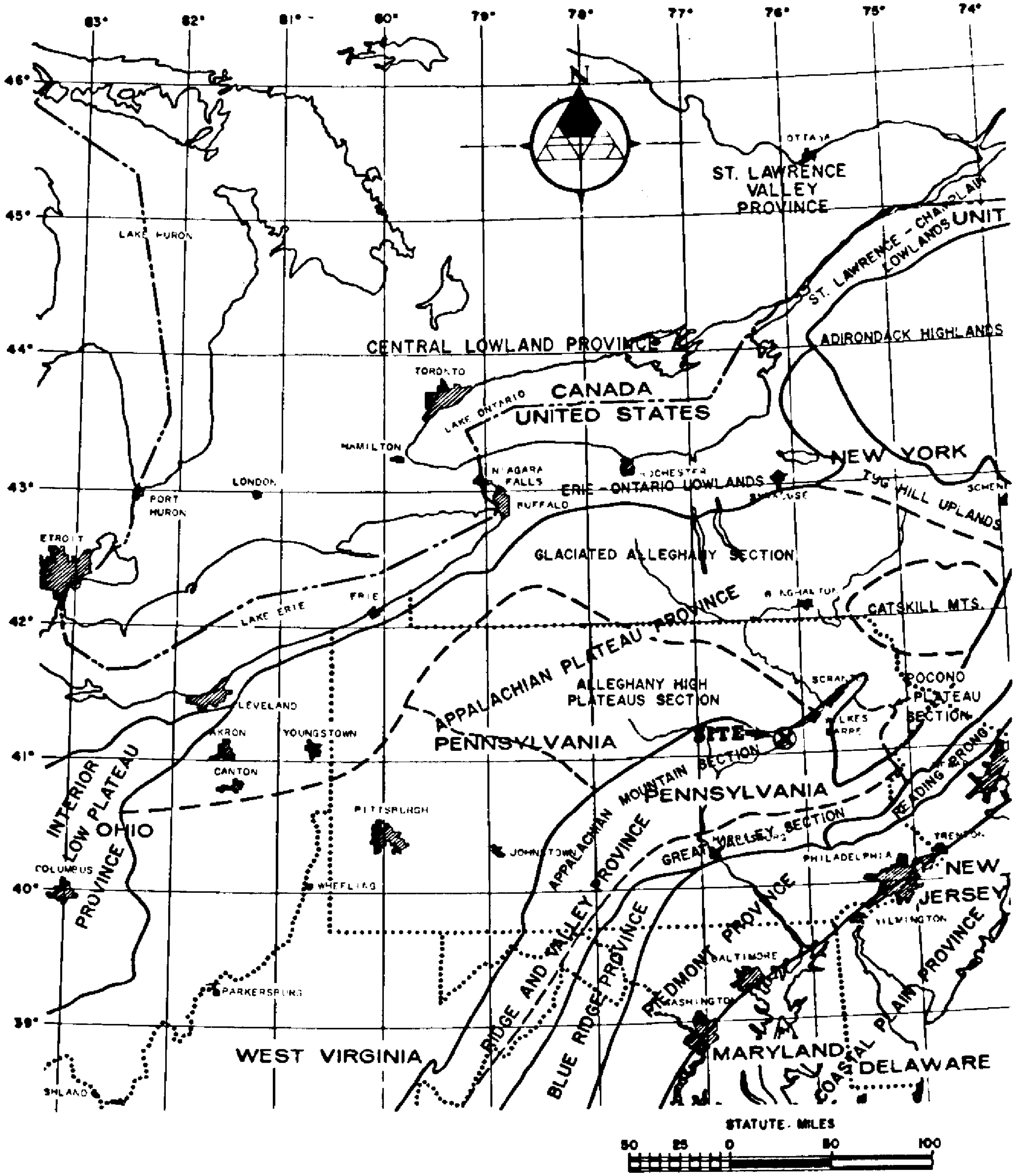
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SITE LOCATION

FIGURE 2.5-1, Rev 47

AutoCAD: Figure Fsar 2_5_1.dwg



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SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

REGIONAL PHYSIOGRAPHIC MAP

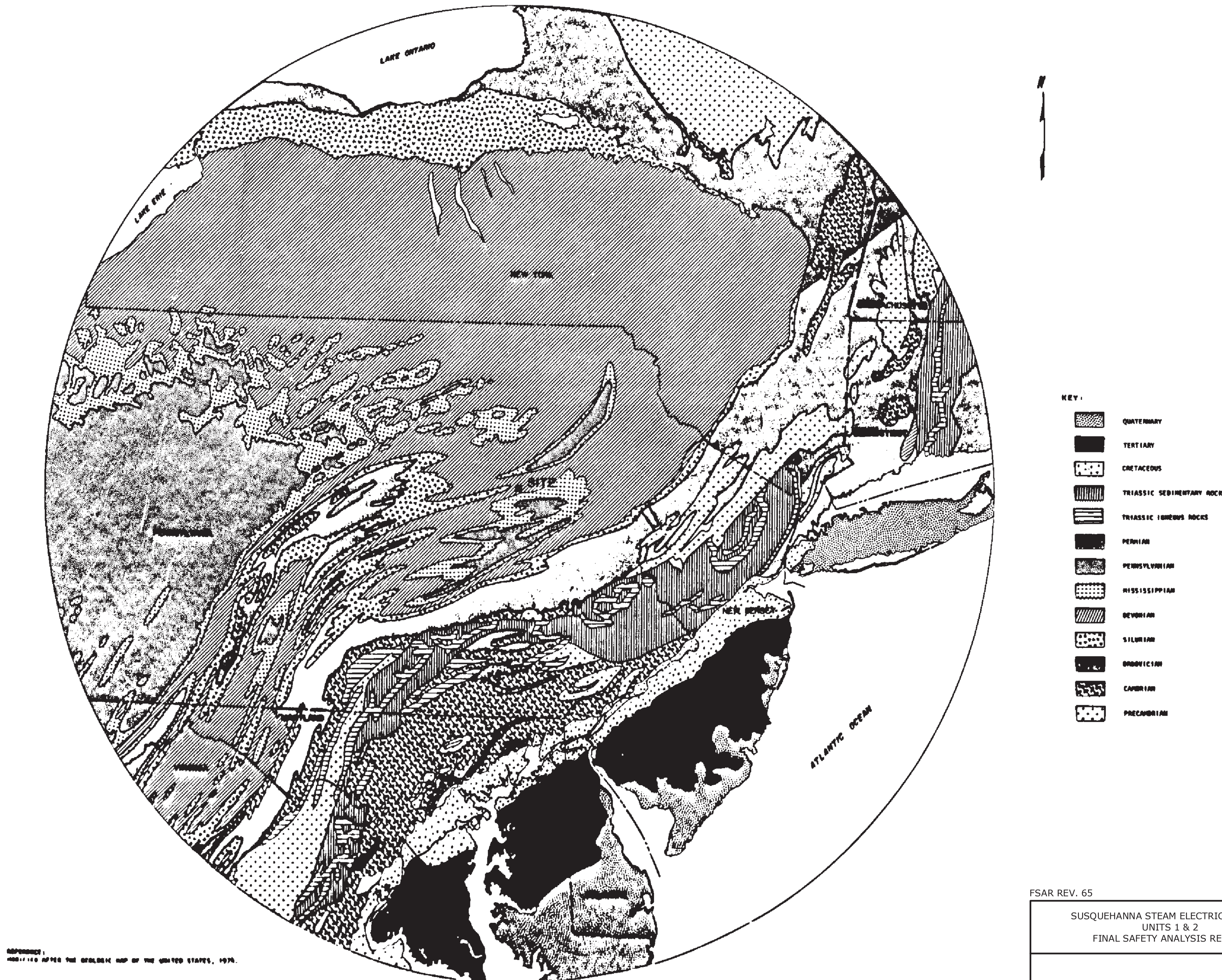
FIGURE 2.5-2, Rev 47

AutoCAD: Figure Fsar 2_5_2.dwg

REFERENCE:

THIS MAP WAS PREPARED FROM A PORTION OF THE U.S.G.S. "IFR WALL PLANNING CHART, EAST AND WEST, 1968".

PHYSIOGRAPHIC DIVISIONS TAKEN FROM PHYSIOGRAPHY OF EASTERN UNITED STATES BY N.M. FENNEMAN, 1938.



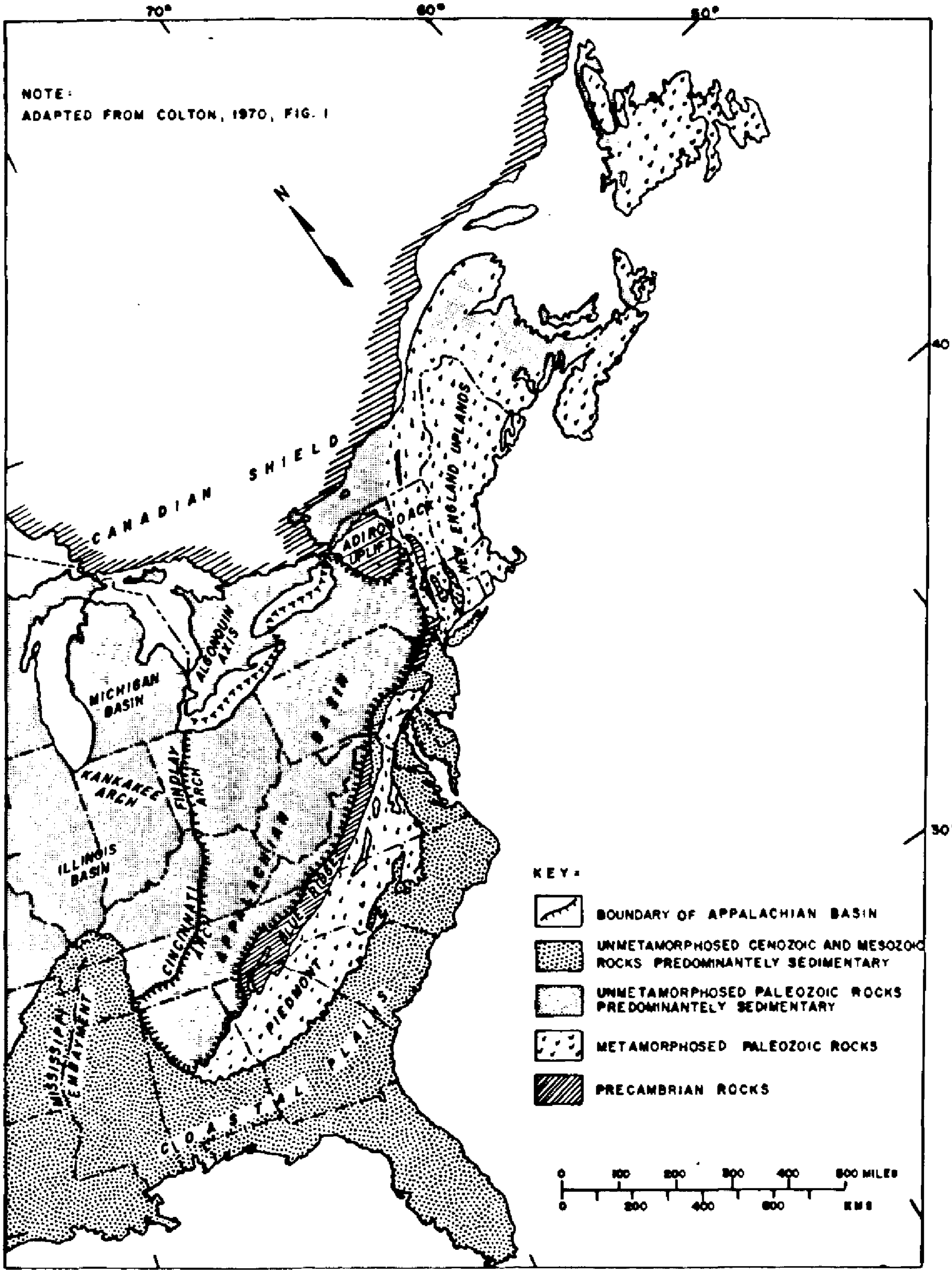
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

REGIONAL GEOLOGIC MAP

FIGURE 2.5-3, Rev 47

AutoCAD: Figure Fsar 2_5_3.dwg



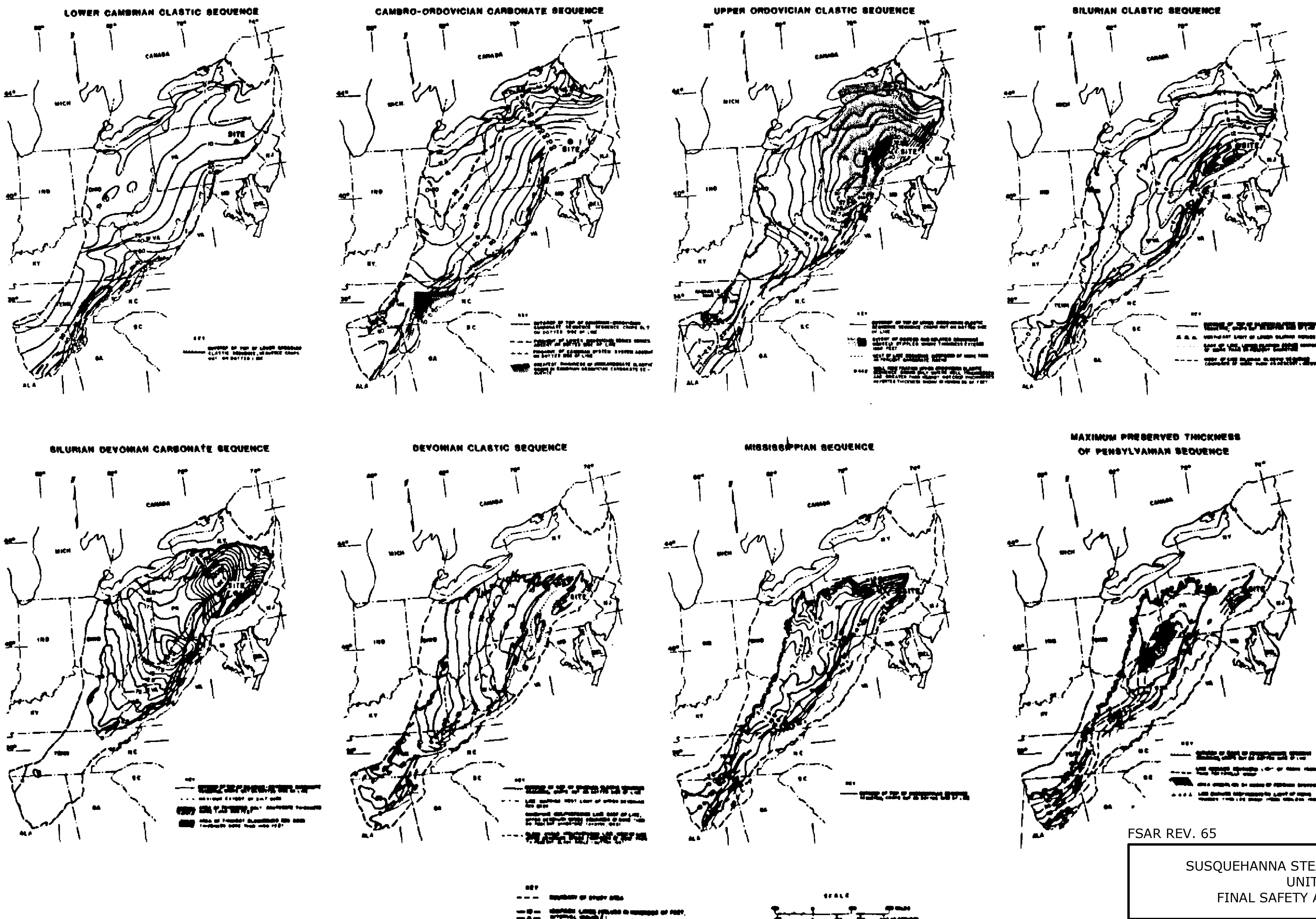
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOCATION OF THE
APPALACHIAN BASIN

FIGURE 2.5-4, Rev 47

AutoCAD: Figure Fsar 2_5_4.dwg



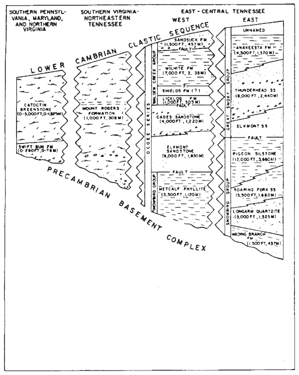
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

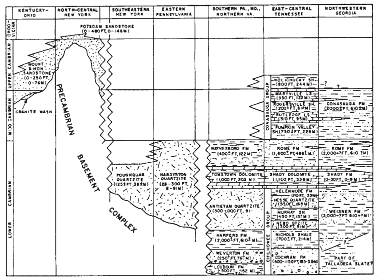
REGIONAL ISOPACH MAPS

FIGURE 2.5-5, Rev 47

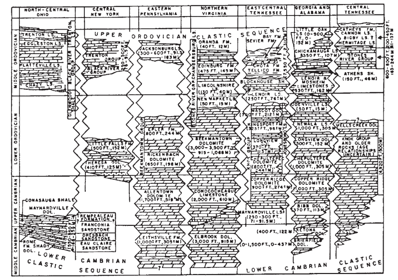
AutoCAD: Figure Fsar 2_5_5.dwg



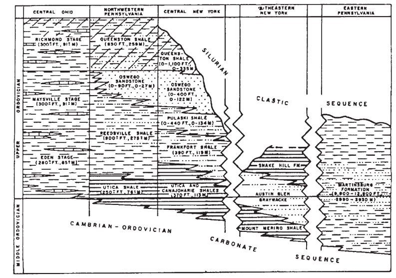
PRECAMBRIAN STRATIFIED SEQUENCE



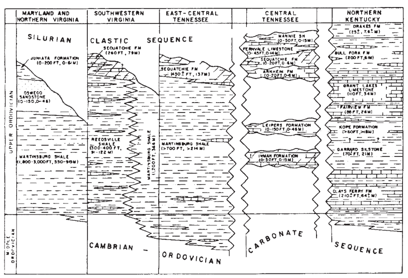
LOWER CAMBRIAN CLASTIC SEQUENCE



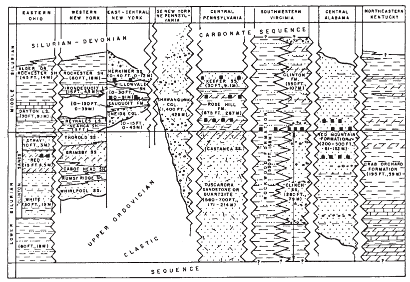
CAMBRO- ORDOVICIAN CARBONATE SEQUENCE



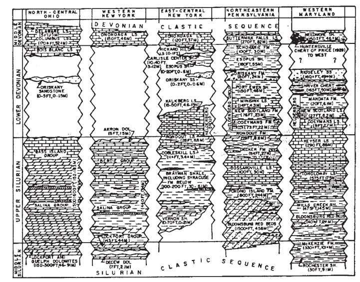
UPPER ORDOVICIAN CLASTIC SEQUENCE IN NORTHERN APPALACHIAN BASIN



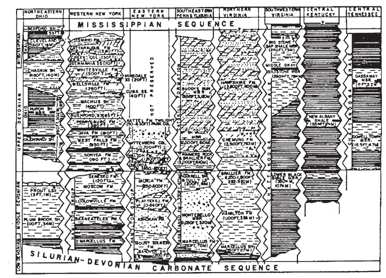
UPPER ORDOVICIAN CLASTIC SEQUENCE IN SOUTHERN APPALACHIAN BASIN



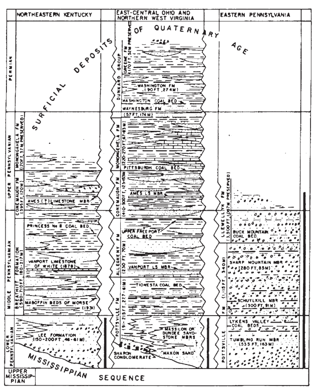
SILURIAN CLASTIC SEQUENCE



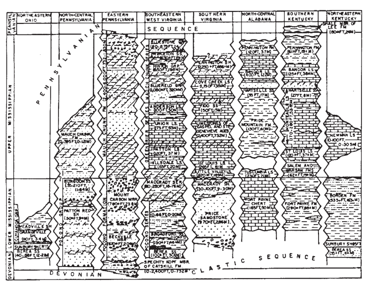
SILURIAN- DEVONIAN CARBONATE SEQUENCE



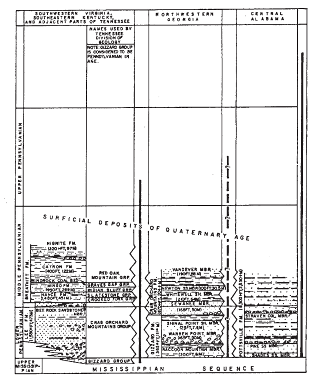
DEVONIAN CLASTIC SEQUENCE



MISSISSIPPIAN SEQUENCE

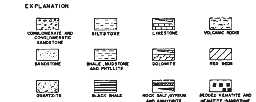


PENNSYLVANIAN SEQUENCE IN NORTHERN APPALACHIAN BASIN



PENNSYLVANIAN SEQUENCE IN THE SOUTHERN APPALACHIAN BASIN

EXPLANATION



NOTES:

BREAKS BETWEEN COLUMNS INDICATE UNCERTAINTY OF LATERAL RELATIONSHIPS;
SOLID BARS INDICATE RELATIVE THICKNESS OF SECTIONS

REFERENCE:

ADAPTED FROM COLTON, 1970, FIGURES 9, 10, 12, 14, 15, 17, 19, 21, 23, 25 & 26

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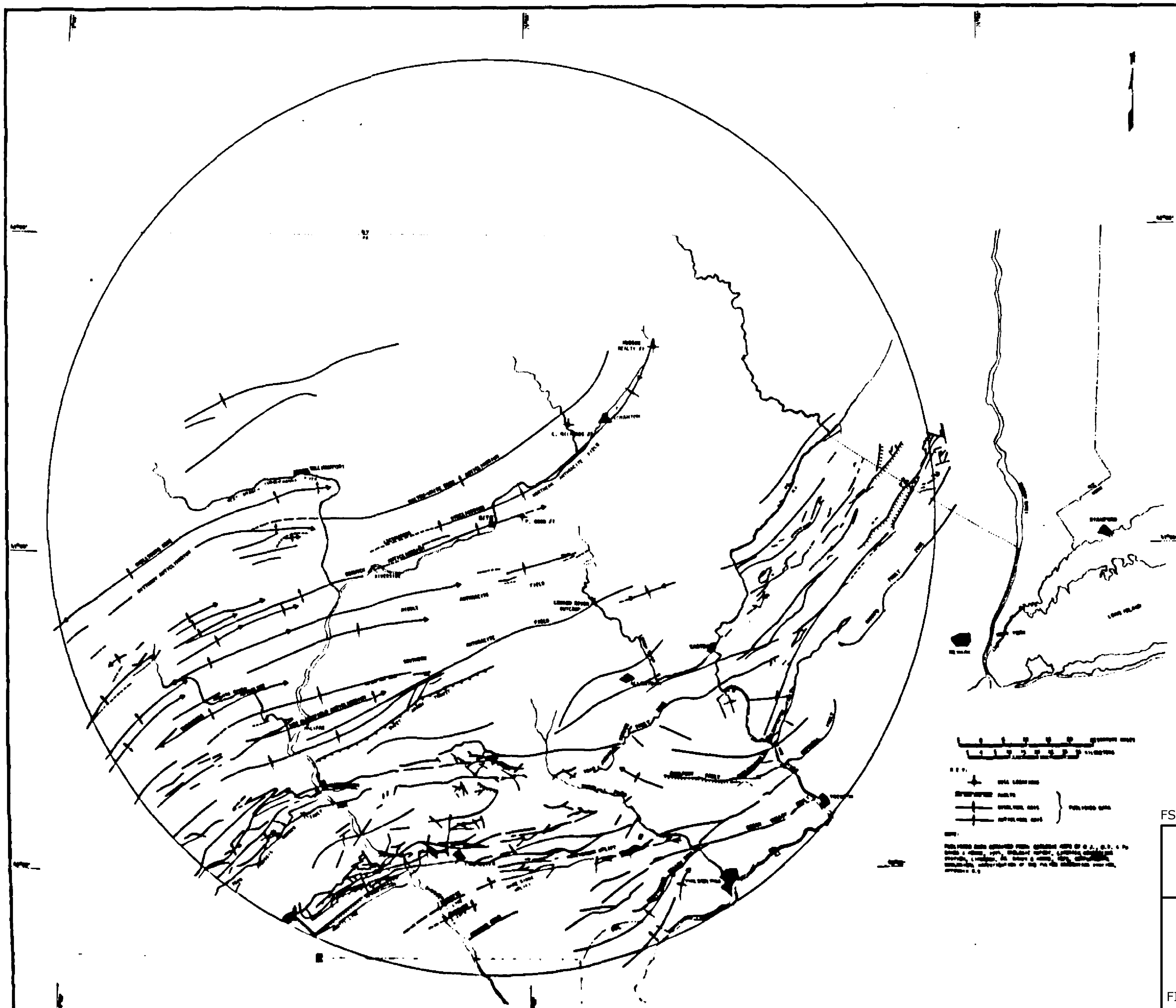
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

STRATIGRAPHIC COLUMNS

FIGURE 2.5-6, Rev 47

AutoCAD: Figure Fsar 2_5_6.dwg

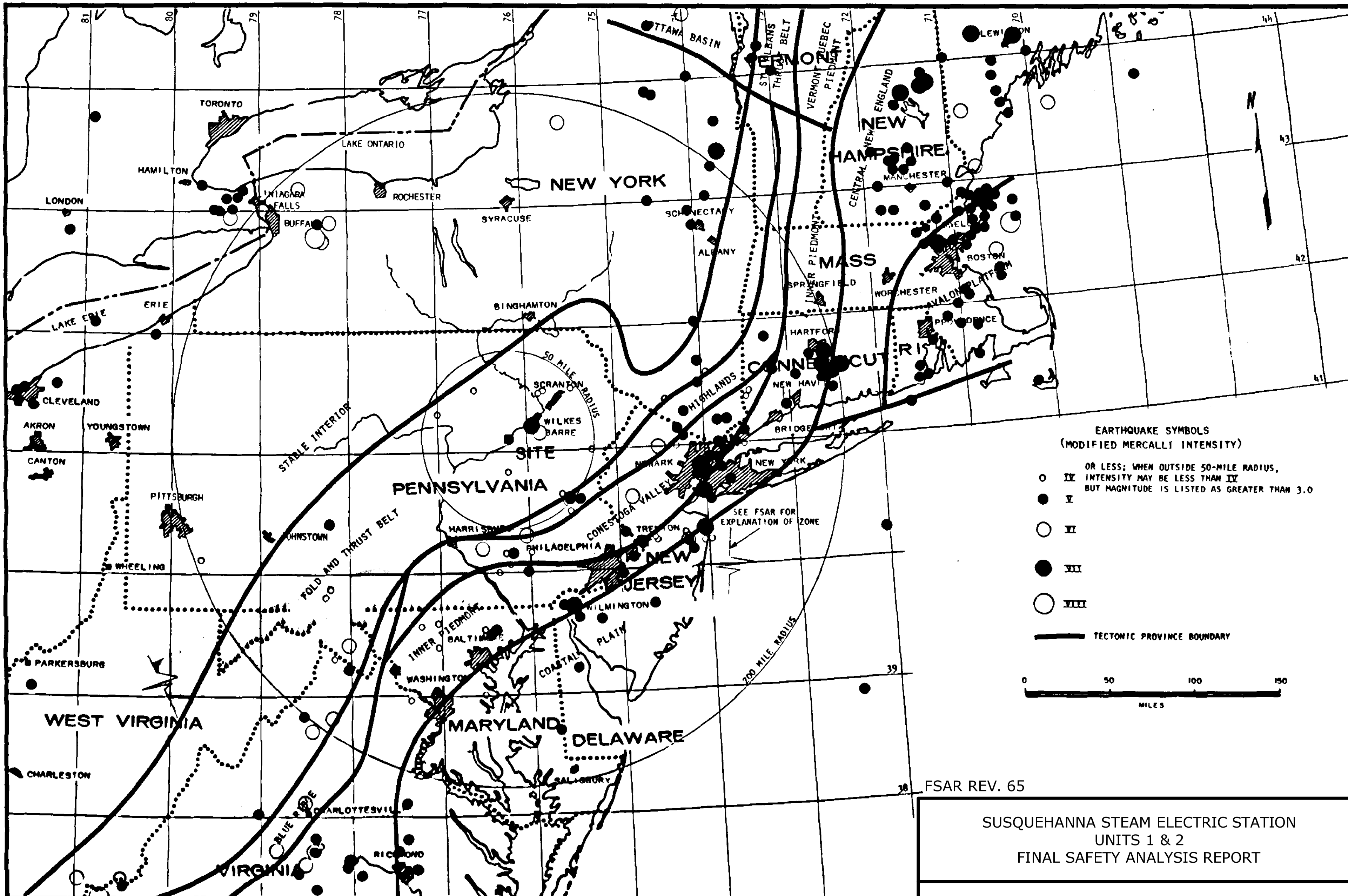


SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

REGIONAL TECTONIC MAP

FIGURE 2.5-7, Rev 47

AutoCAD: Figure Fsar 2_5_7.dwg



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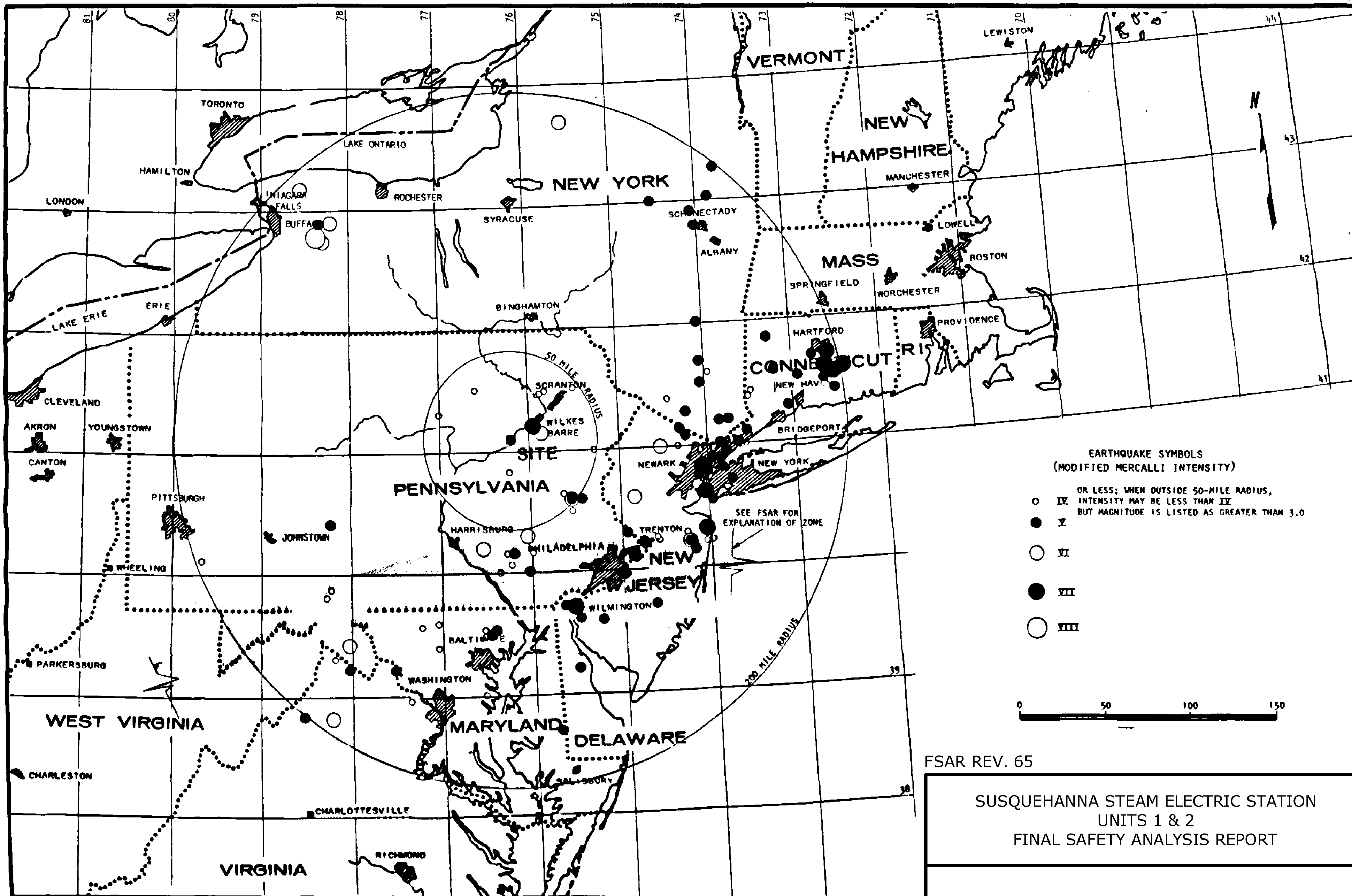
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

EARTHQUAKE EPICENTERS AND TECTONIC
PROVINCE BOUNDARIES
IN THE SITE REGION

FIGURE 2.5-8A, Rev 47

AutoCAD: Figure Fsar 2_5_8A.dwg



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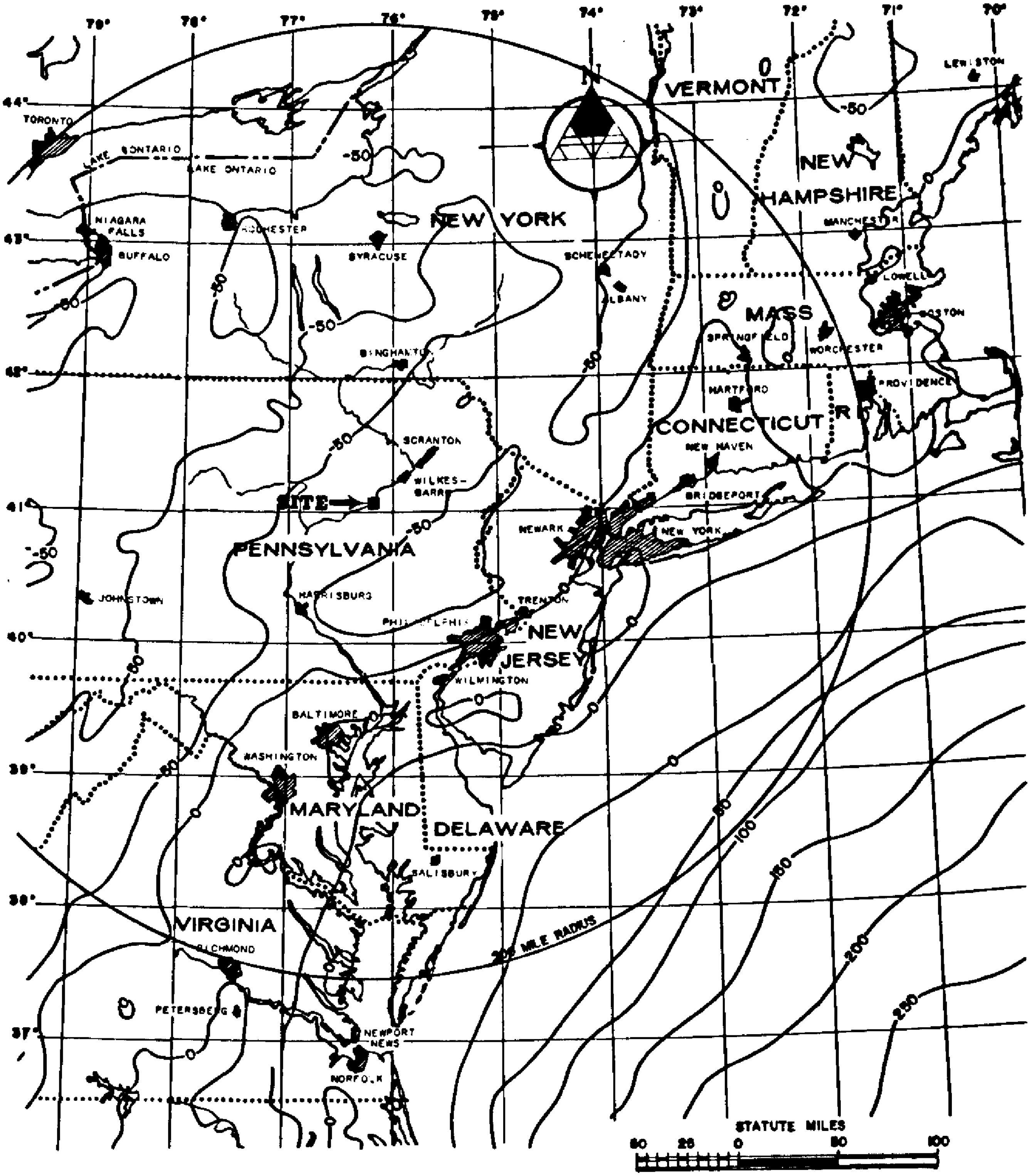
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

EARTHQUAKE EPICENTERS
WITHIN 200 MILES OF THE SITE

FIGURE 2.5-8B, Rev 47

AutoCAD: Figure Fsar 2_5_8B.dwg



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CONTOUR INTERVAL = 50 MILLIGALS

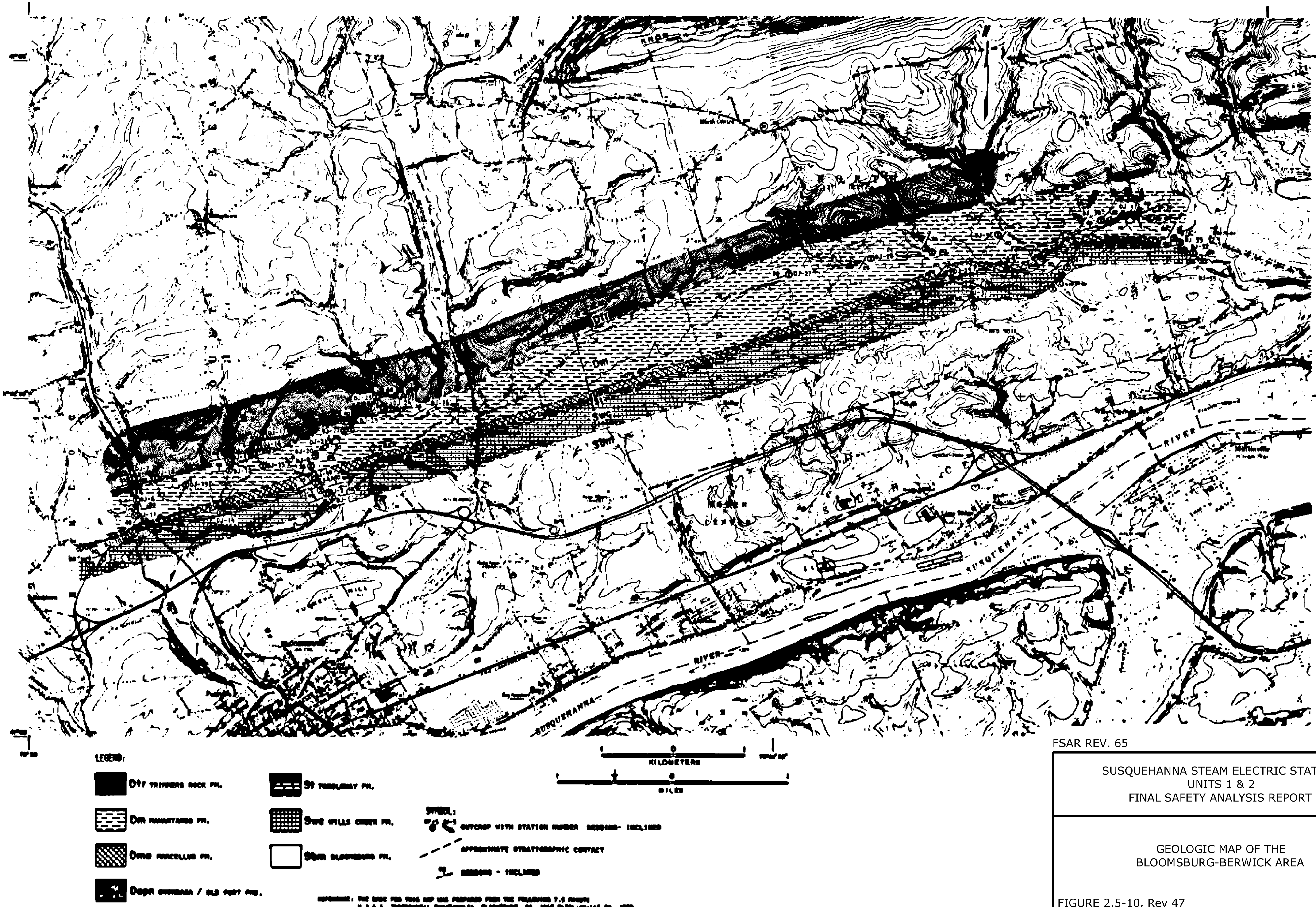
REFERENCE:

THIS MAP WAS PREPARED FROM A PORTION OF THE U.S.G.S. "IFR WALL PLANNING CHART, EAST AND WEST, 1968".

THE "BOUGUER GRAVITY ANOMALY MAP OF THE UNITED STATES" BY THE AMERICAN GEOPHYSICAL UNION AND THE U.S. GEOLOGICAL SURVEY.

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
BOUGUER GRAVITY ANOMALY MAP
FIGURE 2.5-9, Rev 47

AutoCAD: Figure Fsar 2_5_9.dwg



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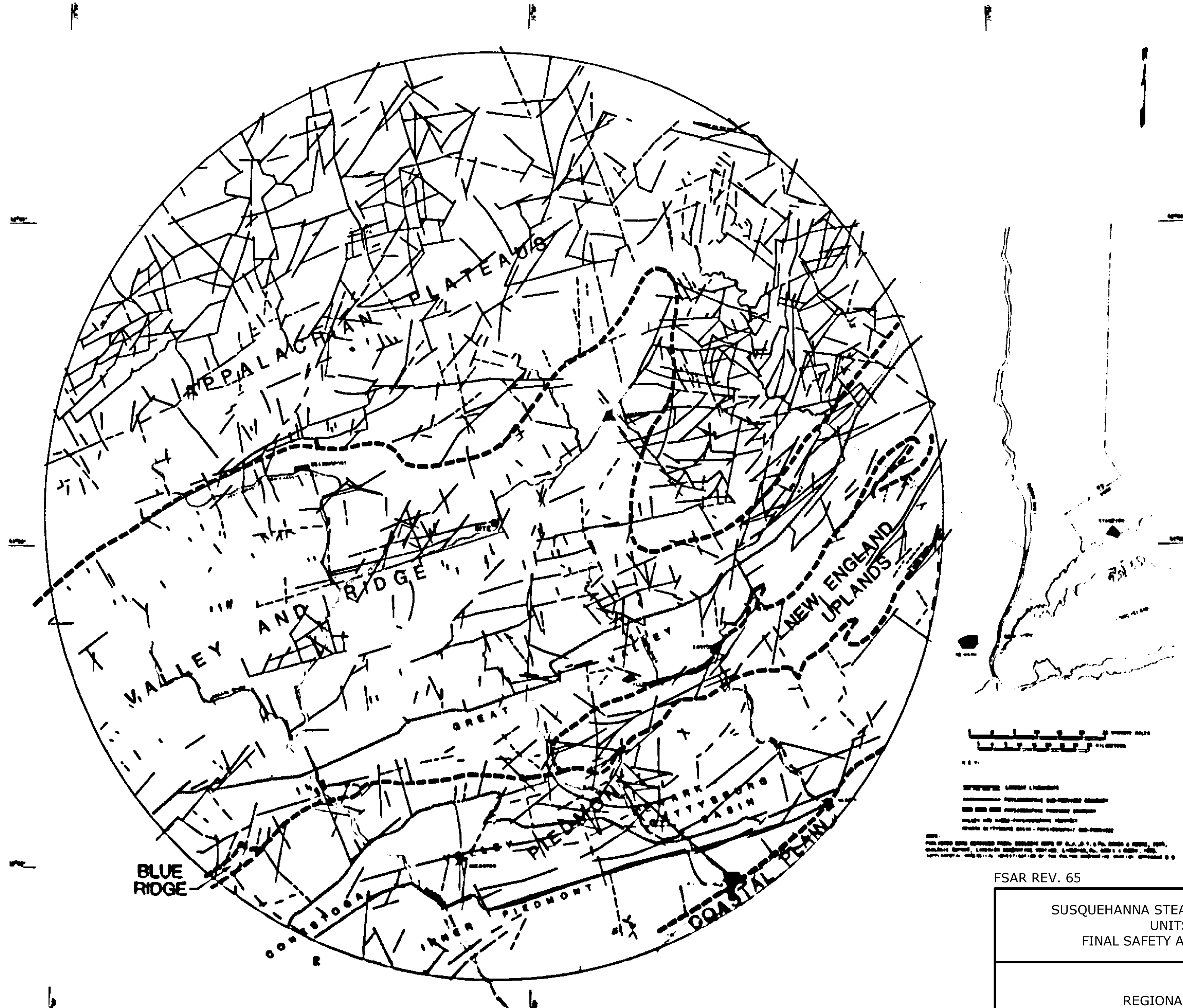
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

GEOLOGIC MAP OF THE
BLOOMSBURG-BERWICK AREA

FIGURE 2.5-10, Rev 47

AutoCAD: Figure Fsar 2_5_10.dwg



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SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

REGIONAL LANDSAT
LINEAMENTS

FIGURE 2.5-11, Rev 47

AutoCAD: Figure Fsar 2_5_11.dwg

N

REFERENCE:

THE BASE FOR THIS MAP WAS PREPARED FROM THE FOLLOWING 7.5 MINUTE U.S.G.S. TOPOGRAPHIC QUADRANGLES: BERWICK, PA., 1969; MIFFLINVILLE, PA., 1969; NANTICOKE, PA., 1969; SHICKSHINNY, PA., 1969; SYBERTSVILLE, PA., 1969.

SYMBOLS :

SOUTHERN EXTENT OF WISCONSIN DRIFT AS MAPPED BY GRAY AND OTHERS, 1960. GEOLOGIC MAP OF PENNSYLVANIA.

JW-8 OUTCROP WITH STATION NUMBER

APPROXIMATE STRATIGRAPHIC CONTACT

BEDDING INCLINED, VERTICAL, HORIZONTAL, OVERTURNED

ANTICLINE, SYNCLINE

MINOR FOLDS-ANTICLINE, SYNCLINE

LEGEND:

Qal QUATERNARY ALLUVIUM

Qkt PLEISTOCENE KAME TERRACE

PENNSYLVANIAN

IPp POTTSVILLE AND POST-POSTSVILLE FORMATIONS

MISSISSIPPIAN-PENNSYLVANIAN

Mmc MAUCH CHUNK FORMATION

MISSISSIPPIAN

Mp POCONO FORMATION

DEVONIAN

Dck CATSKILL FORMATION

Dtr TRIMMERS ROCK FORMATION

Dm MAHANTANGO FORMATION (INCLUDES MARCELLUS, MAHANTANGO, TULLY AND HARRELL FMS.)

1 0 1
MILE

1 0 1
KILOMETER

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SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SITE GEOLOGIC MAP

FIGURE 2.5-12, Rev 47

AutoCAD: Figure Fsar 2_5_12.dwg

Looking south into large pothole in Unit 1 turbine area.
Workmen performing final clean-up operations provide scale.

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SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

PHOTOGRAPH OF GEOLOGIC
FEATURE FOUND IN TURBINE
BUILDING EXCAVATION

FIGURE 2.5-13, Rev 47

AutoCAD: Figure Fsar 2_5_13.dwg

ERA	PERIOD		FORMATION	THICKNESS IN FEET	LOCATION OF SURFACE EXPOSURE	TECTONIC SIGNIFICANCE
CENOZOIC	HOLOCENE			10 - 20		
CENOZOIC	PLEISTOCENE			0 - 300		
PALEOZOIC	PENNSYLVANIAN		LLEWELLYN OR "POST POTTSVILLE" POTTSVILLE CONGLOMERATE	1800 ± 500 ±	ANTHRACITE REGION	UPPER PORTION GREATLY FOLDED, FAULTED
PALEOZOIC	MISSISSIPPIAN		MAUCH CHUNK SHALE POCONO SANDSTONE	2000 ± 600 ±	ANTHRACITE REGION	FOLDED, THRUST FAULTS
PALEOZOIC	DEVONIAN	UPPER	CATSKILL SHALE AND SANDSTONE TRIMMERS ROCK SANDSTONE	1700 ± 2100 ±	VALLEY AND RIDGE AND PLATEAU VALLEY AND RIDGE AND PLATEAU	COMPETENT, LARGE OPEN FOLDS, FEW FAULTS
PALEOZOIC	DEVONIAN	MIDDLE	MAHANTANGO SHALE INCLUDING: HARRELL AND TULLY EQUIVALENTS, SHERMAN CREEK AND MARCELLUS SHALES	2000 ±	VALLEY AND RIDGE AND PLATEAU VALLEY AND RIDGE AND PLATEAU	DECOLLEMENT (?) IN MARCELLUS, GENERALLY TIGHT FOLDS, FAULTED
PALEOZOIC	DEVONIAN	LOWER	ONONDAGA LIMESTONE OLD PORT SANDSTONE KEYSER LIMESTONE	165 ± 100 ± 200 ±	VALLEY AND RIDGE VALLEY AND RIDGE VALLEY AND RIDGE	MODERATELY COMPETENT, GENERALLY LONG OPEN FOLDS
PALEOZOIC	SILURIAN	UPPER	TONOLOWAY LIMESTONE WILLS CREEK SHALE	100 ± 350 ±	VALLEY AND RIDGE VALLEY AND RIDGE	DECOLLEMENT UNDER PLATEAU IN EQUIVALENT SALINA GROUP
PALEOZOIC	SILURIAN	MIDDLE	BLOOMSBURG SHALE CLINTON SANDSTONE	500 ± 640 ±	VALLEY AND RIDGE VALLEY AND RIDGE	MODERATELY COMPETENT, GENERALLY LONG OPEN FOLDS
PALEOZOIC	SILURIAN	LOWER	TUSCARORA SANDSTONE	250 ±	VALLEY AND RIDGE	
PALEOZOIC	CAMBRO-ORDOVICIAN		CAMBRO-ORDOVICIAN CLASTICS AND CARBONATES	8000 - 12000	GREAT VALLEY ANTICLINAL CORES ALONG PA. CULMINATION	DECOLLEMENTS IN UPPER PORTION (MARTINSBURG) AND LOWER CAMBRIAN
PRE-CAMBRIAN	PRECAMBRIAN BASEMENT		APPROXIMATELY 30,000 FEET BELOW SURFACE AT SITE			NOT INVOLVED IN VALLEY AND RIDGE AND PLATEAU TECTONICS

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SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

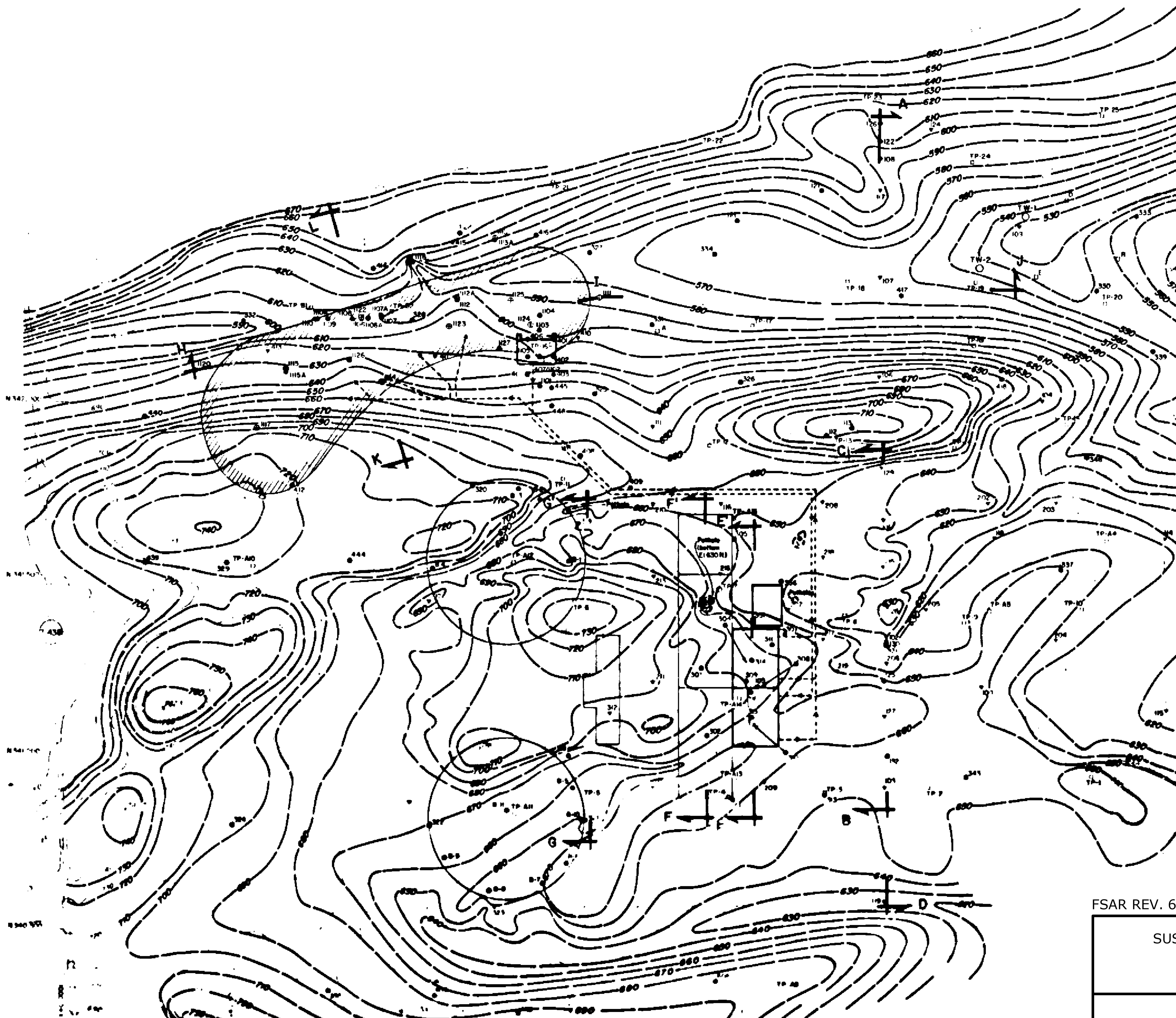
SITE VICINITY GEOLOGIC COLUMN

FIGURE 2.5-14, Rev 47

AutoCAD: Figure Fsar 2_5_14.dwg

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
GEOLOGIC MAP OF SPRAY POND AREA FIGURE 2.5-15



EXPLANATION

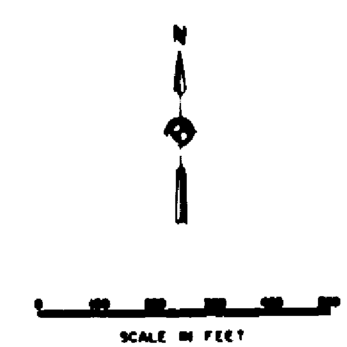
- 610 — Contour on top of rock (see note 2); contour interval 10 feet.
- Bedrock outcrop; may include minor amounts of talus or overburden. Boundaries are approx.
- 311, 504 — Exploration hole; vertical hole (left) and angle hole showing trace of hole projected to surface (right).
- 208 — Exploration hole utilized for water level observation.
- 1111 — Exploration hole used for permeability measurements.
- TW-1 — Water well (used for construction).
- A, TP 11 — Test pits.
- E E' — Location of line of geologic section (see note 3).

Seismic Category I facilities

- Category I structure.
- Category I pipeline.
- Category I impoundment.

NOTES:

1. Screened topographic base shows preconstruction terrain; contour interval 10 feet.
2. Top-of-rock contours show original top of rock surface, and are based on interpretation of drill hole and seismic refraction data, modified where appropriate by information obtained during construction. Contours north of town road T-419 are not shown.



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SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

CONTOURS ON TOP
OF
BEDROCK SURFACE

FIGURE 2.5-17, Rev 47

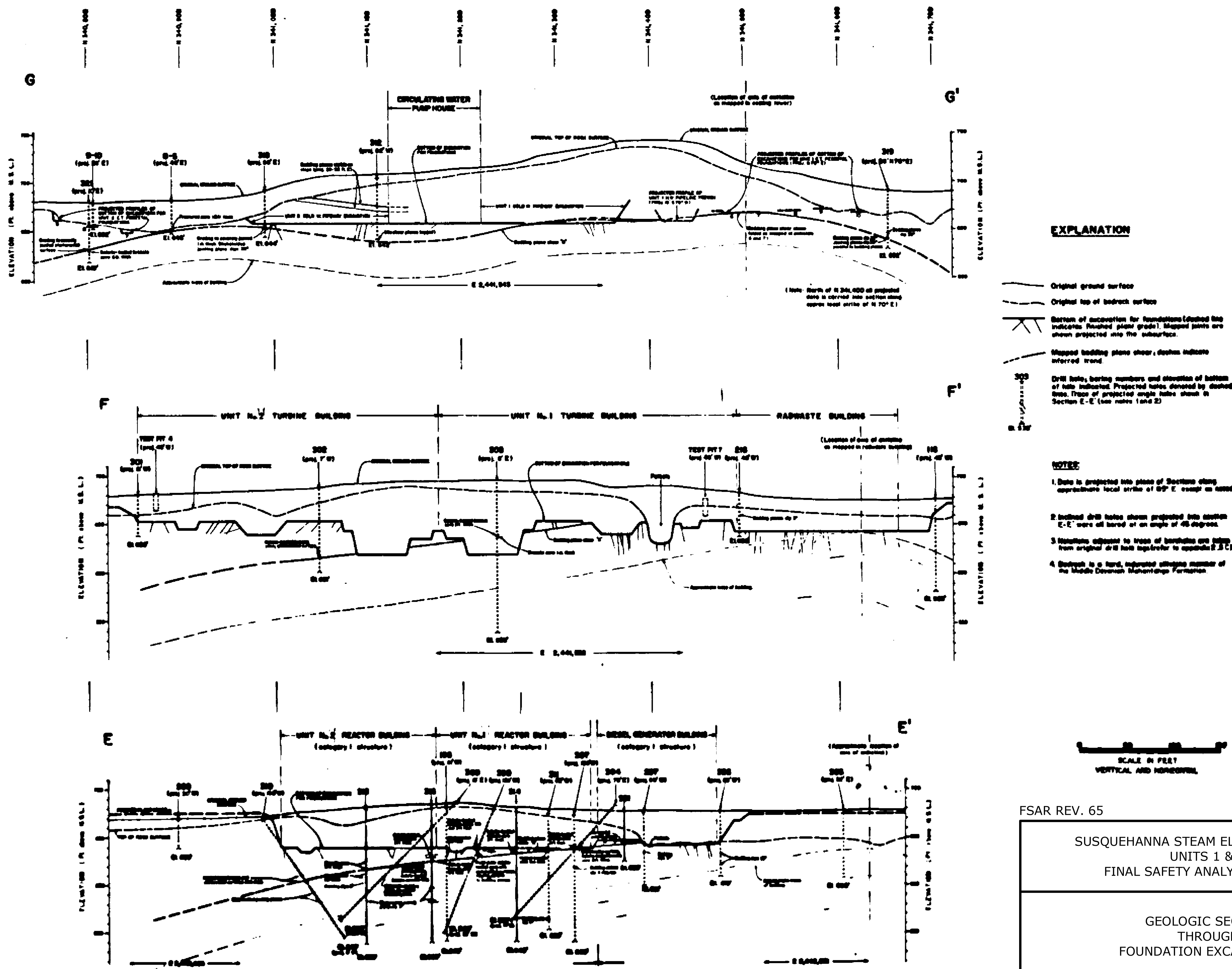
AutoCAD: Figure Fsar 2_5_17.dwg

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
EXTENT OF ROCK AND SOIL FOUNDATIONS FIGURE 2.5-17A

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
GEOLOGIC MAP OF FOUNDATION EXCAVATION FIGURE 2.5-18



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SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

GEOLOGIC SECTION
THROUGH
FOUNDATION EXCAVATIONS

FIGURE 2.5-19, Rev 47

AutoCAD: Figure Fsar 2_5_19.dwg

Mahantango slaty siltstone exposed in west slope of Unit 2 cold water pipe trench adjacent to circulation water pumphouse, at approximate N340,995,E2,441,510. The exposure which is viewed northwest, displays prominent fracture cleavage here dipping 30 degrees southeast. Bedding planes, observable to both above and below the pick head, dip very gently south. The vertical, east-west oriented joint face is a natural surface forming top of rock, eroded by stream and ice scour.

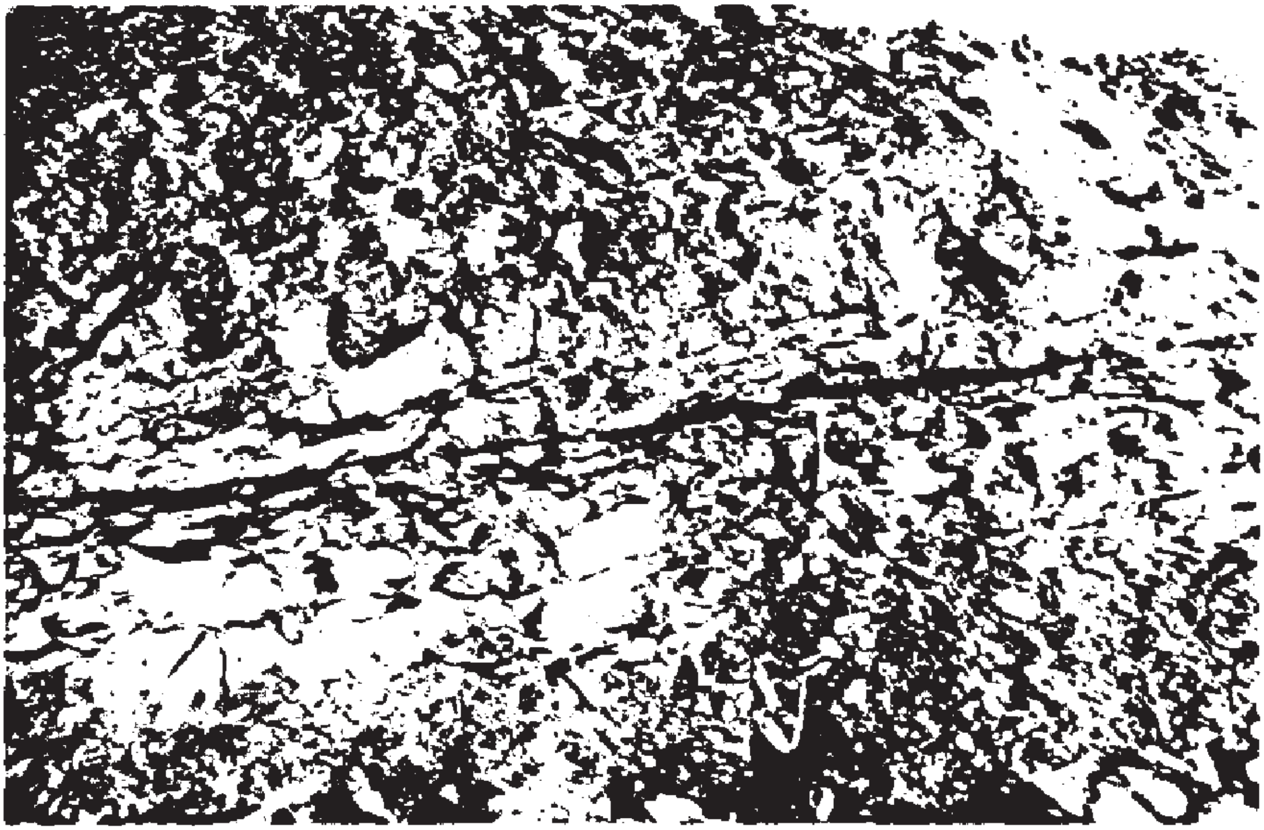
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

PHOTOGRAPH OF GEOLOGIC
FEATURES EXPOSED IN
EXCAVATIONS

FIGURE 2.5-20A, Rev 47

AutoCAD: Figure Fsar 2_5_20A.dwg



Weathered shear plane (bedding plane shear A) exposed in rock slope at N341,360,E2,441,935. View is to the west. The shear plane parallels bedding, which here dips seven degrees south. Weathering has accentuated visibility of the shear, which is here 1/2 to 1 inch wide, containing leached quartz and brown-weathered siltstone fragments. There is no sign of brecciation along the shear plane. Photo taken April 22, 1974.

FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

PHOTOGRAPH OF GEOLOGIC
FEATURES EXPOSED IN
EXCAVATIONS

FIGURE 2.5-20B, Rev 47

AutoCAD: Figure Fsar 2_5_20B.dwg

Looking west at an excavated slope which shows the intersection of the weathered shear plane, bedding plane A, with the eroded rock surface. The intersection of these two surfaces is slightly above and to the left of the top of the hammer, as indicated in the photograph on the right. Scale is 8 inches long.

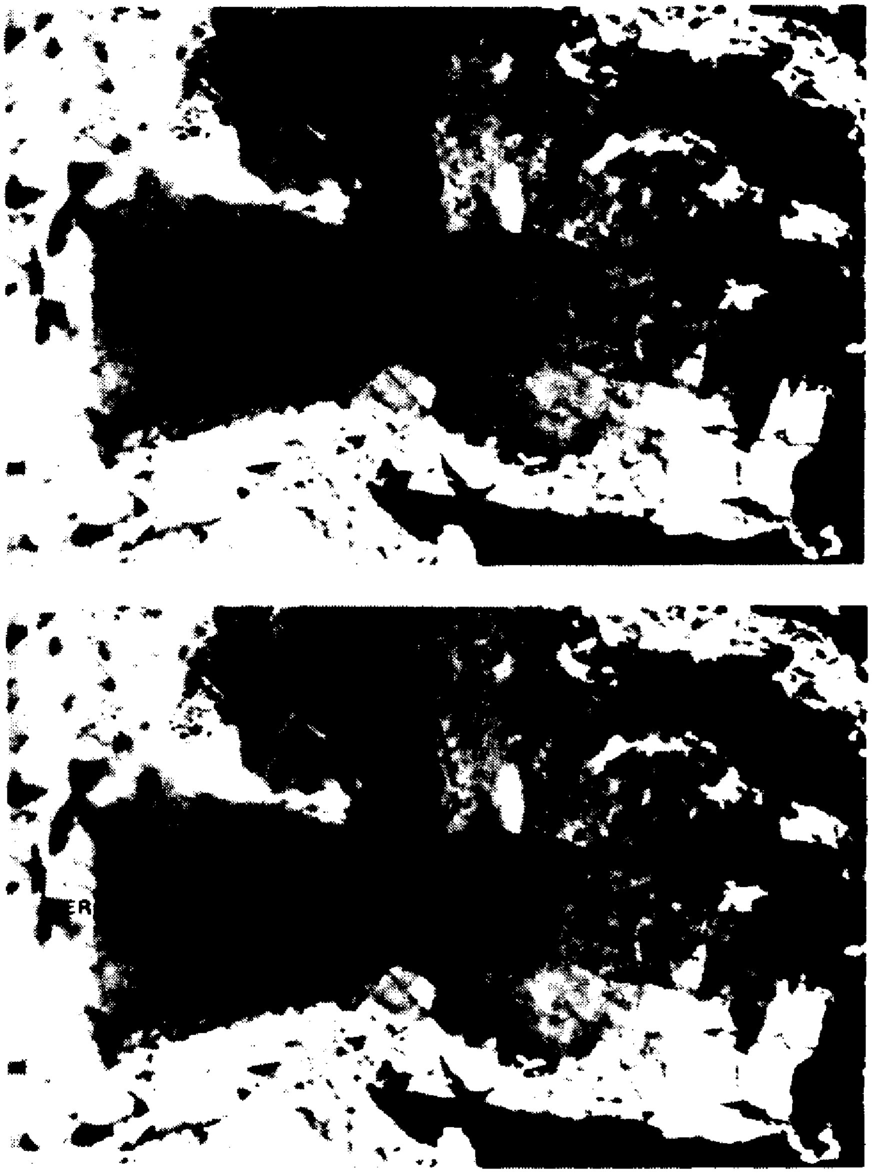
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

PHOTOGRAPH OF GEOLOGIC
FEATURES EXPOSED IN
EXCAVATIONS

FIGURE 2.5-20C, Rev 47

AutoCAD: Figure Fsar 2_5_20C.dwg



Looking east at an excavated slope which shows the intersection of bedding plane A with the erosional rock surface, as outlined in the lower photograph. The erosional rock surface is essentially vertical. Mahantango siltstone at right, bouldery glacial drift on left. Location is about 30 feet from preceding photograph (at the approximate intersection of column line N and N341,375). Photo taken May 15, 1974.

FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

PHOTOGRAPH OF GEOLOGIC
FEATURES EXPOSED IN
EXCAVATIONS

FIGURE 2.5-20D, Rev 47

AutoCAD: Figure Fsar 2_5_20D.dwg

View south of a calcite veinlet crossing a slickensided bedding-plane shear (flat surface near observer) in Unit 1 turbine excavation (intersection of column lines G and 19. Note the calcite veinlet is not displayed above the shear plane. Photo taken May 22, 1974.

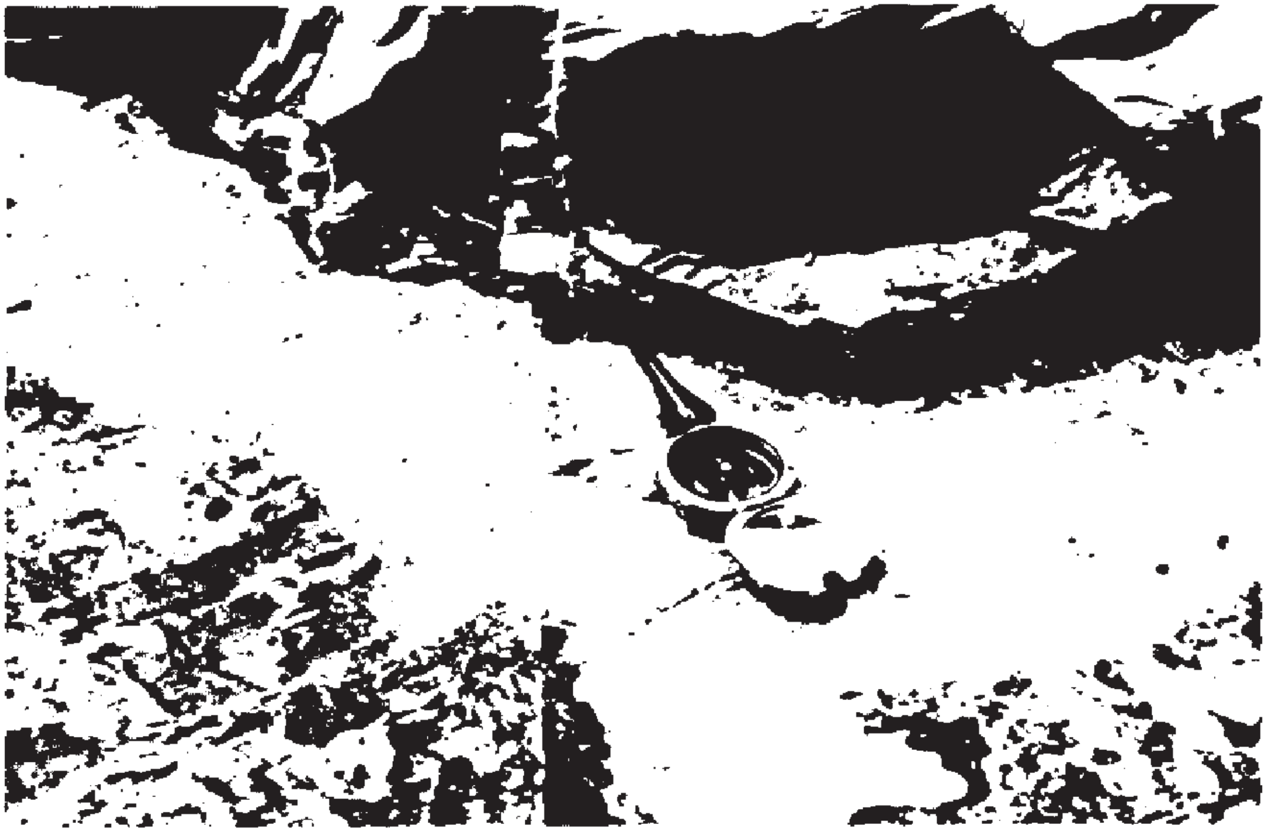
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

PHOTOGRAPH OF GEOLOGIC
FEATURES EXPOSED IN
EXCAVATIONS

FIGURE 2.5-20E, Rev 47

AutoCAD: Figure Fsar 2_5_20E.dwg



A closer view of the same exposure described in the preceding photograph. Compass is oriented parallel to slickens ide lineation which trends S3°E. If movement represented by the slickens ide had occurred subsequent to formation of the calcite veinlet, the vein would have been offset.
Photo taken May 22, 1974.

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SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

PHOTOGRAPH OF GEOLOGIC
FEATURES EXPOSED IN
EXCAVATIONS

FIGURE 2.5-20F, Rev 47

AutoCAD: Figure Fsar 2_5_20F.dwg

Detail of excavation in condensate pump pit, Unit 2 turbine area. This photograph shows typical foundation excavation in the hard, unweathered Mahantango siltstone at the site. Here the slopes were presplit along closely spaced, line-drilled blast holes. View it to south. Photo taken July 3, 1974.

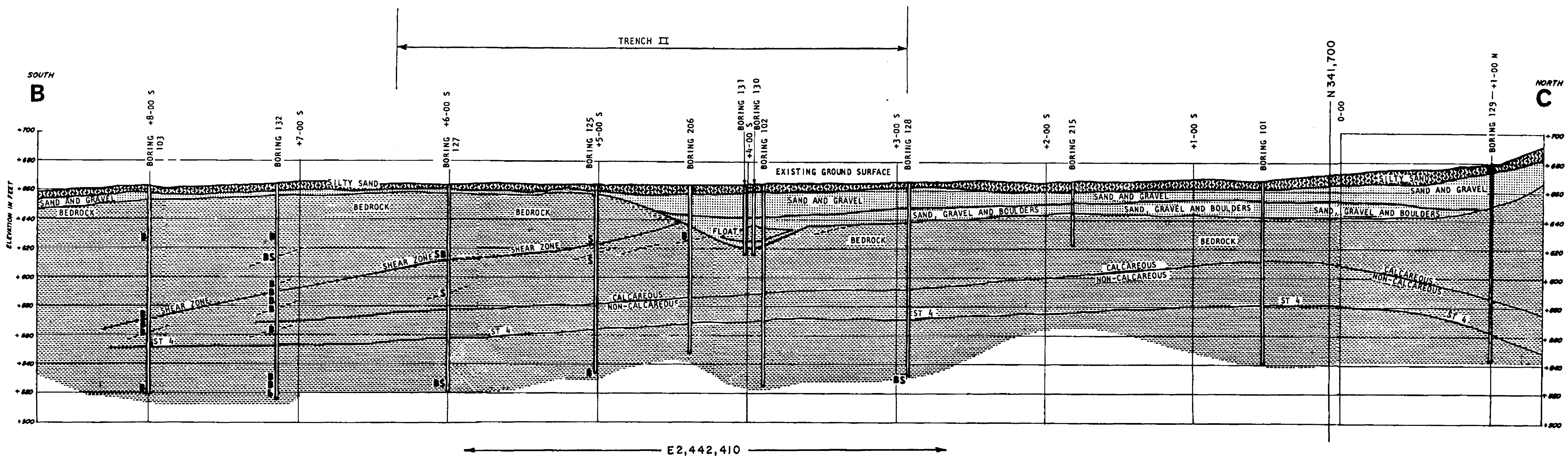
FSAR REV. 65

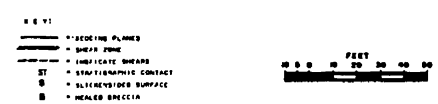
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

PHOTOGRAPH OF GEOLOGIC
FEATURES EXPOSED IN
EXCAVATIONS

FIGURE 2.5-20G, Rev 47

AutoCAD: Figure Fsar 2_5_20G.dwg





NOTE:

This section prepared by Dames & Moore and duplicated for this report.

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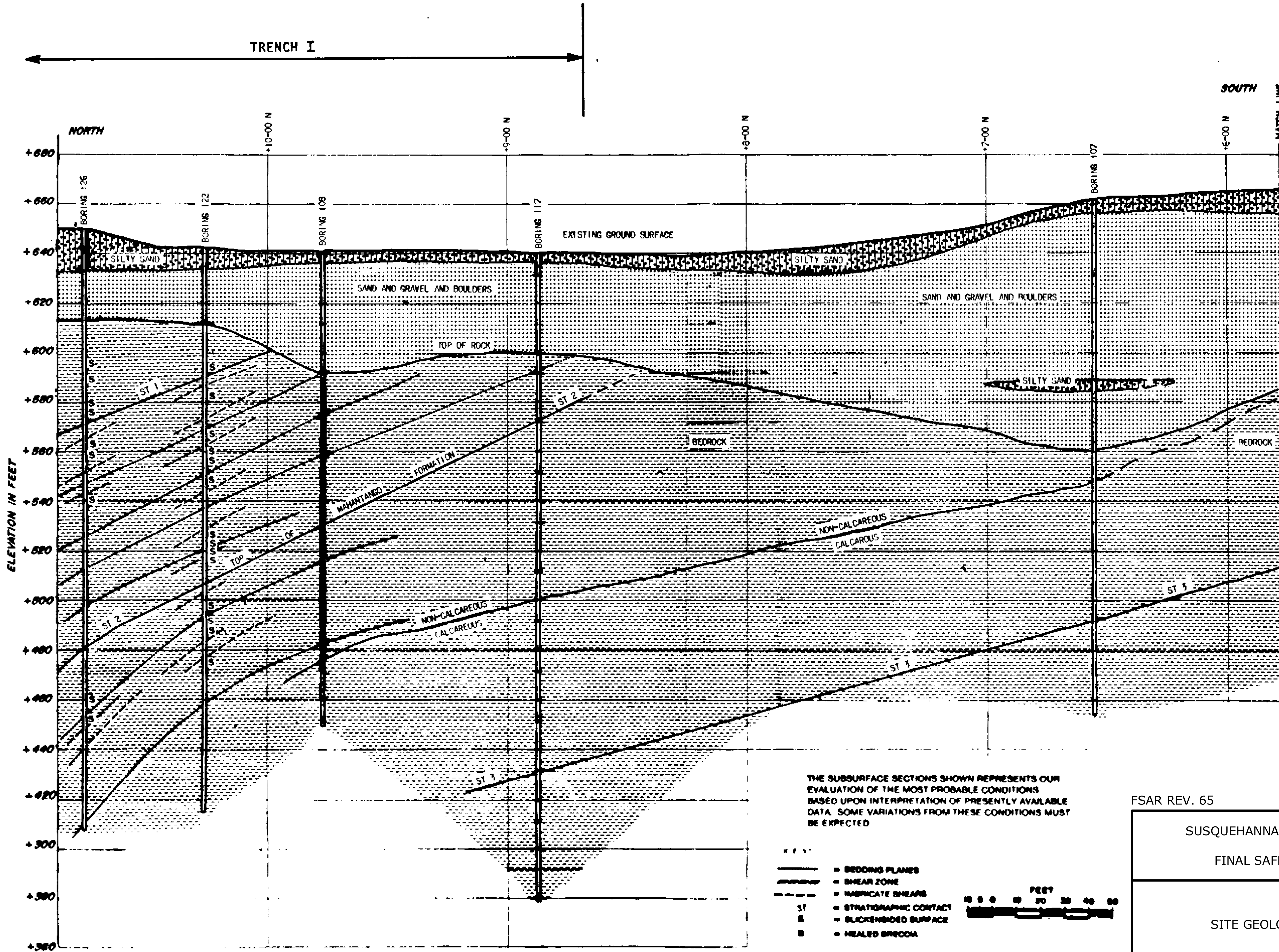
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SITE GEOLOGIC CROSS SECTIONS

FIGURE 2.5-21A, Rev 47

AutoCAD: Figure Fsar 2_5_21A.dwg



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SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SITE GEOLOGIC CROSS SECTIONS

FIGURE 2.5-21B, Rev 47

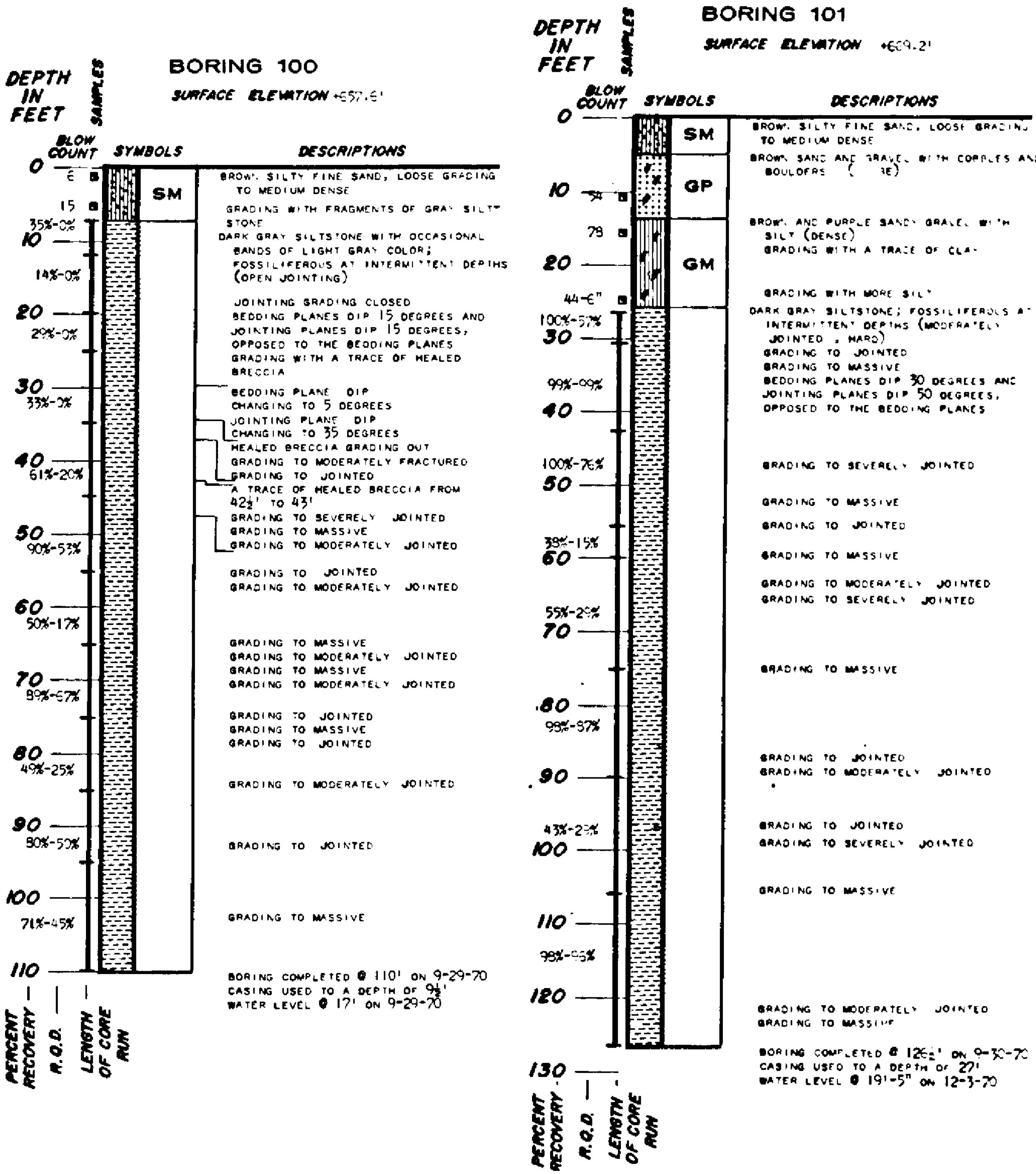
AutoCAD: Figure Fsar 2_5_21B.dwg

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
PLOT PLAN
FIGURE 2.5-22

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
PLOT PLAN DETAILS FIGURE 2.5-22A



NOTE: 1 THE FIGURES IN THE COLUMN LABELED "BLOW COUNT" REFER TO THE NUMBER OF BLOWS REQUIRED TO DRIVE THE DAMES & MOORE SOIL SAMPLER A DISTANCE OF ONE FOOT USING A 300-POUND HAMMER FALLING 18 INCHES, OR A STANDARD SPLIT-SPOON SAMPLER A DISTANCE OF ONE FOOT, USING A 140-POUND DRIVE WEIGHT FALLING 30 INCHES. THE DAMES & MOORE SAMPLER IS 3¼" O.D. AND APPROXIMATELY 2½" I.D. THE STANDARD SPLIT-SPOON SAMPLER IS 2" O.D. AND 1-3/8" I.D.

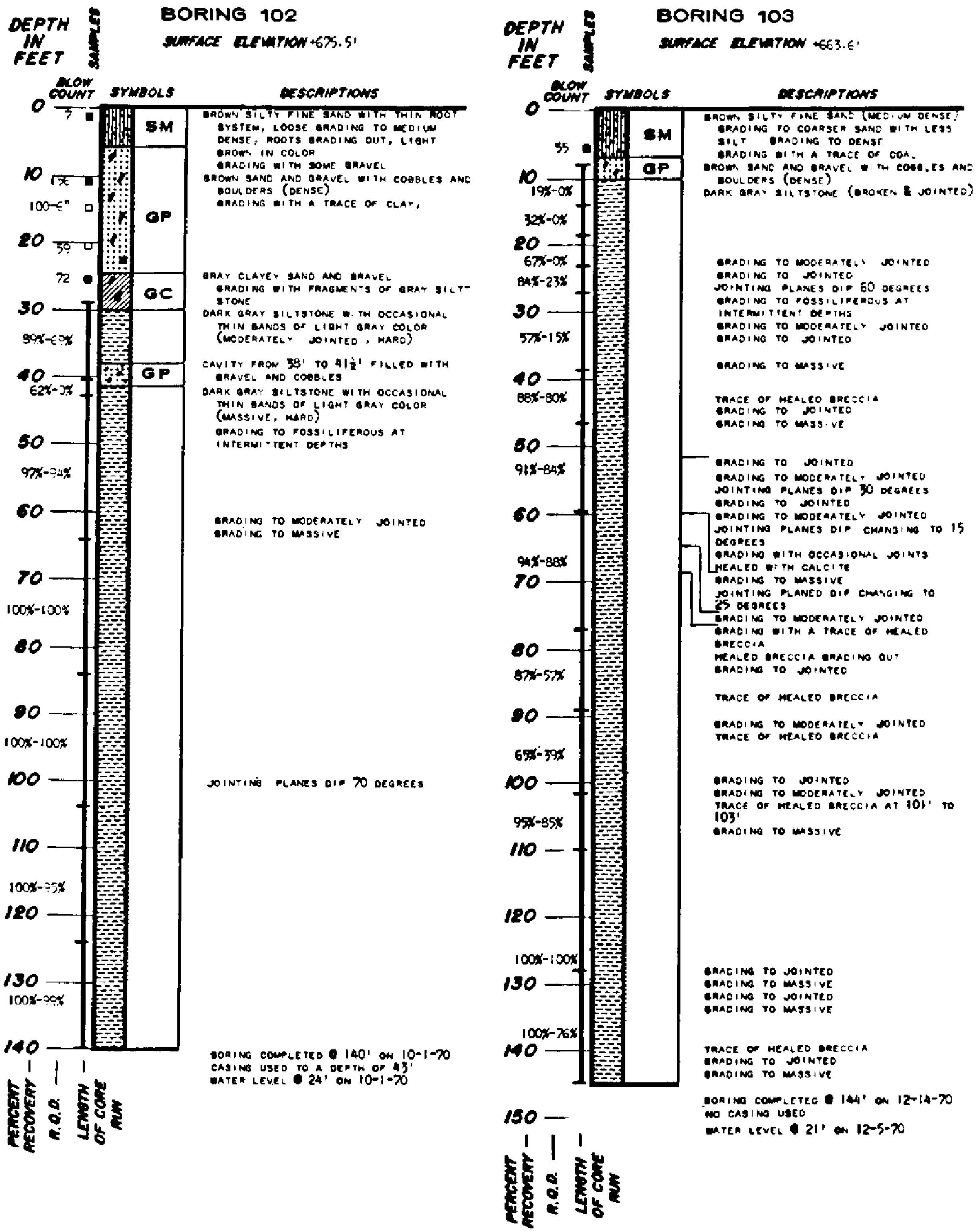
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23A, Rev 47

AutoCAD: Figure Fsar 2_5_23A.dwg



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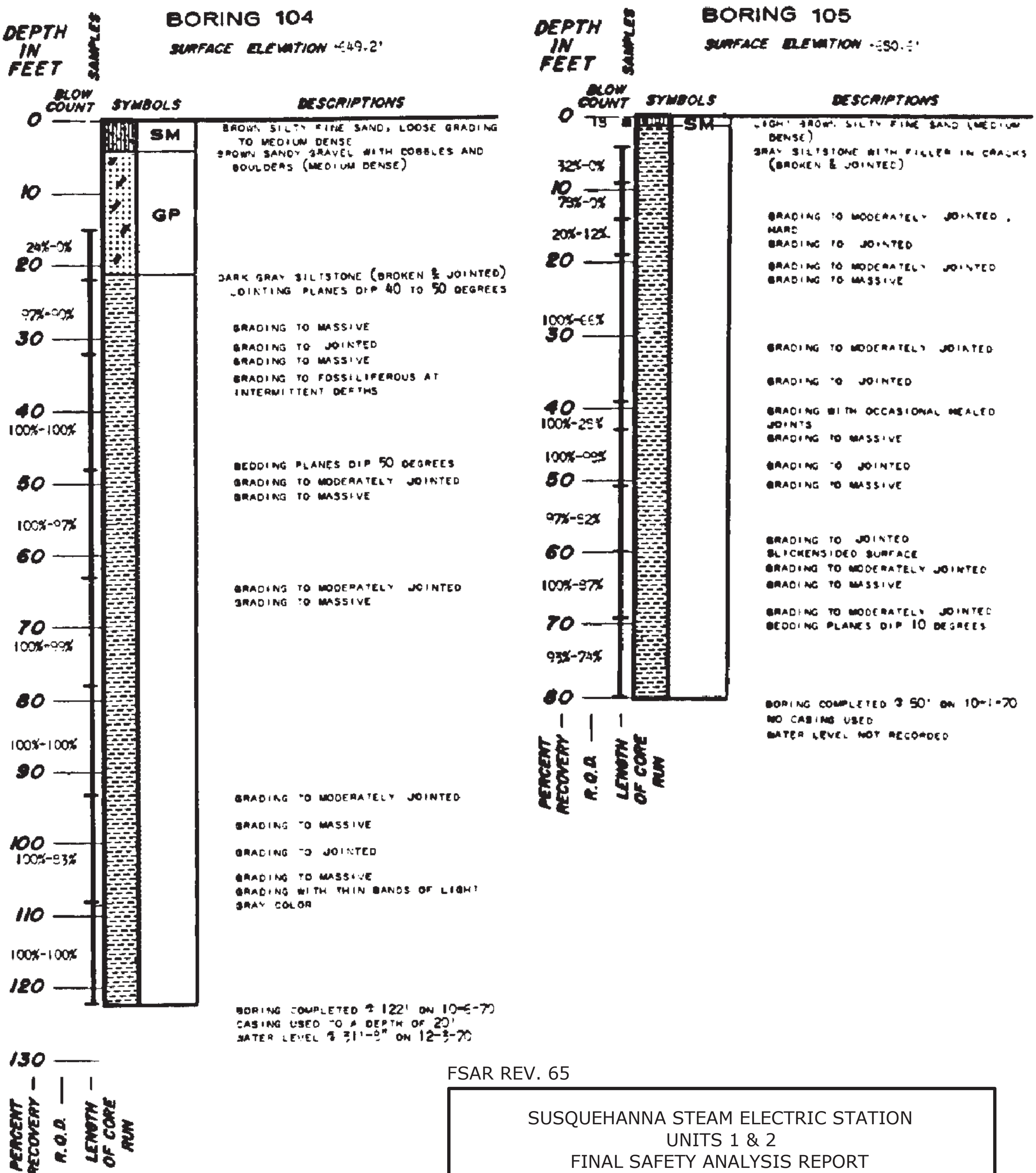
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23B, Rev 47

AutoCAD: Figure Fsar 2_5_23B.dwg



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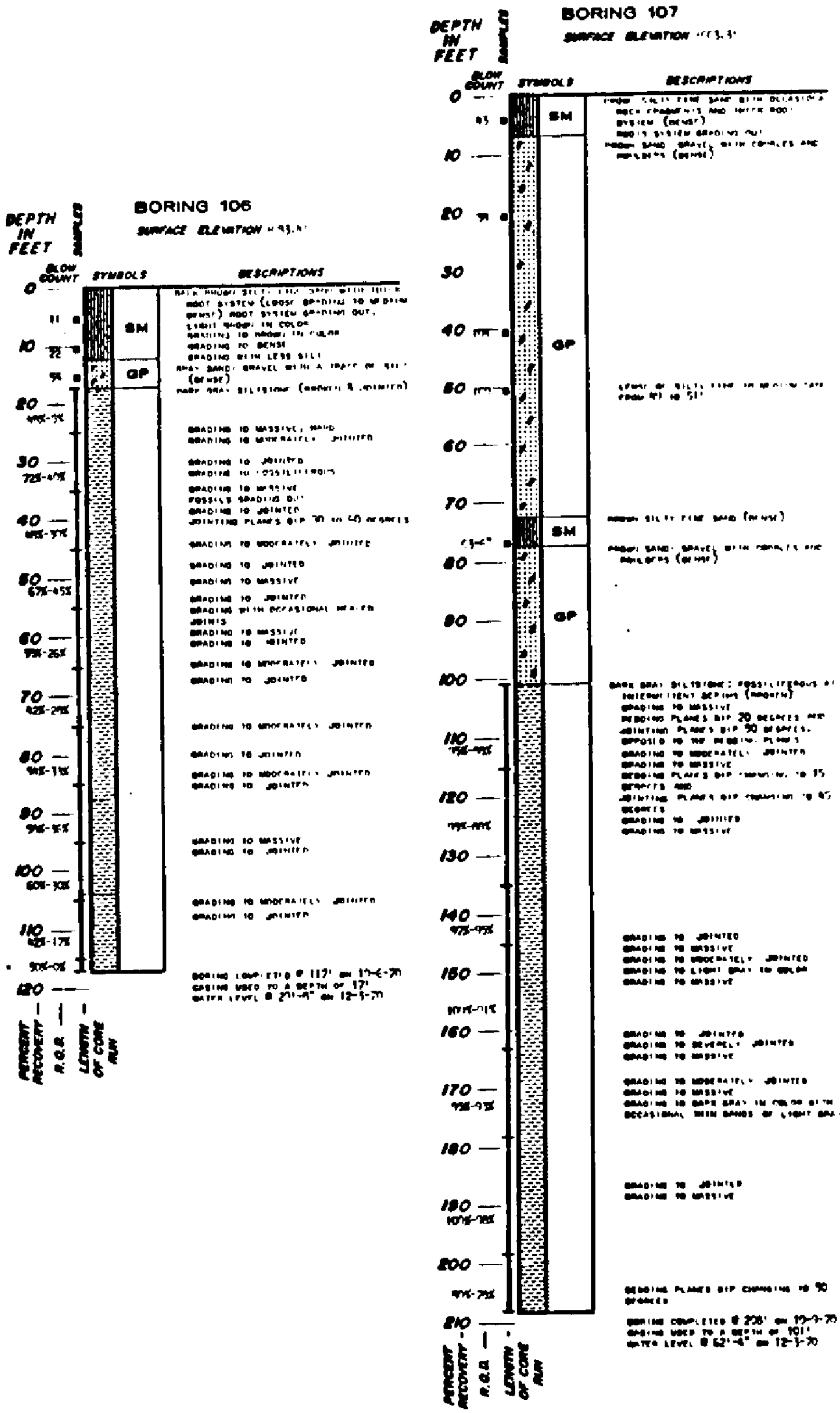
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23C, Rev 47

AutoCAD: Figure Fsar 2_5_23C.dwg



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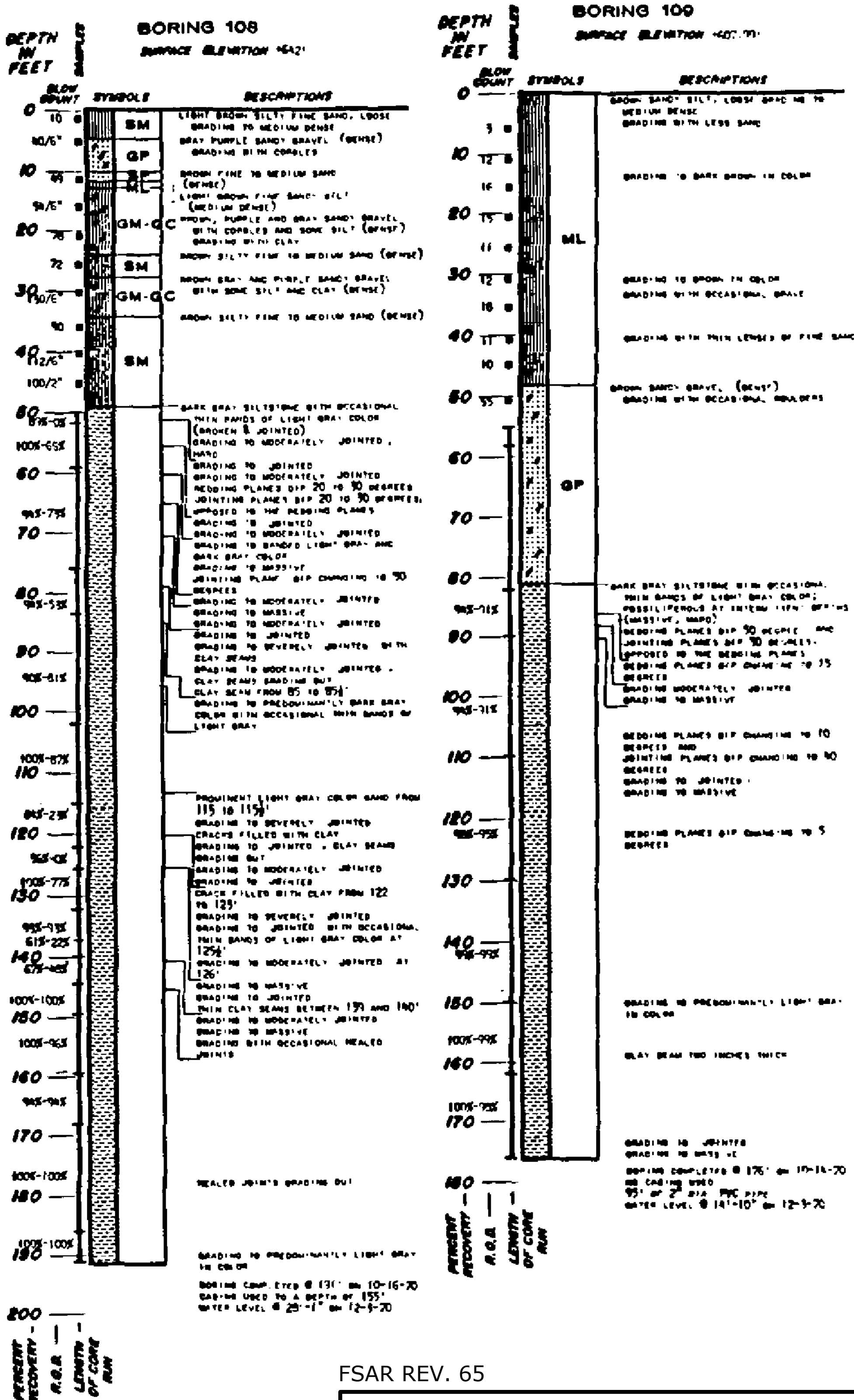
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23D, Rev 47

AutoCAD: Figure Fsar 2_5_23D.dwg



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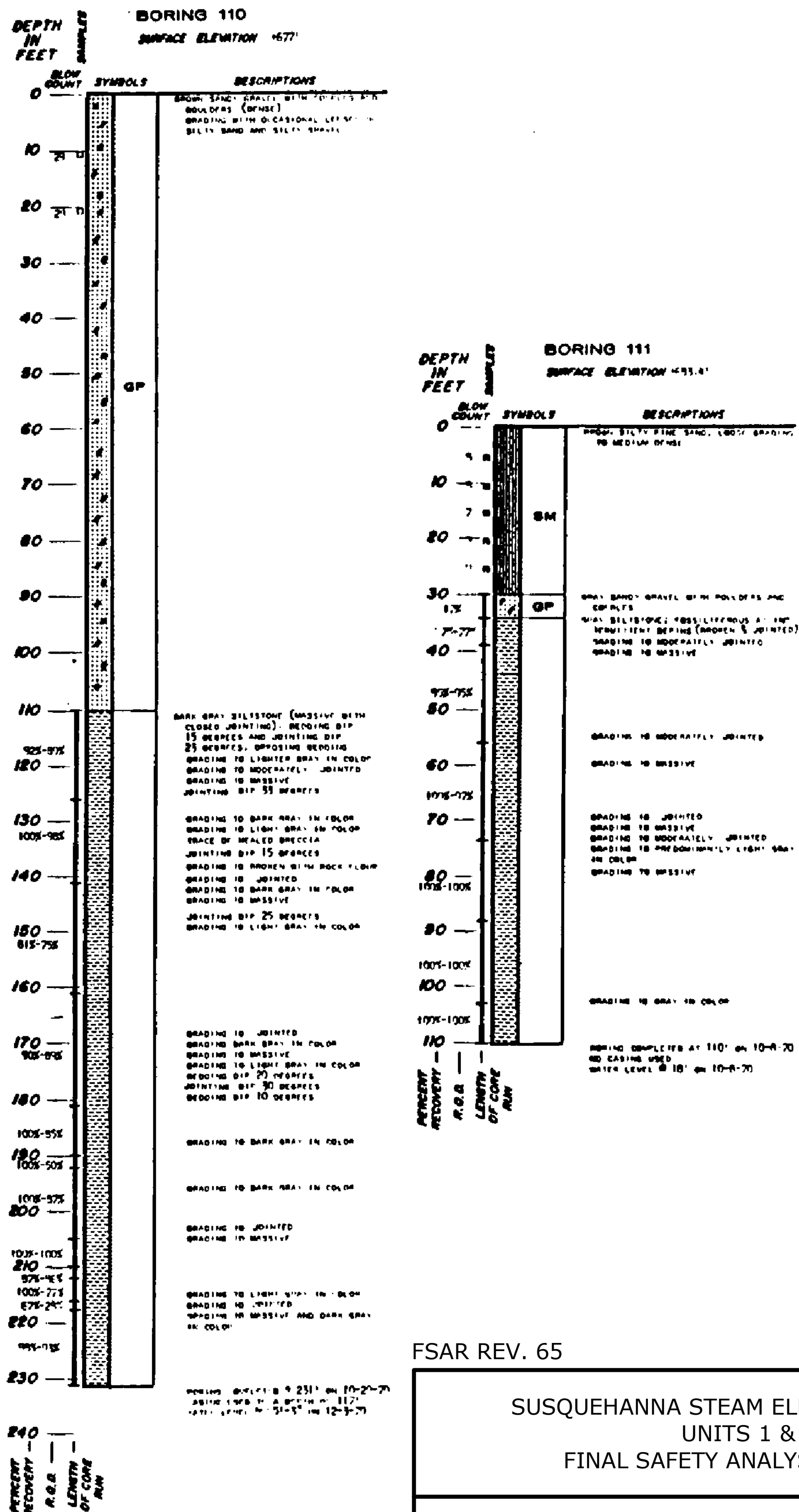
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23E, Rev 47

AutoCAD: Figure Fsar 2_5_23E.dwg



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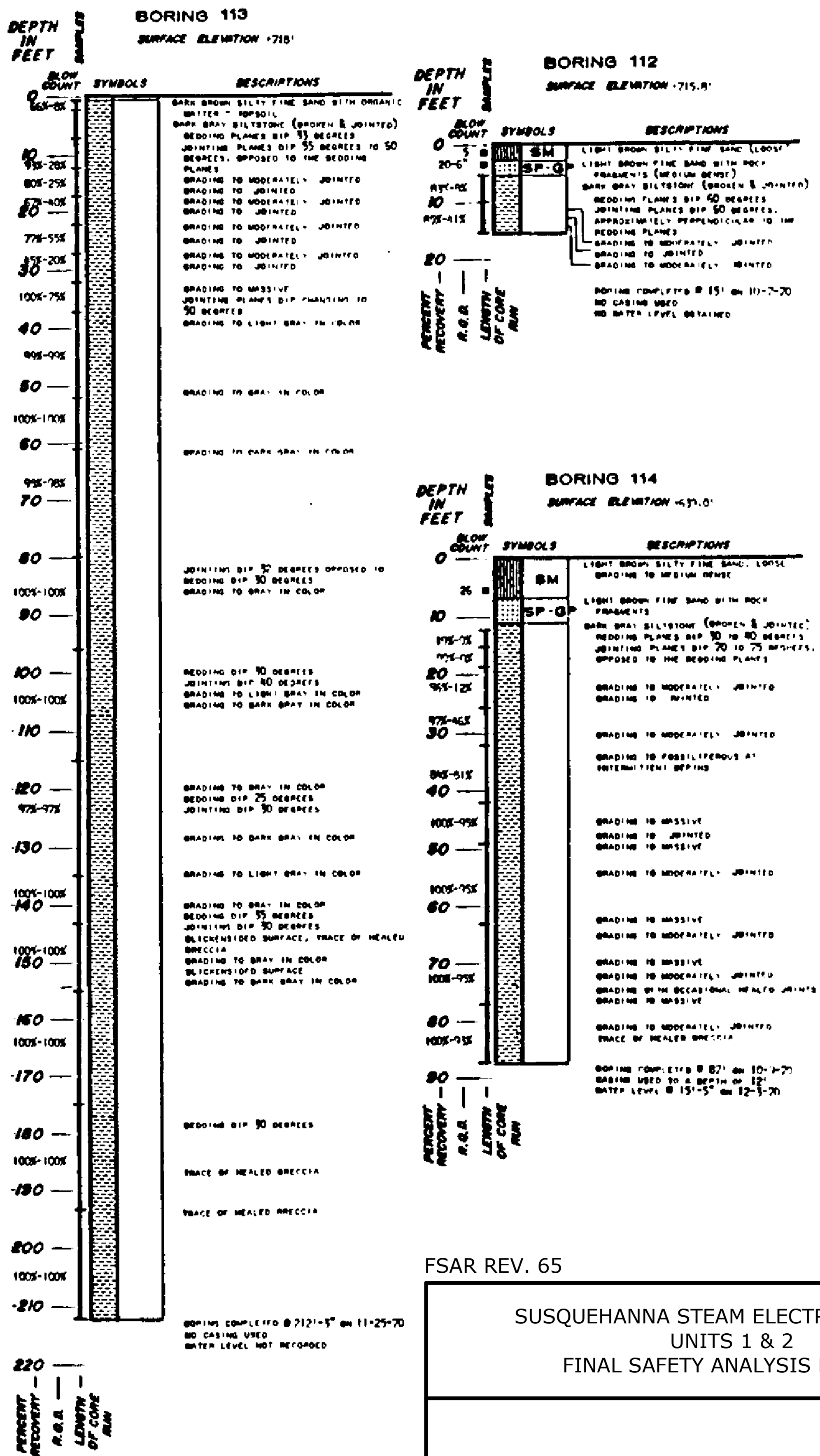
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23F, Rev 47

AutoCAD: Figure Fsar 2_5_23F.dwg



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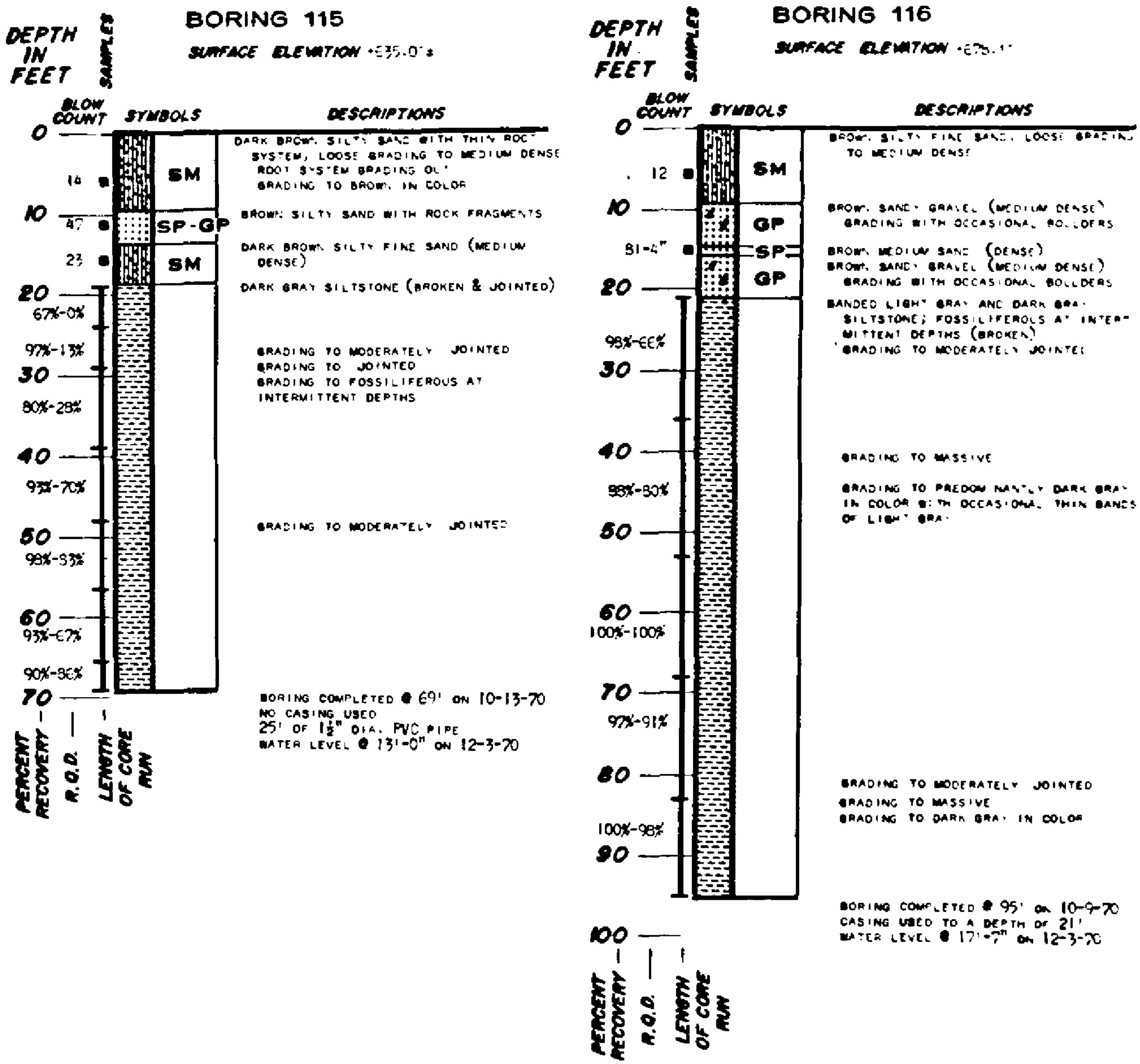
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23G, Rev 47

AutoCAD: Figure Fsar 2_5_23G.dwg



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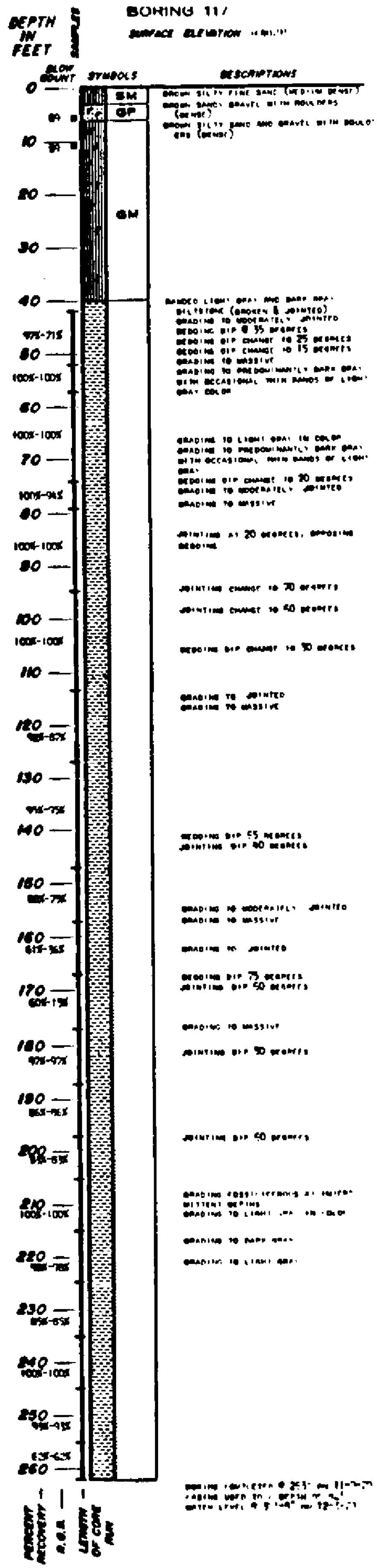
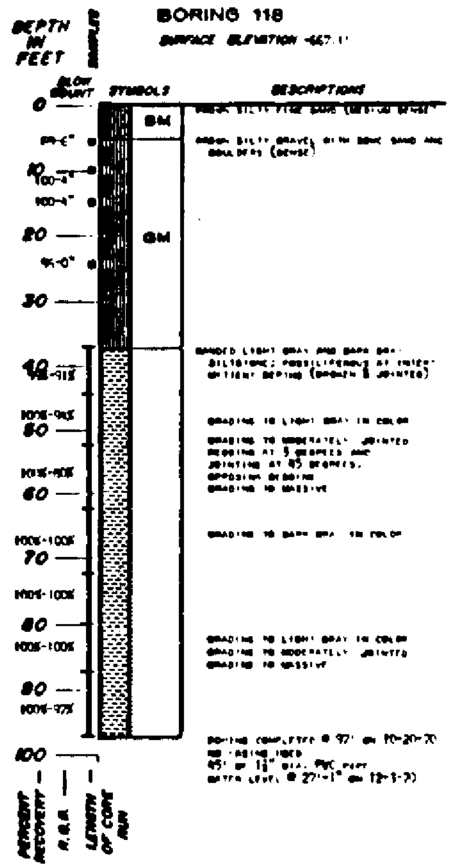
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23H, Rev 47

AutoCAD: Figure Fsar 2_5_23H.dwg





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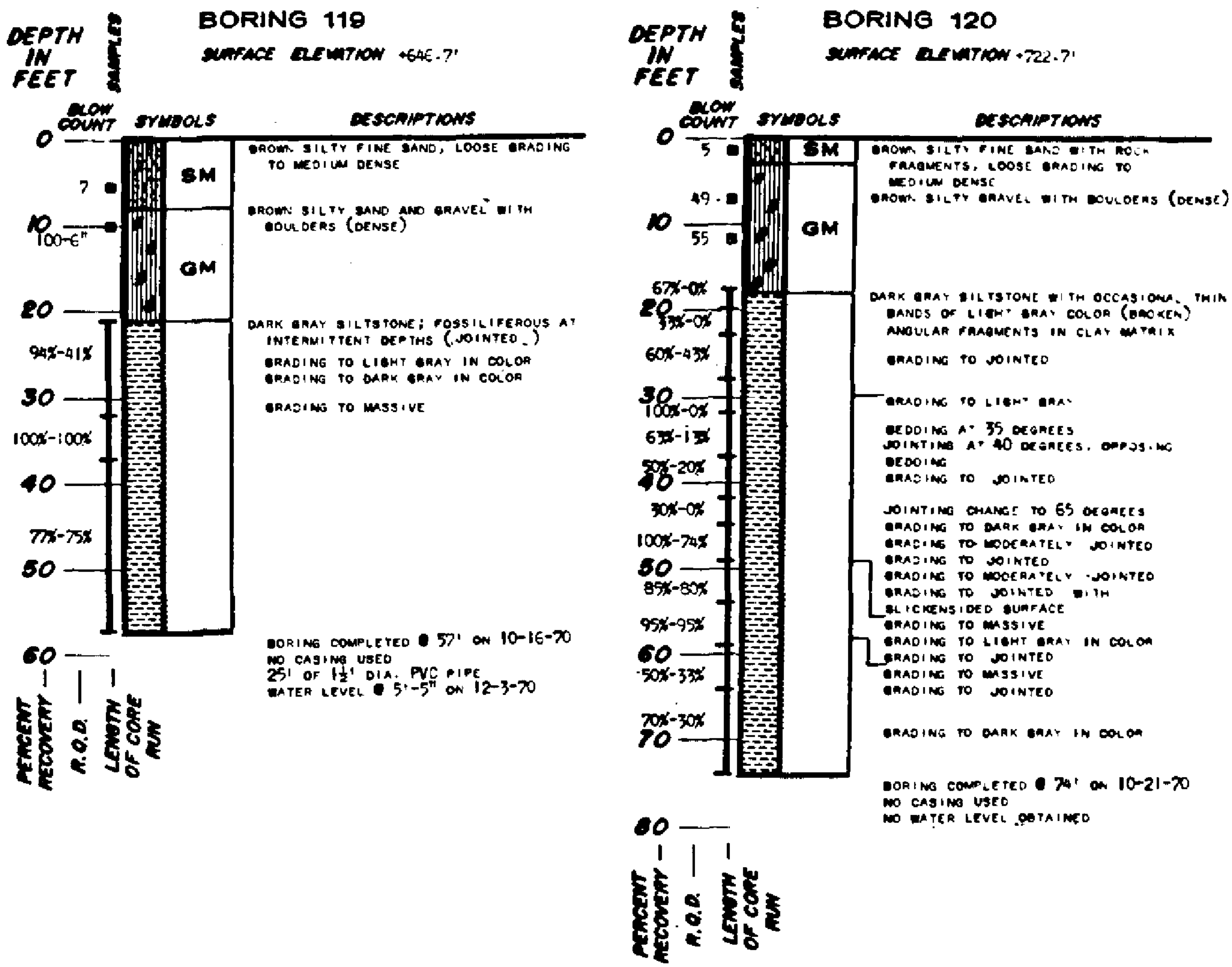
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23I, Rev 47

AutoCAD: Figure Fsar 2_5_23I.dwg



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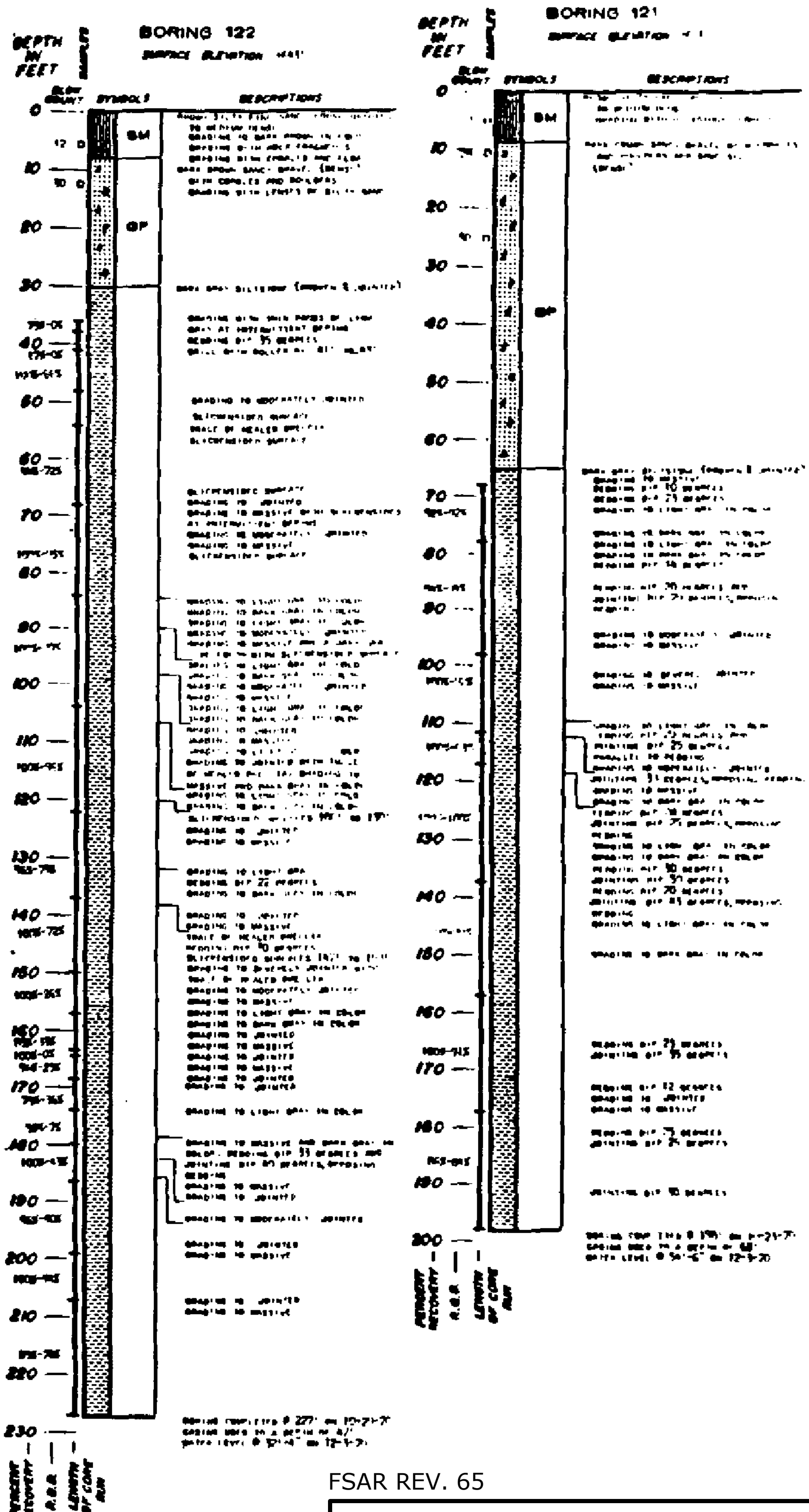
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23J, Rev 47

AutoCAD: Figure Fsar 2_5_23J.dwg



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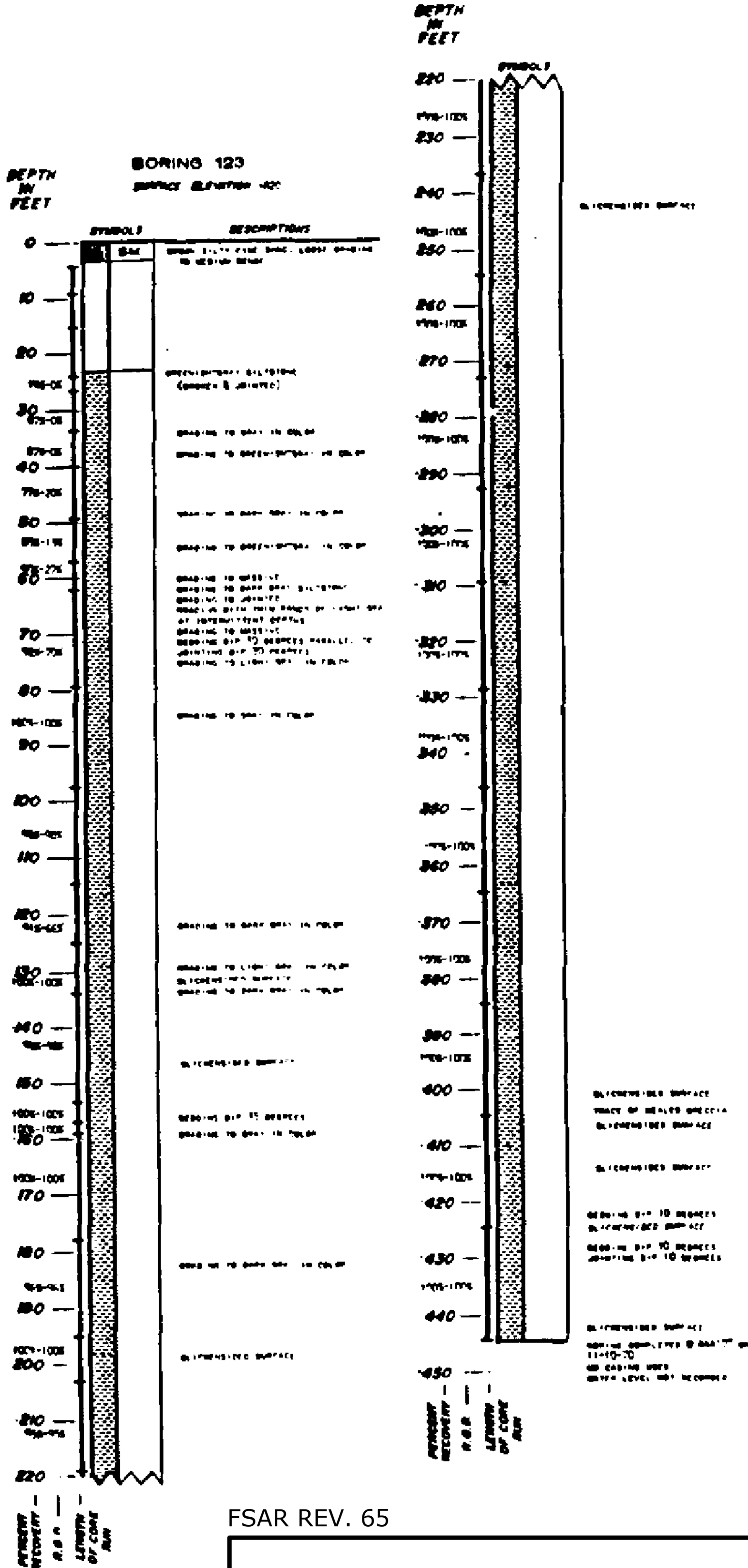
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23K, Rev 47

AutoCAD: Figure Fsar 2_5_23K.dwg



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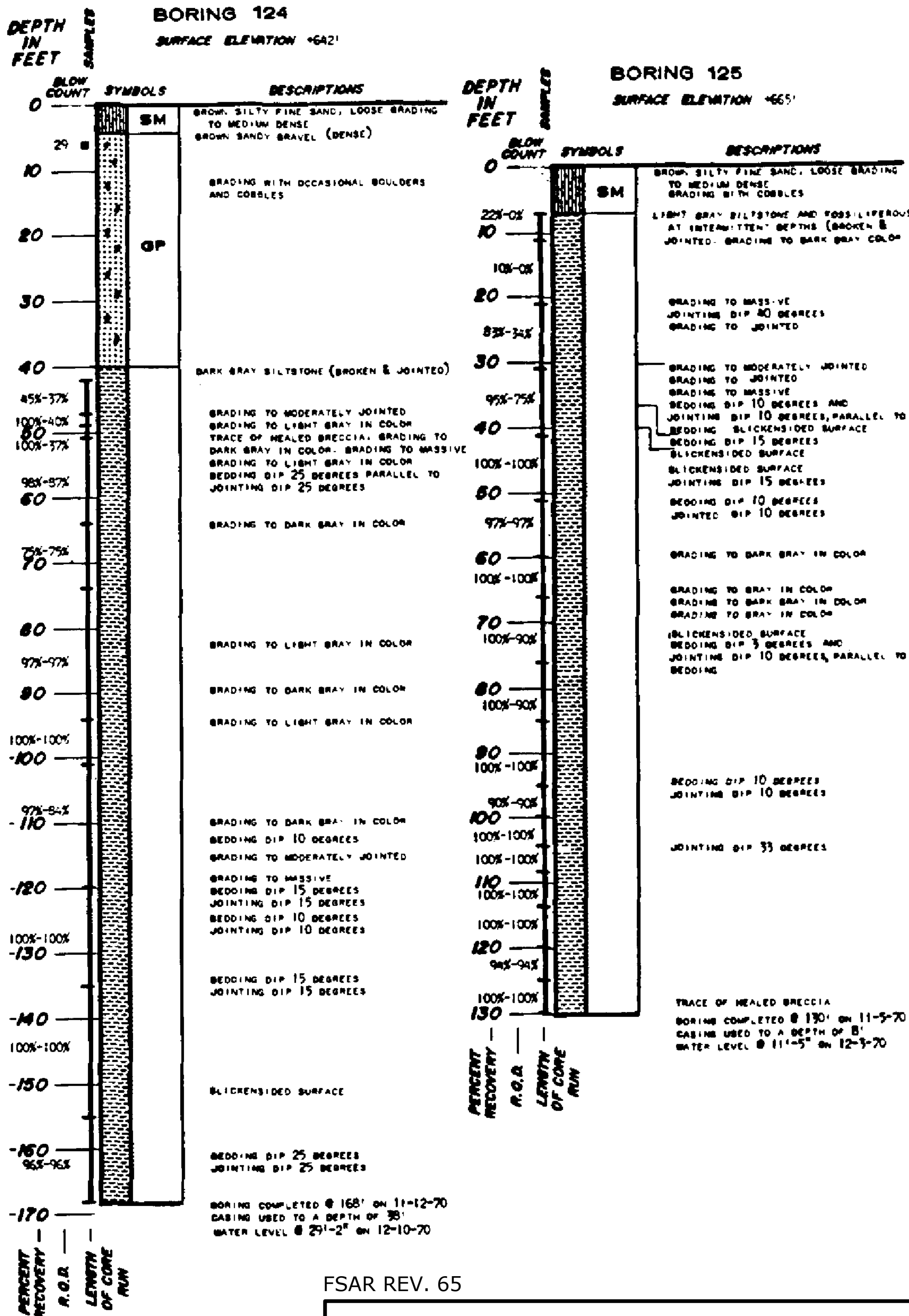
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23L, Rev 47

AutoCAD: Figure Fsar 2_5_23L.dwg



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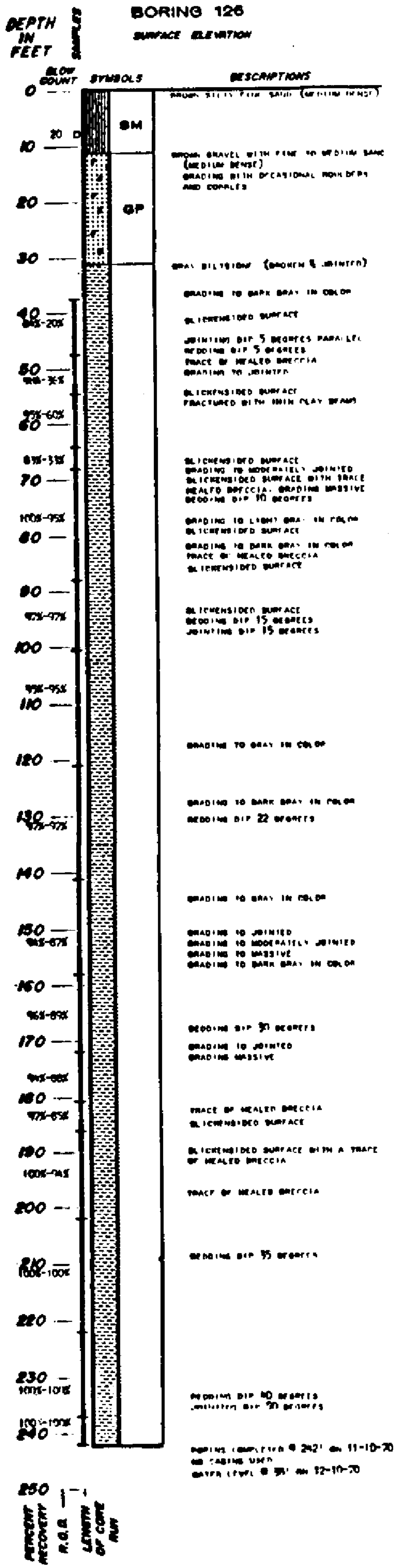
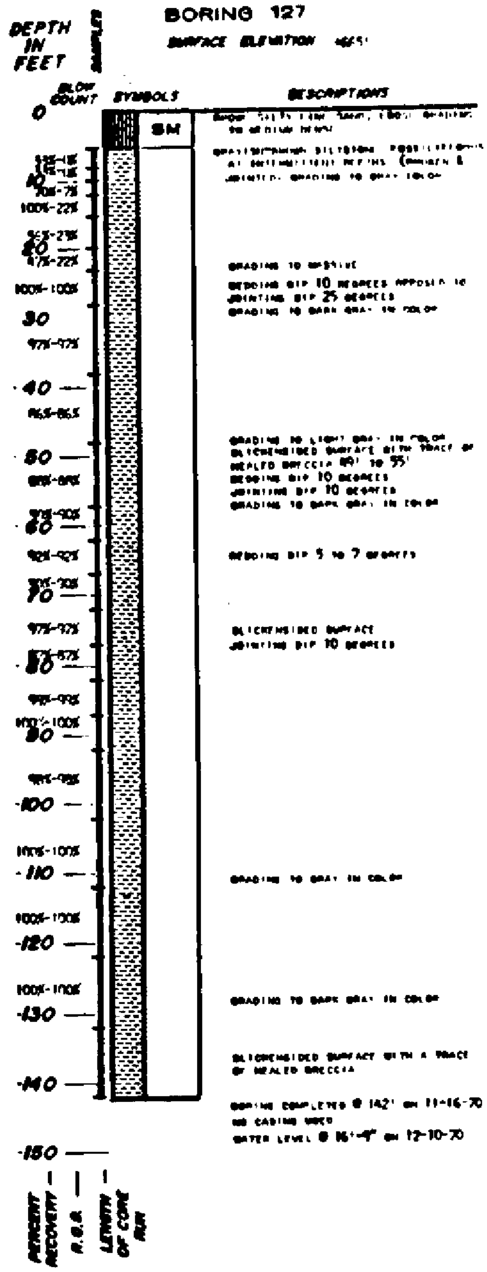
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23M, Rev 47

AutoCAD: Figure Fsar 2_5_23M.dwg





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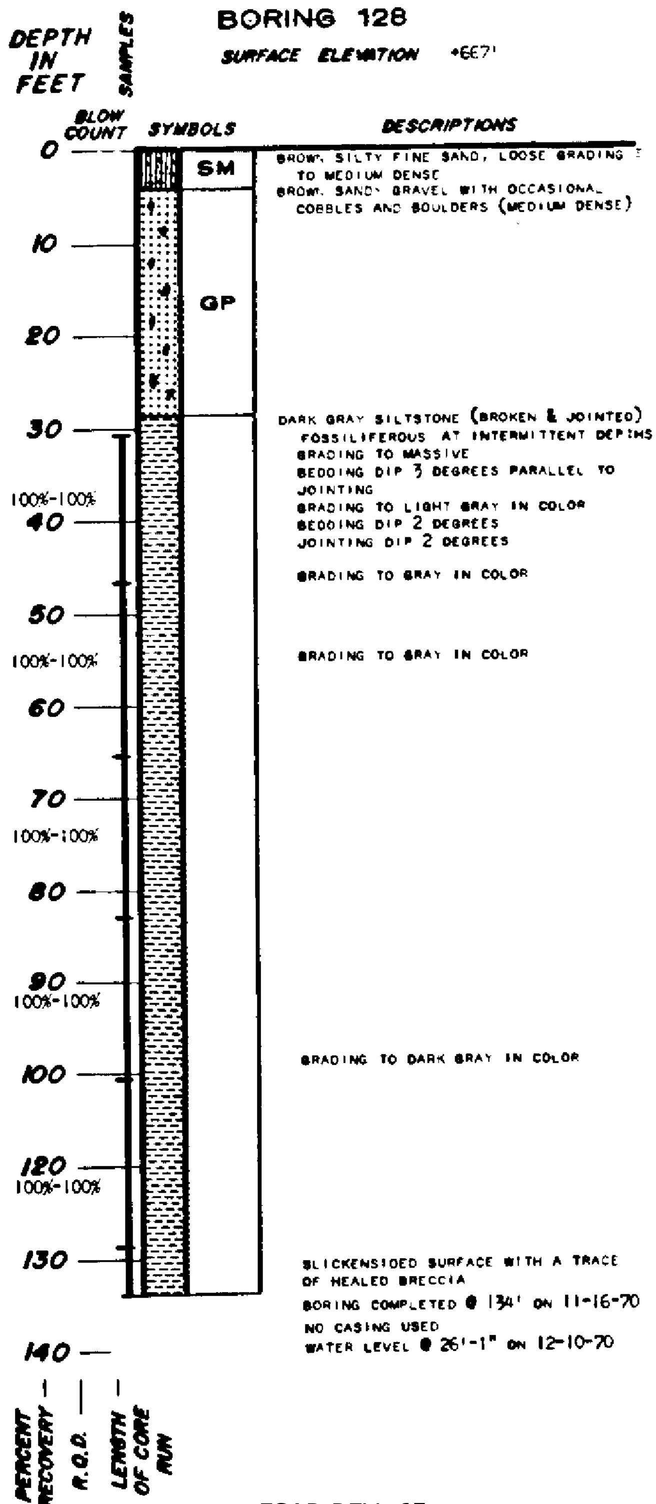
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23N, Rev 47

AutoCAD: Figure Fsar 2_5_23N.dwg



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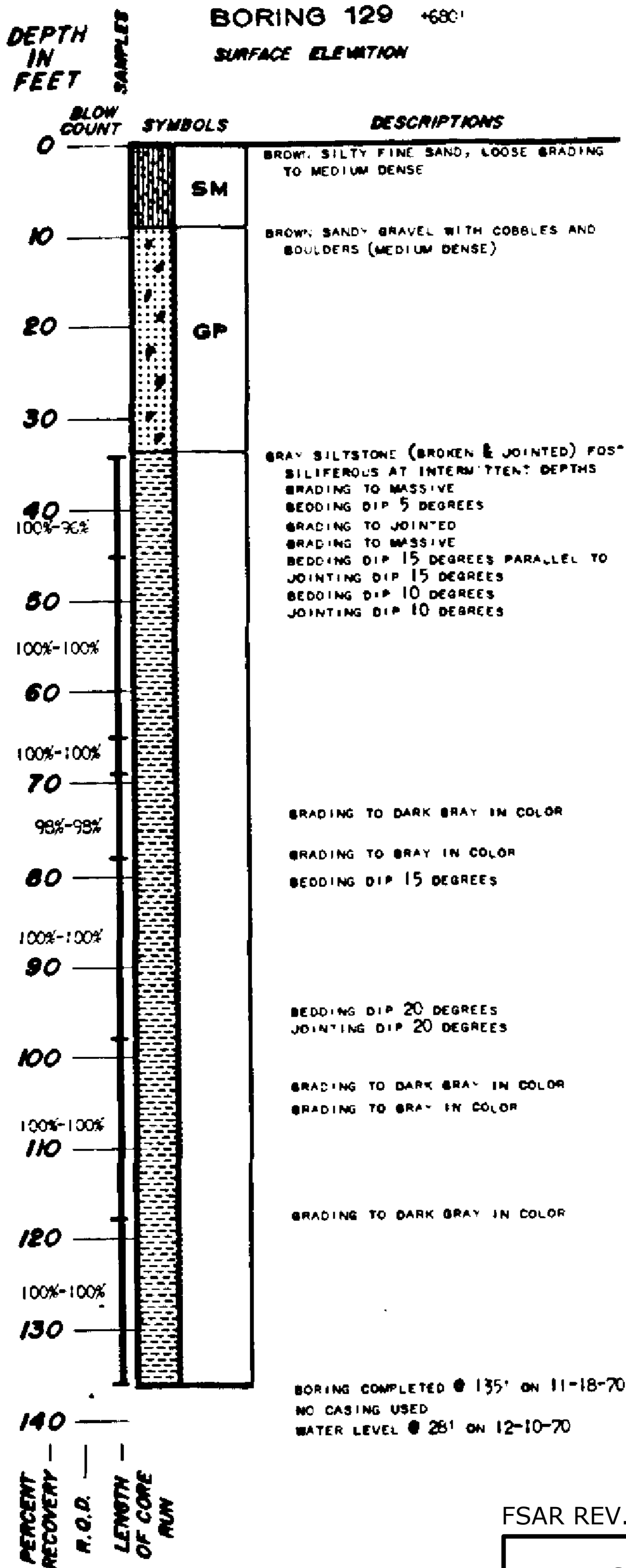
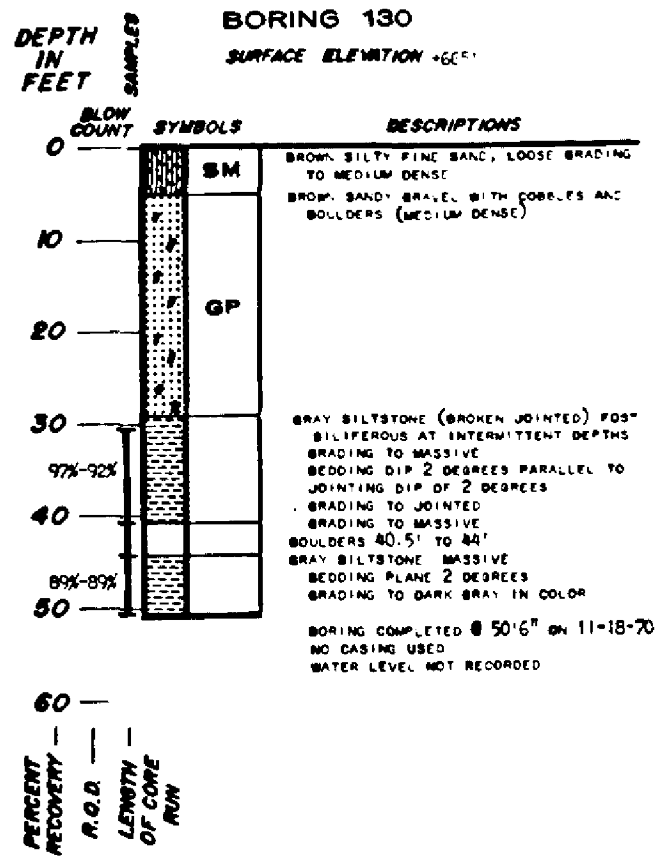
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23O, Rev 47

AutoCAD: Figure Fsar 2_5_23O.dwg





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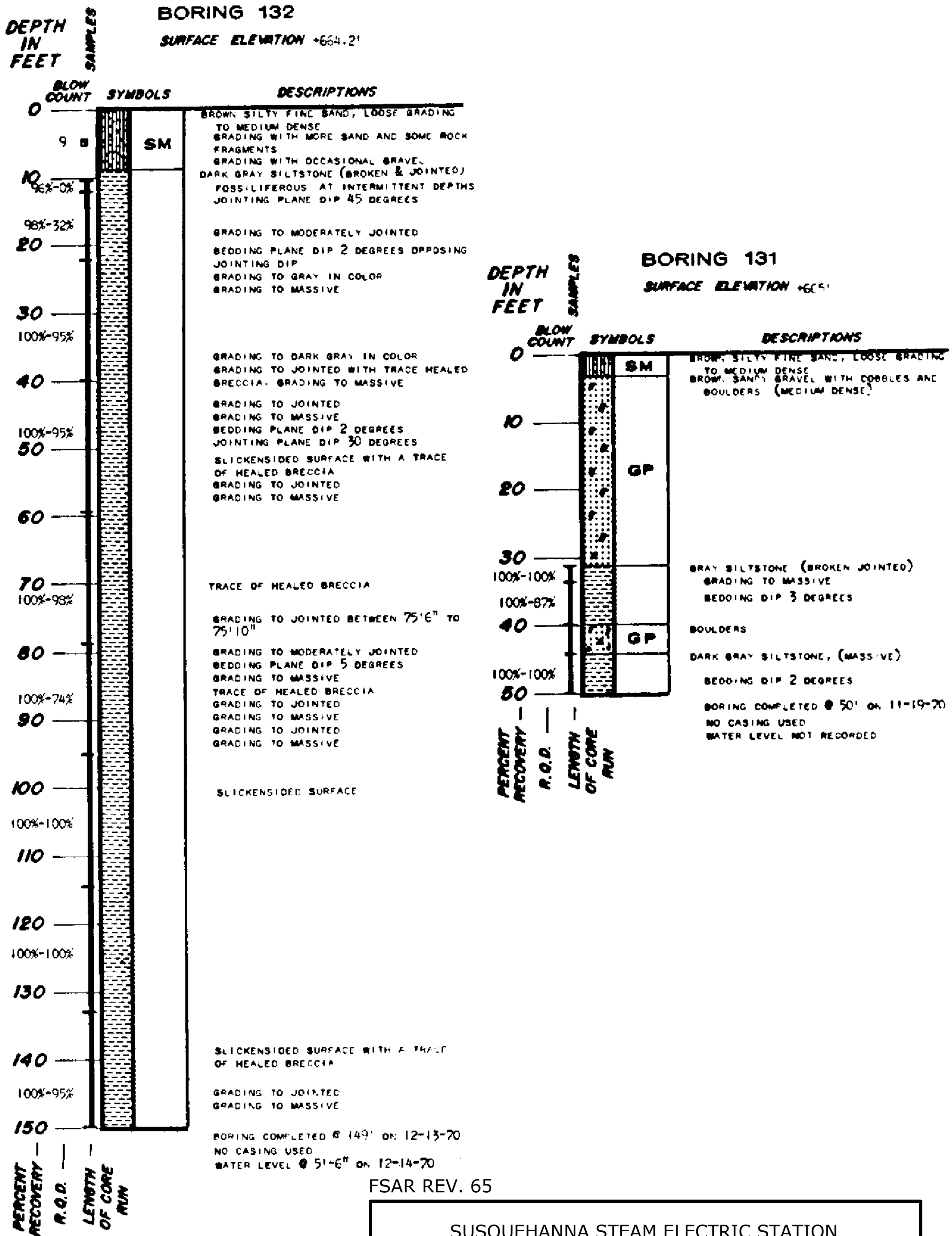
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23P, Rev 47

AutoCAD: Figure Fsar 2_5_23P.dwg



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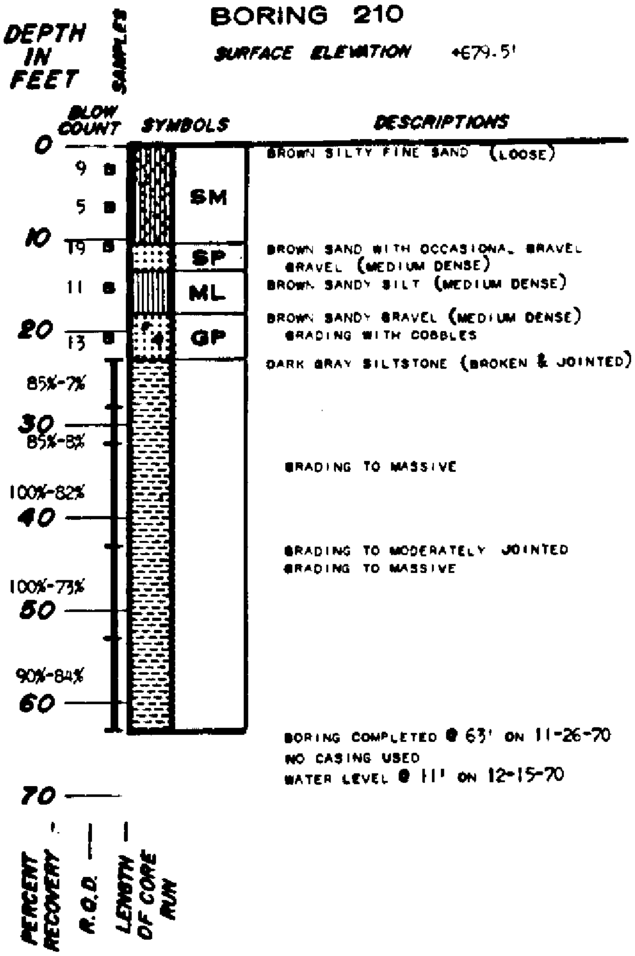
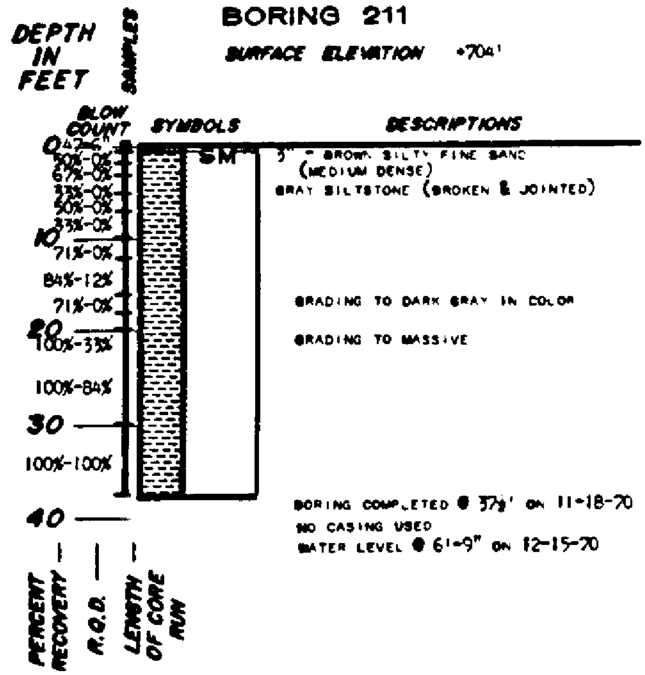
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

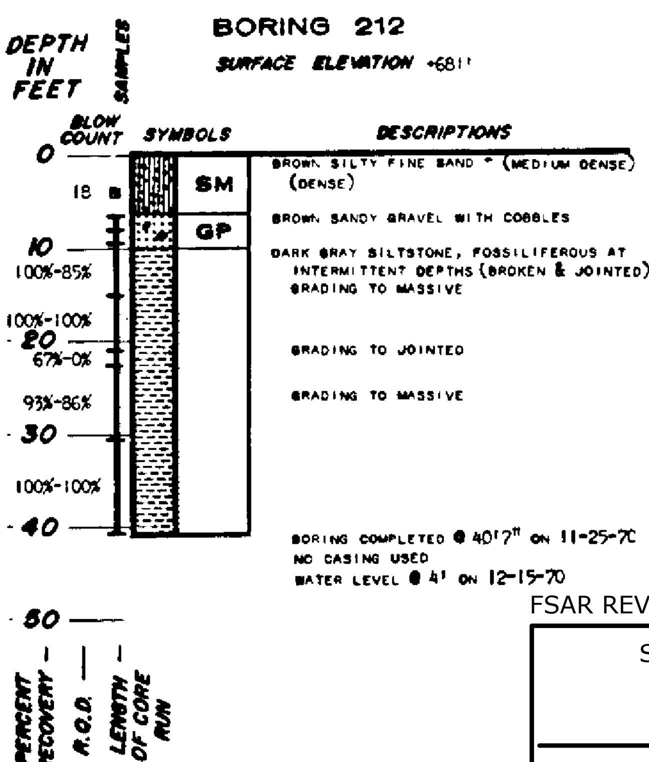
LOG OF BORINGS

FIGURE 2.5-23Q, Rev 47

AutoCAD: Figure Fsar 2_5_23Q.dwg







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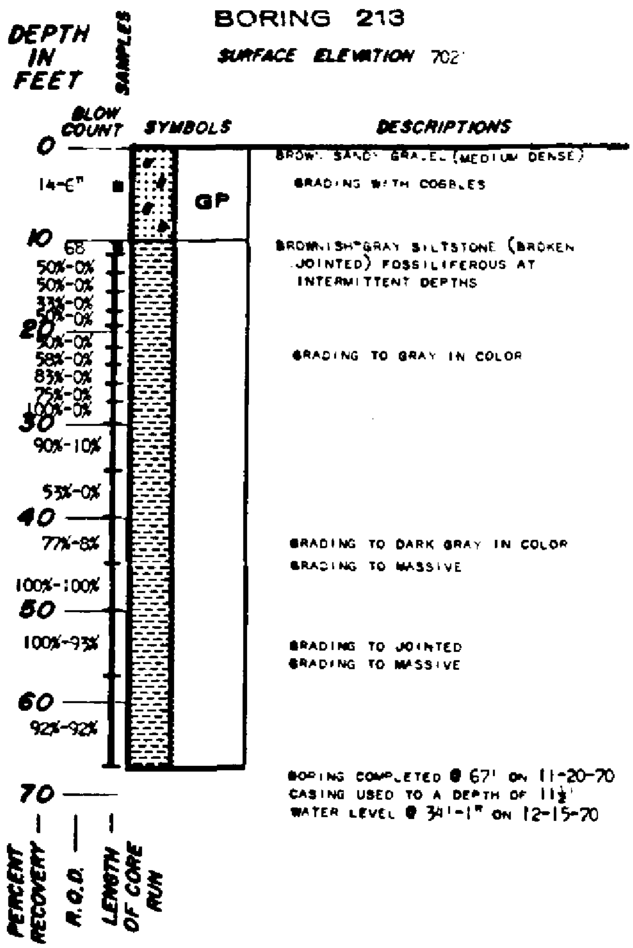
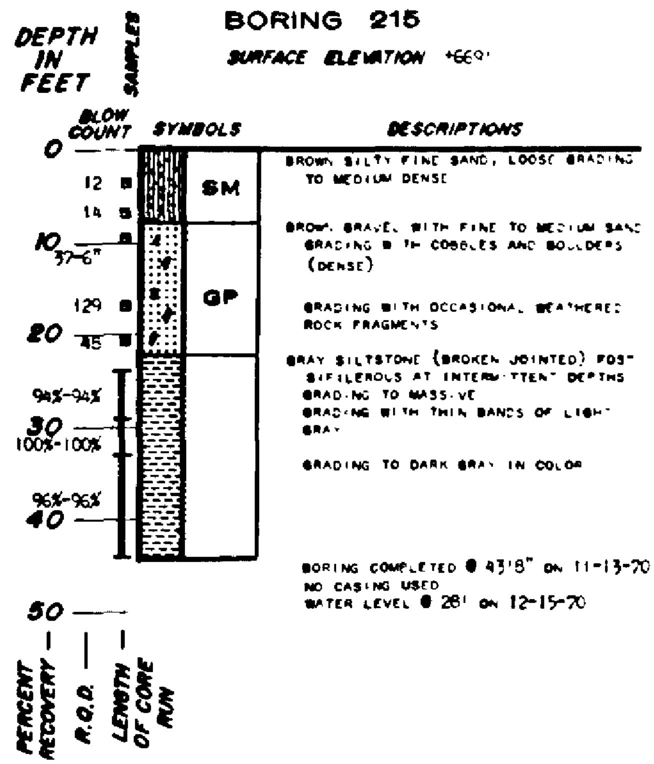
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

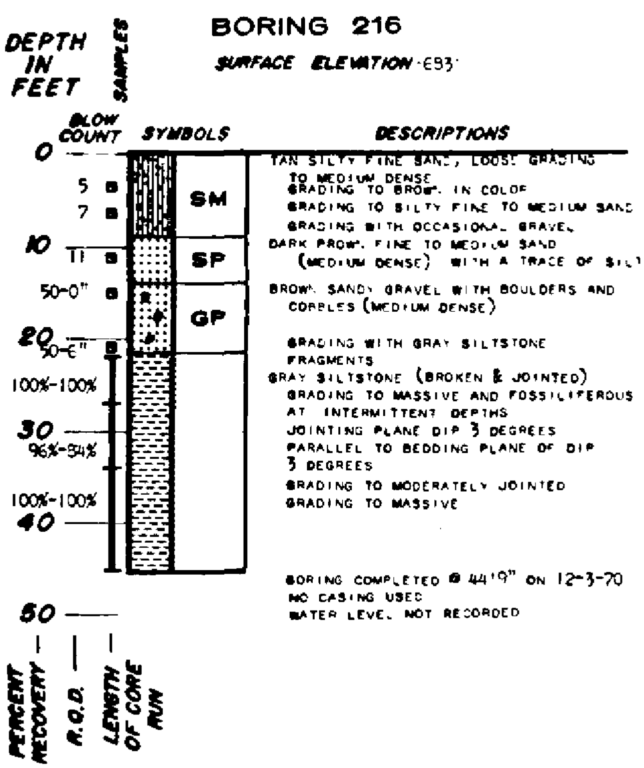
LOG OF BORINGS

FIGURE 2.5-23R, Rev 47

AutoCAD: Figure Fsar 2_5_23R.dwg







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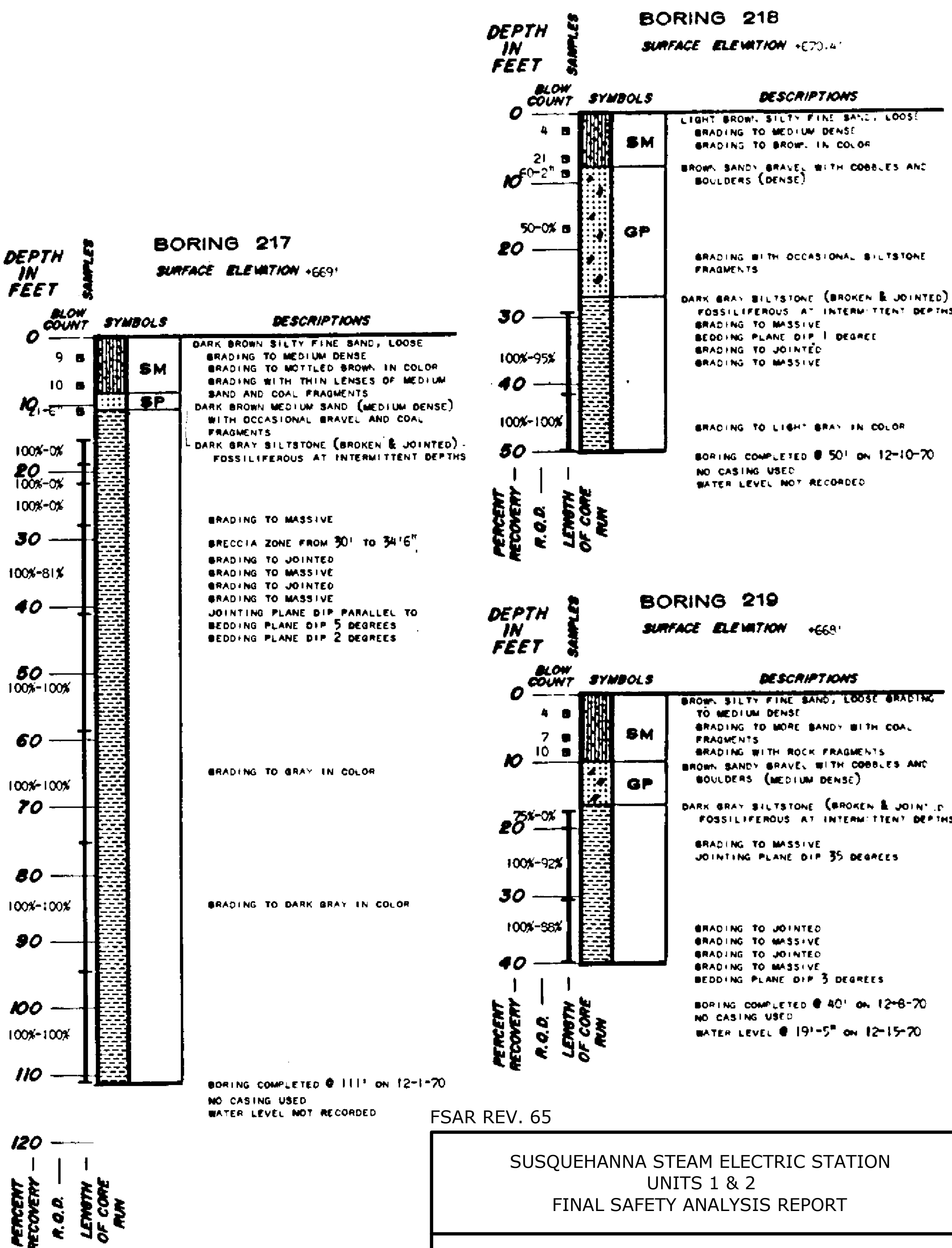
SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
LOG OF BORINGS FIGURE 2.5-23S, Rev 47

AutoCAD: Figure Fsar 2_5_23S.dwg



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SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOG OF BORINGS

FIGURE 2.5-23T, Rev 47

AutoCAD: Figure Fsar 2_5_23T.dwg

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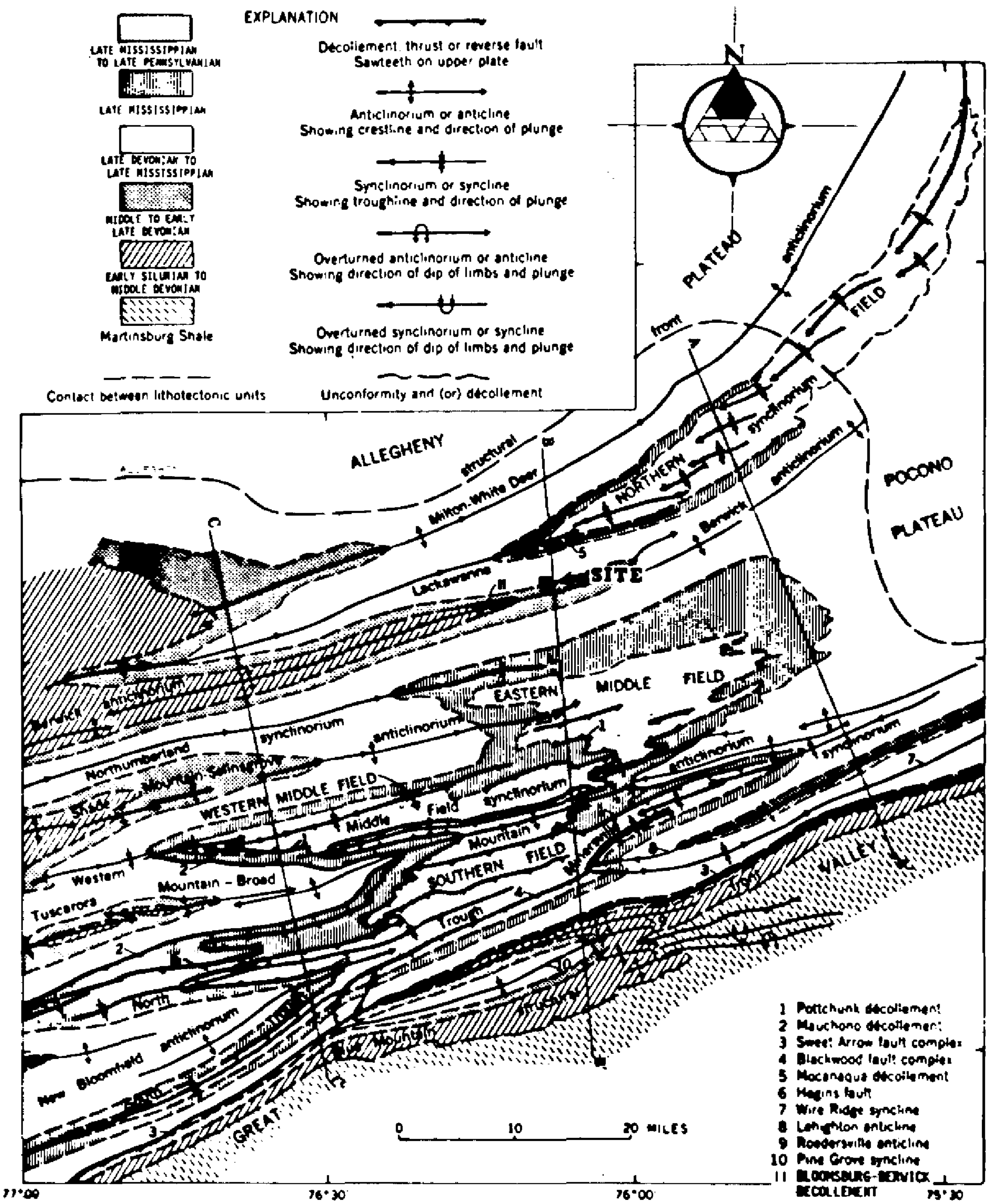
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
FINAL PLANT GRADES FIGURE 2.5-24



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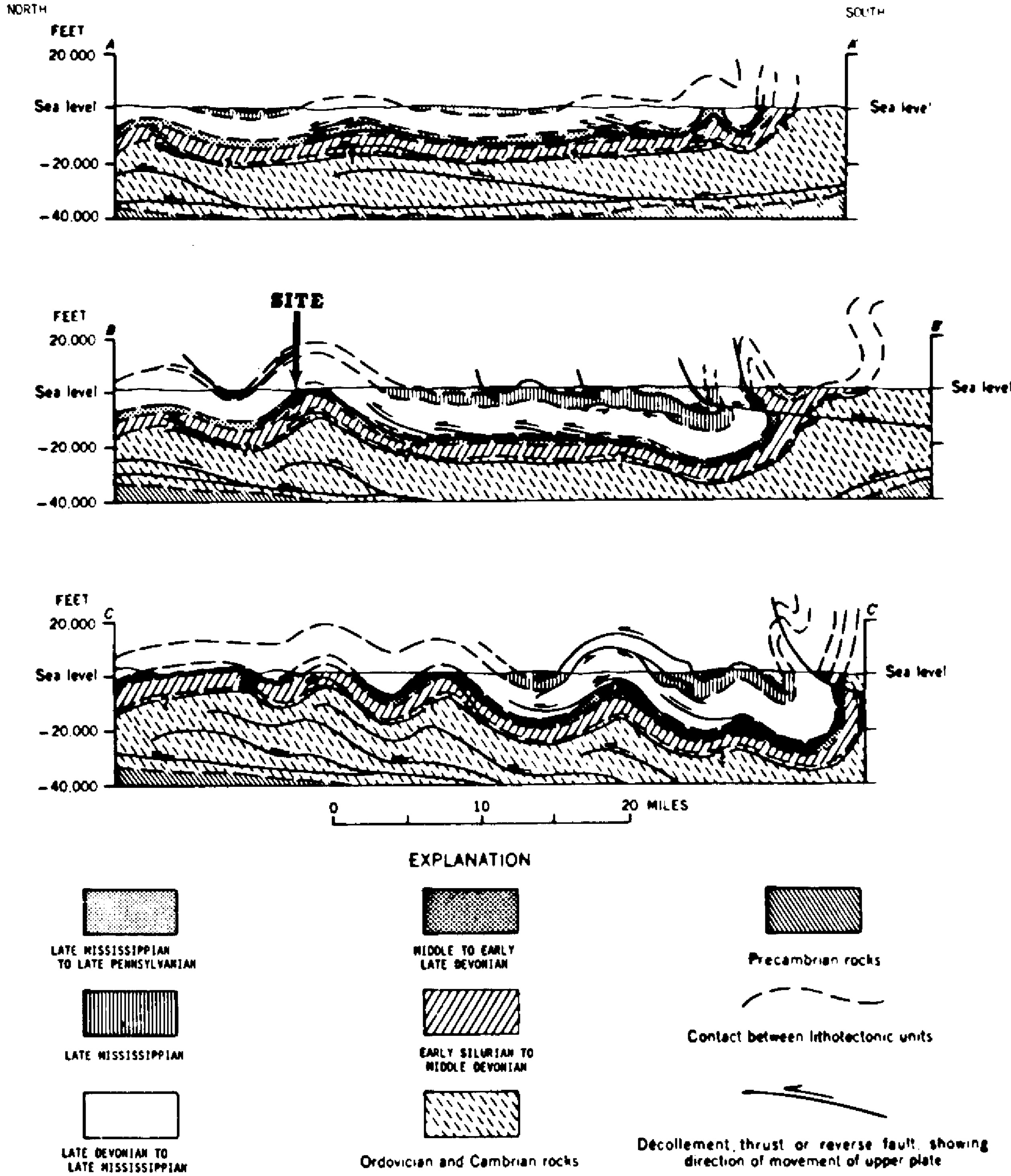
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

TECTONIC MAP OF
ANTHRACITE REGION

FIGURE 2.5-25, Rev 47

AutoCAD: Figure Fsar 2_5_25.dwg



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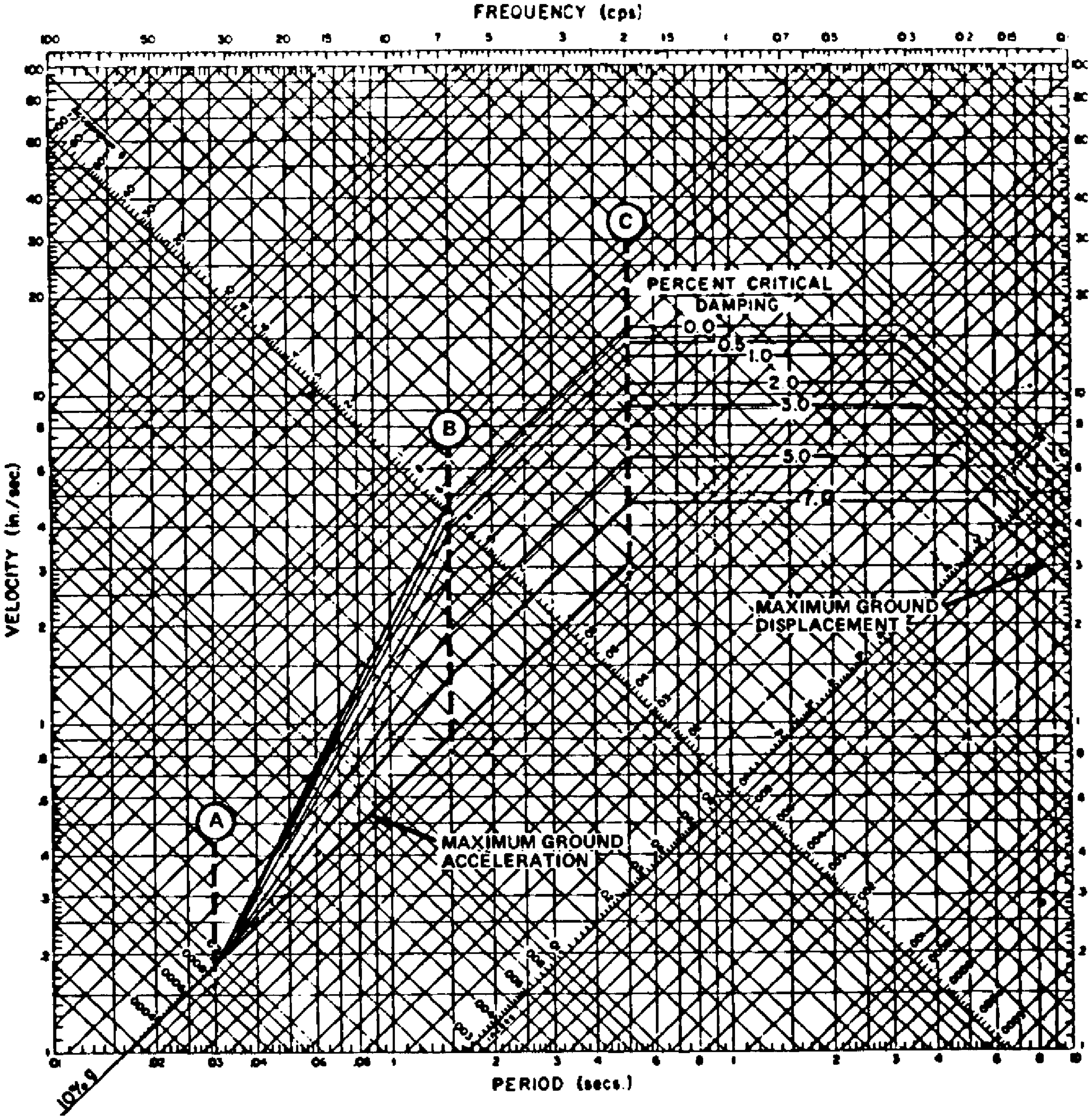
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

REPRODUCED WITH PERMISSION FROM STUDIES OF APPALACHIAN GEOLOGY CENTRAL AND SOUTHERN: JOHN WILEY AND SONS, 1970.

GEOLOGIC SECTION OF
ANTHRACITE REGION

FIGURE 2.5-26, Rev 47

AutoCAD: Figure Fsar 2_5_26.dwg



* FOR ALL ROCK FOUNDED SEISMIC CATEGORY I STRUCTURES EXCEPT THE DIESEL GENERATOR 'E' BUILDING. FOR THE DIESEL GENERATOR 'E' BUILDING SEE FIGURE 3.7B-2A.

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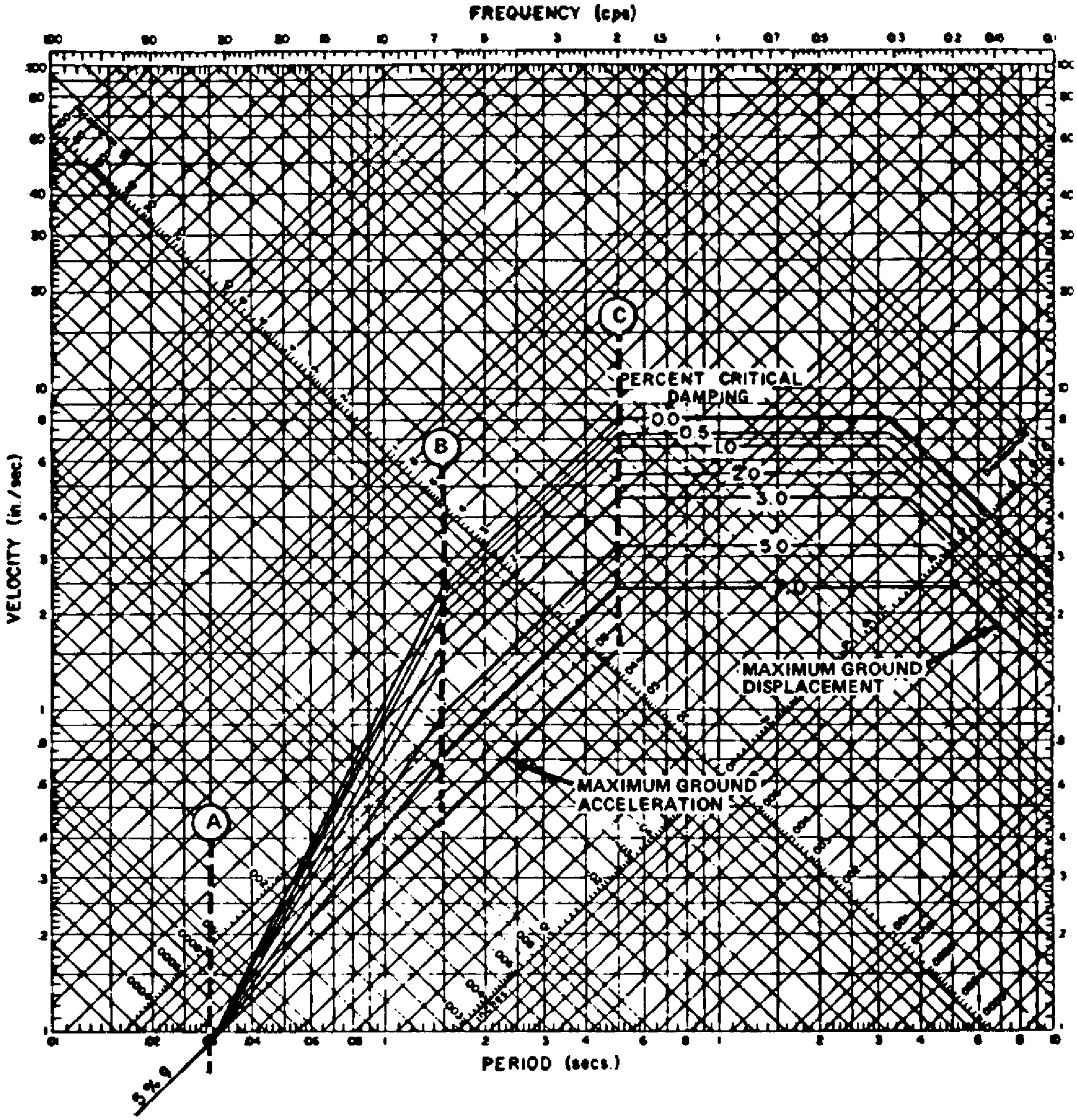
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

DESIGN RESPONSE SPECTRA-
SAFE SHUTDOWN EARTHQUAKE
HORIZONTAL COMPONENT

FIGURE 2.5-27, Rev 47

AutoCAD: Figure Fsar 2_5_27.dwg



* FOR ALL ROCK FOUNDED SEISMIC CATEGORY I
STRUCTURES EXCEPT THE DIESEL GENERATOR 'E' BUILDING.
FOR THE DIESEL GENERATOR 'E' BUILDING SEE FIGURE 3.7B-1A.

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SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
DESIGN RESPONSE SPECTRA - OPERATING BASIS EARTHQUAKE FIGURE 2.5-28, Rev 47

AutoCAD: Figure Fsar 2_5_28.dwg

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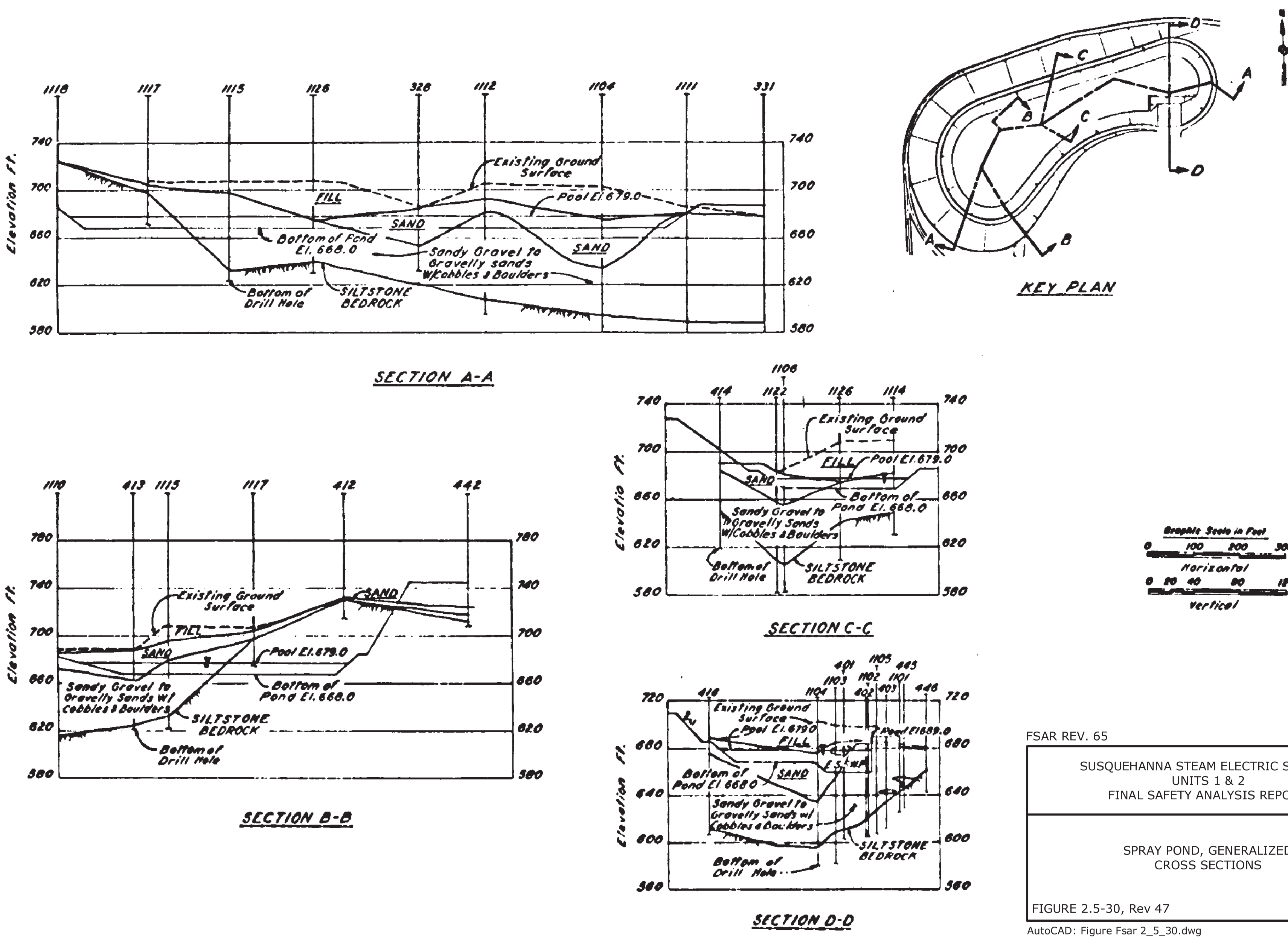
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
GEOPHYSICAL SURVEYS
FIGURE 2.5-29



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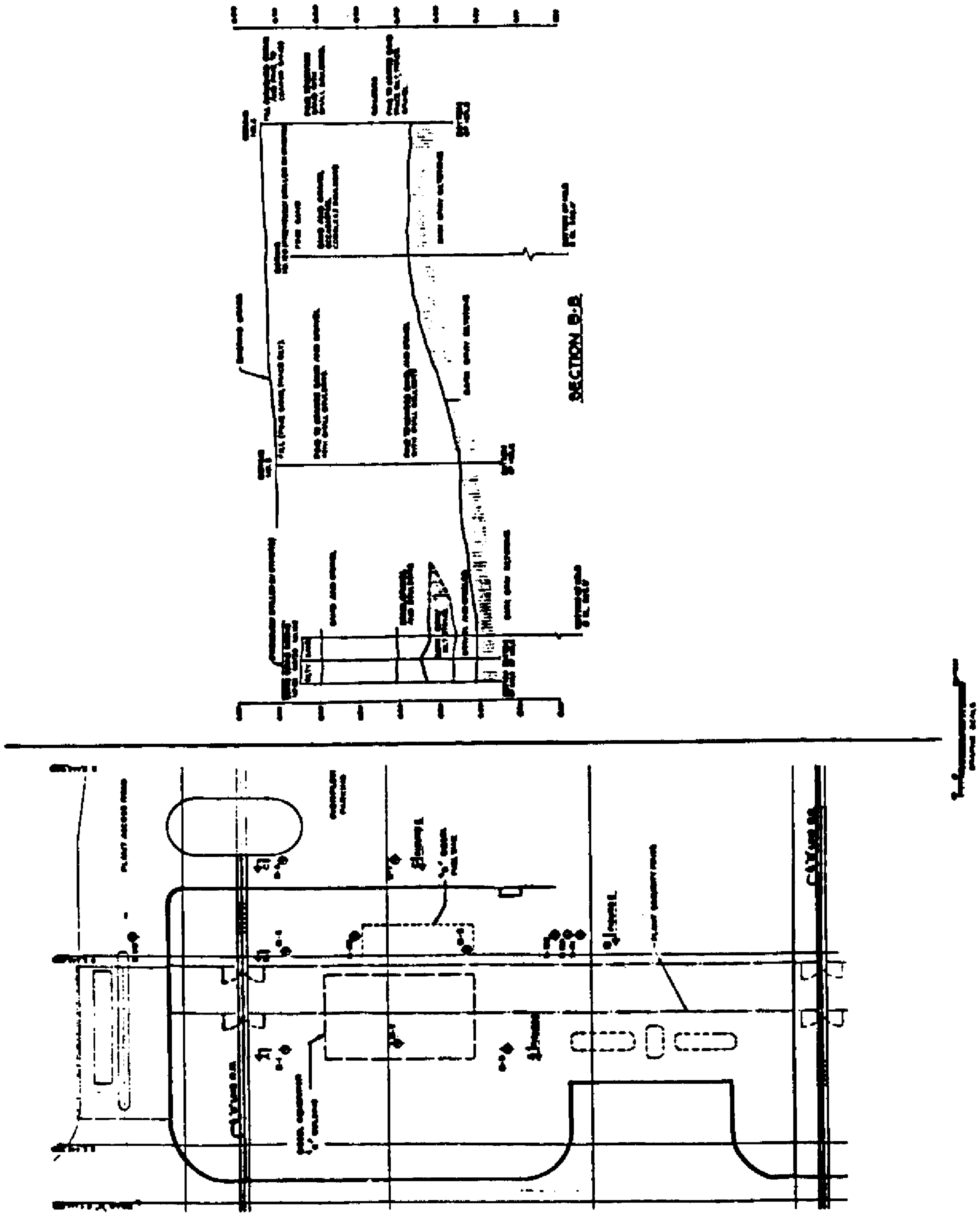
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND, GENERALIZED
CROSS SECTIONS

FIGURE 2.5-30, Rev 47

AutoCAD: Figure Fsar 2_5_30.dwg



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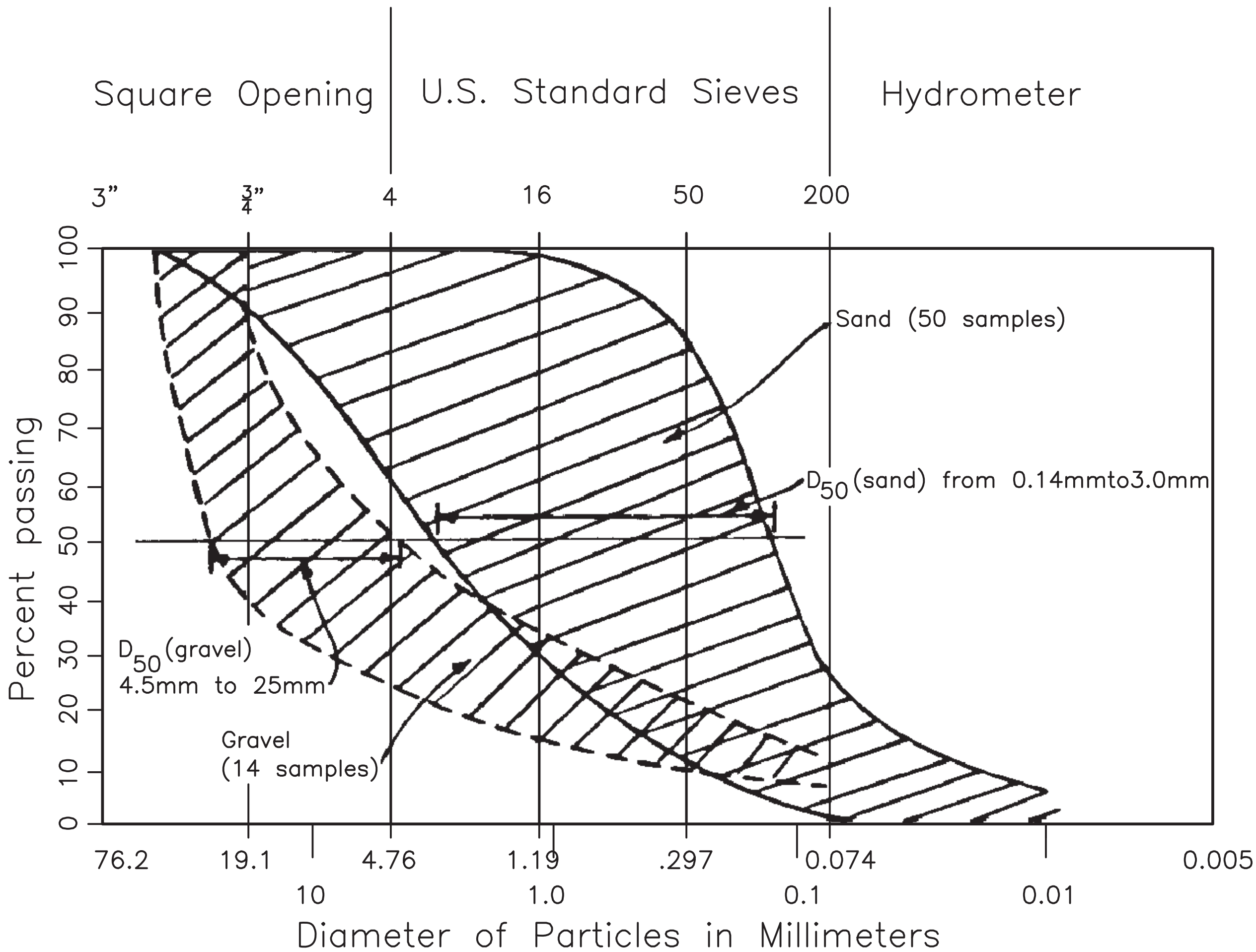
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

PLAN AND GENERALIZED
CROSS-SECTION
THROUGH DIESEL FUEL TANK
'E' DIESEL BUILDING

FIGURE 2.5-30A, Rev 47

AutoCAD: Figure Fsar 2_5_30A.dwg



Gravel		Gravel			Silt to Clay
Coarse	Fine	Coarse	Medium	Fine	

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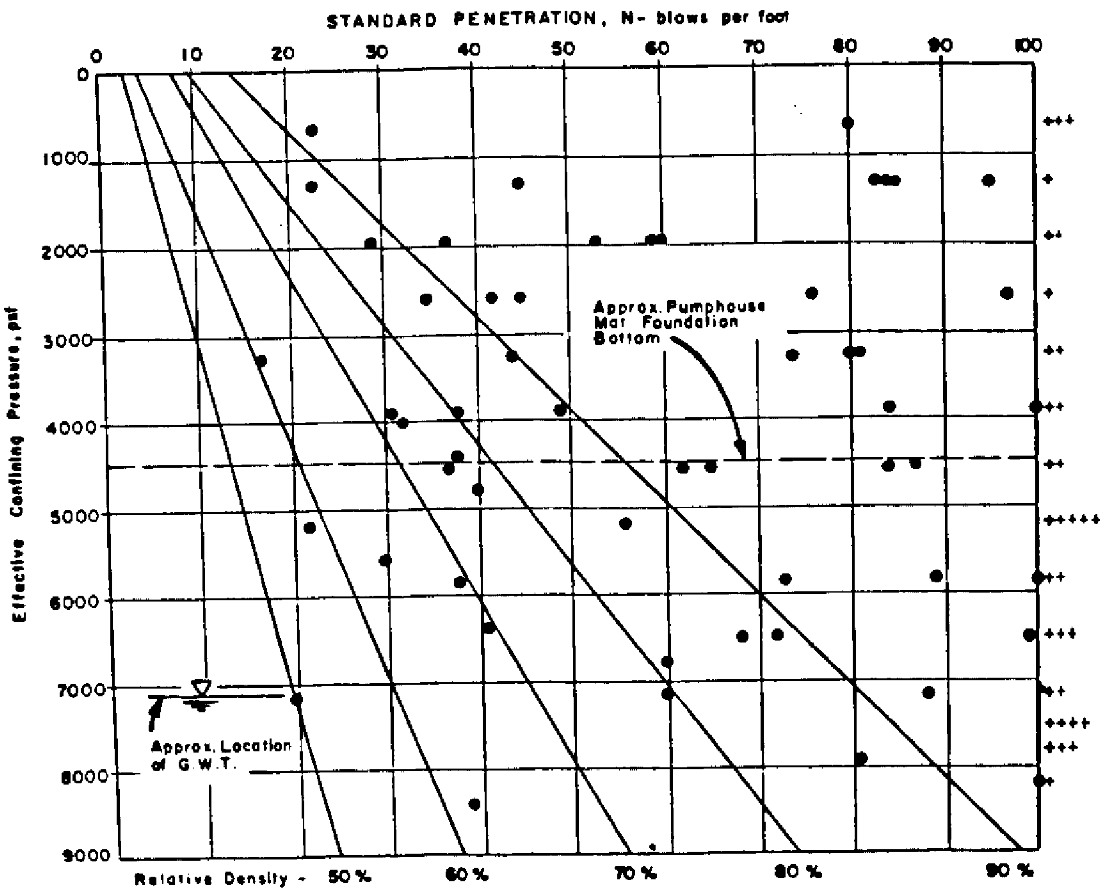
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND,
RANGE OF GRAIN
SIZE CURVES

FIGURE 2.5-31, Rev 47

AutoCAD: Figure Fsar 2_5_31.dwg



NOTES:

1. Effective Confining Pressure Vs "N" for Borings 401-407, and 1104.
2. Effective Confining Pressure Based on Average Values of $\gamma_t = 130$ pcf, $\gamma_b = 67.5$ pcf.
3. Ref. Gibbs and Holtz (1957)

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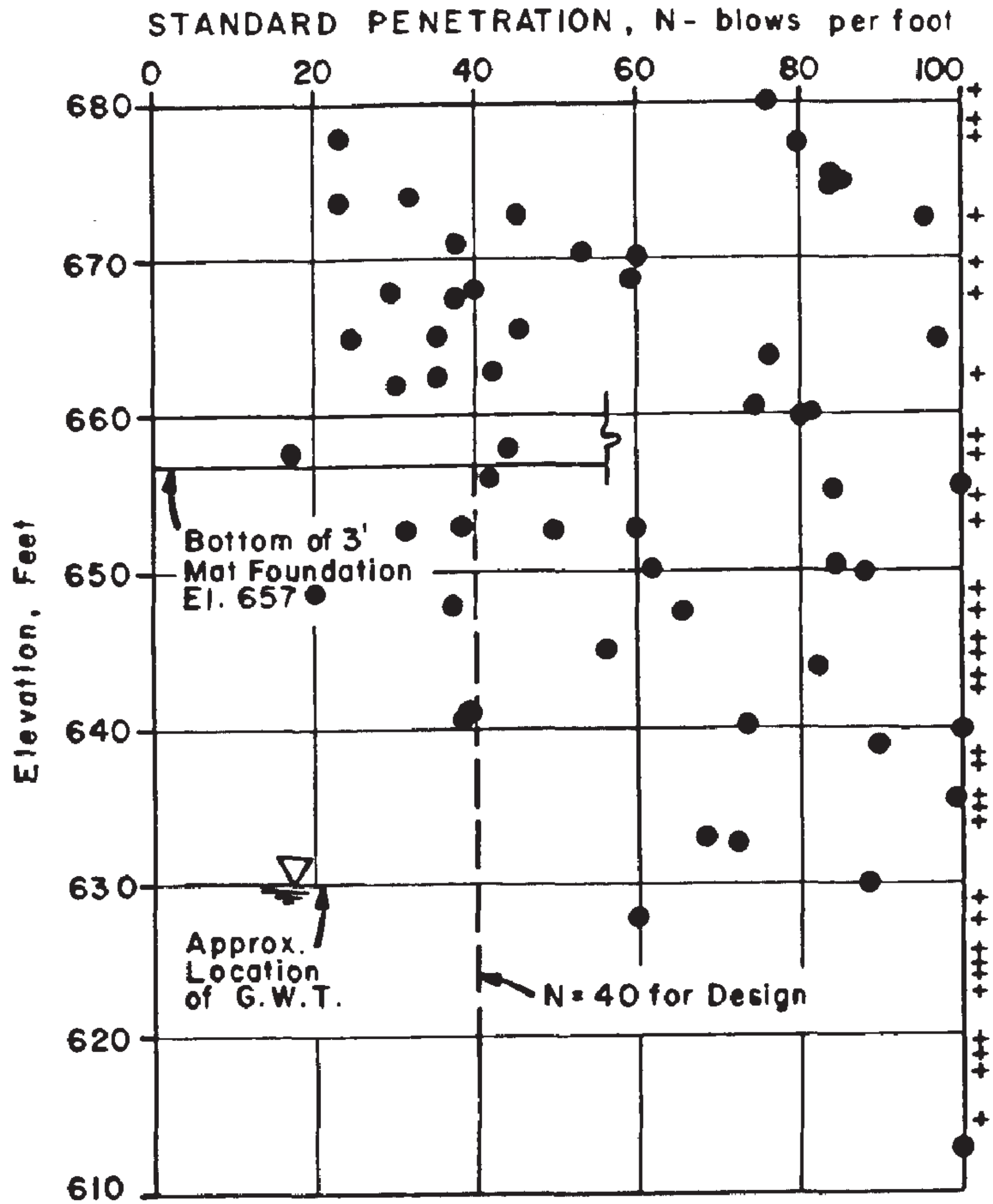
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

RELATIVE DENSITY RELATED
TO N VALUE AT
ESSW PUMPHOUSE SITE

FIGURE 2.5-32, Rev 47

AutoCAD: Figure Fsar 2_5_32.dwg



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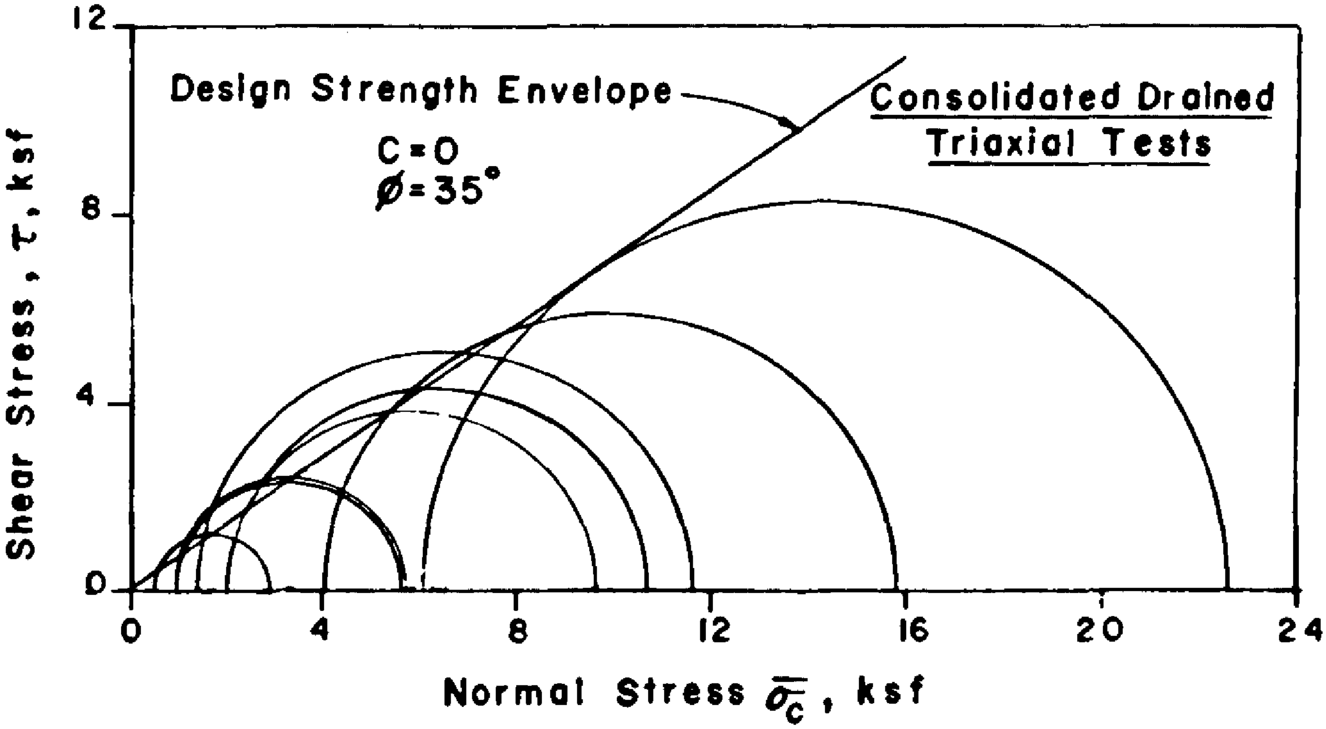
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

"N" VALUE VERSUS
ELEVATION AT ESSW PUMPHOUSE SITE

FIGURE 2.5-33, Rev 47

AutoCAD: Figure Fsar 2_5_33.dwg



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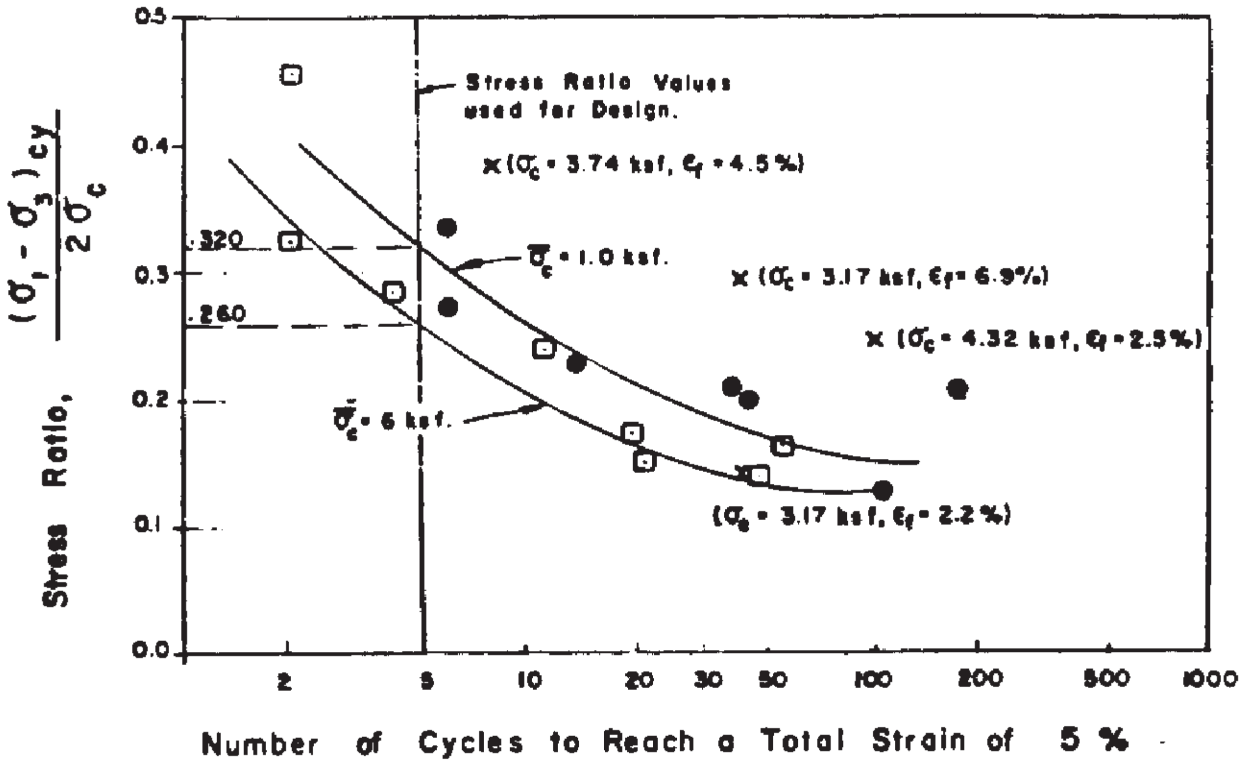
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND,
CONSOLIDATED DRAINED TRIAXIAL
TEST - MOHR CIRCLE DIAGRAM

FIGURE 2.5-34, Rev 47

AutoCAD: Figure Fsar 2_5_34.dwg



LEGEND

⊡ Undisturbed Test @ $\bar{\sigma}_c$ = 6 ksf

● Undisturbed Test @ $\bar{\sigma}_c$ = 1 ksf.

$\bar{\sigma}_c$ = Effective Consolidation Pressure

$(\sigma_1 - \sigma_3)_{cy}$ = Cyclic Deviator Stress.

x Test conducted by Dames & Moore with confining pressure (σ_c) and failure strain (ϵ_f) shown in the parentheses

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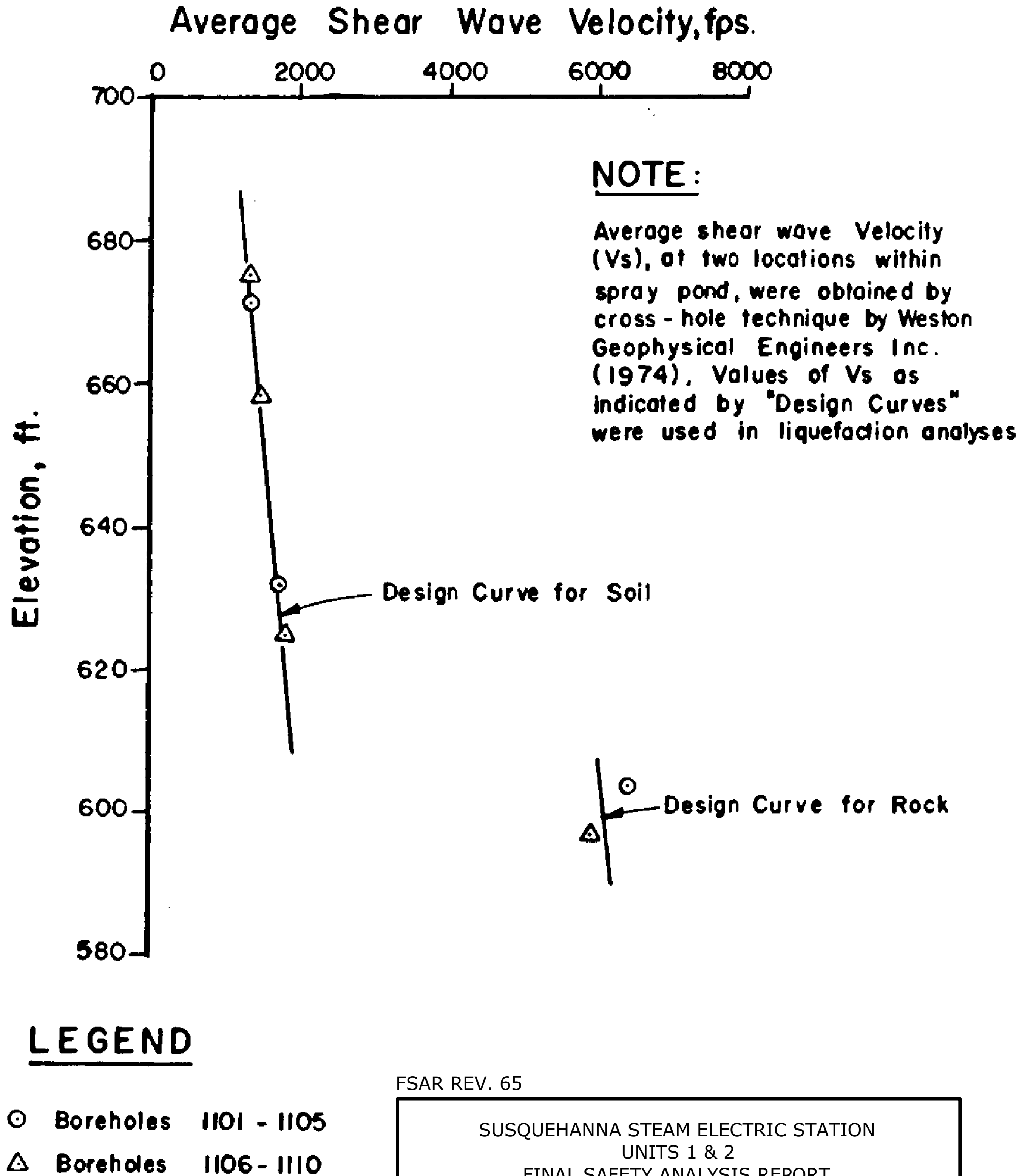
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND,
SUMMARY OF CYCLIC SHEAR TEST RESULTS

FIGURE 2.5-35, Rev 47

AutoCAD: Figure Fsar 2_5_35.dwg



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UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND,
SHEAR WAVE VELOCITY
DESIGN CURVES

FIGURE 2.5-36, Rev 47

AutoCAD: Figure Fsar 2_5_36.dwg

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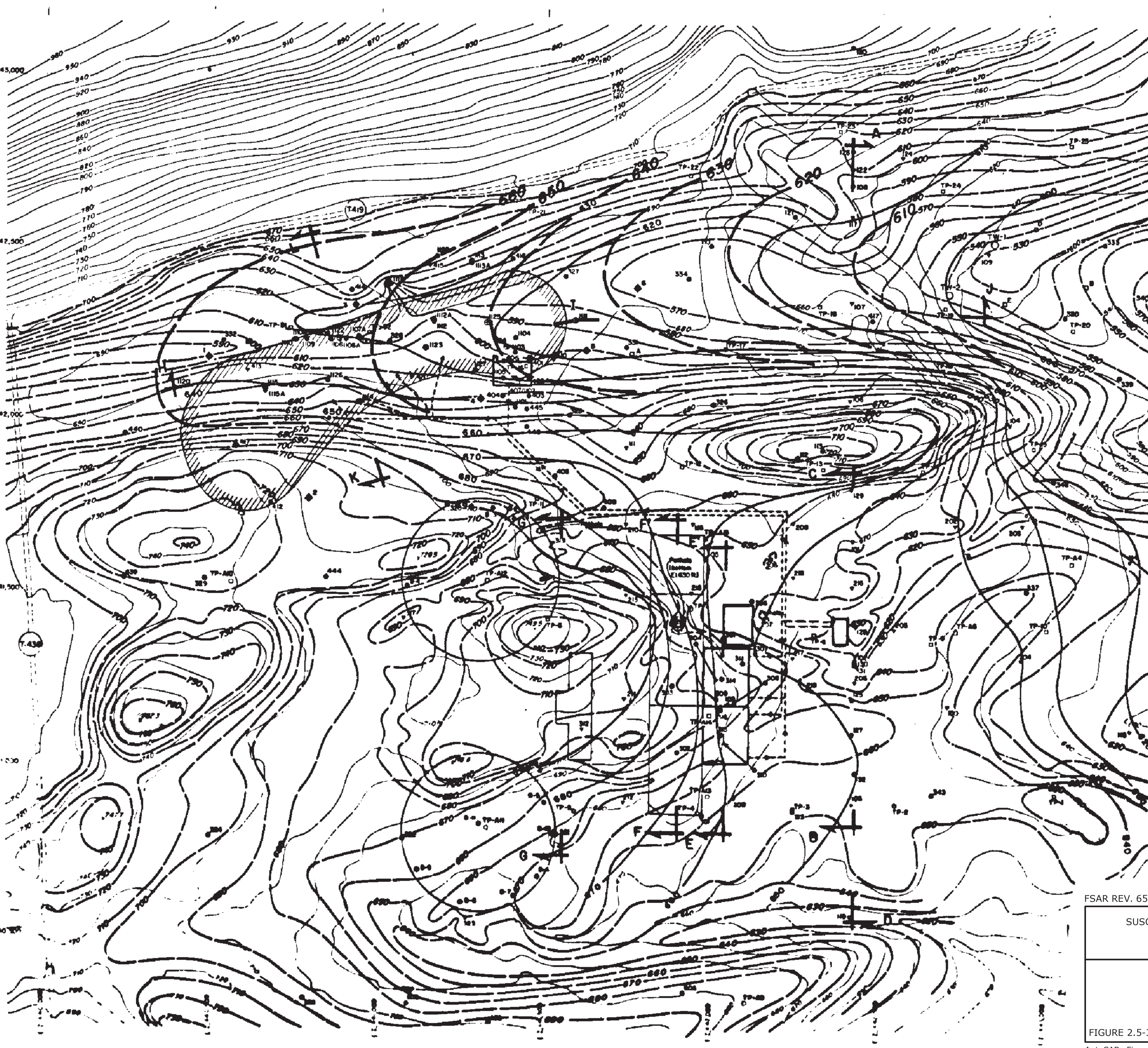
SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
LOCATION AND LIMITS OF EXCAVATION FILL AND BACKFILL FOR CLASS I STRUCTURES FIGURE 2.5-37



EXPLANATION

- 610 — Contours for estimated ground water table, incorporating seepage loss from spray pond having liner such that seepage loss does not exceed 1.2 X 10⁶ gallons in 30 days.
- 600 — Water table contours in the plant site area are based on measurements taken on June 30, 1971. Measurements in the pond area were taken on August 6, 1974.
- E E' — Location of section line.
- 610 — Contour on top of rock (see note 2); contour interval 10 feet.
- Bedrock outcrop; may include minor amounts of talus or overburden. Boundaries are approx.
- 311, 304 — Exploration hole; vertical hole (left) and angle hole showing trace of hole projected to surface (right).
- 208 — Exploration hole utilized for water level observation.
- 111 — Exploration hole used for permeability measurements.
- TW-1 — Water well (used for construction).
- A, TP 11 — Test pits.
- 2 — Spray pond ground water observation well

Seismic Category I facilities

- Category I structure.
- Category I pipeline.
- Category I impoundment.

NOTES:

1. Screened topographic base shows preconstruction terrain: contour interval 10 feet.
2. Top-of-rock contours show original top of rock surface based on interpretation of drill hole and seismic refraction data modified, where appropriate, by information obtained during construction. Contours north of town road T-419 are not shown.

SCALE IN FEET

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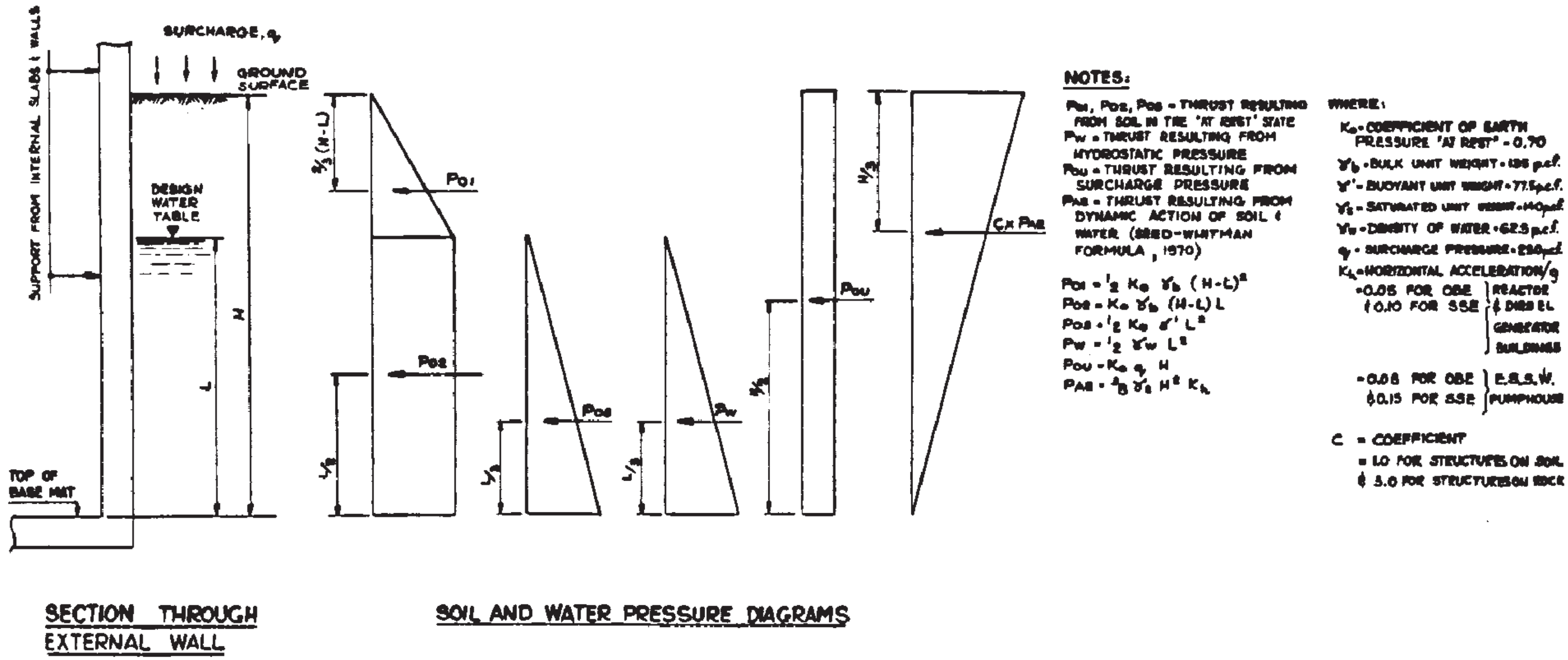
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

LOCATION OF SPRAY POND
WITH CONTOURS ON TOP OF
ROCK AND WATER TABLE

FIGURE 2.5-38, Rev 47

AutoCAD: Figure Fsar 2_5_38.dwg



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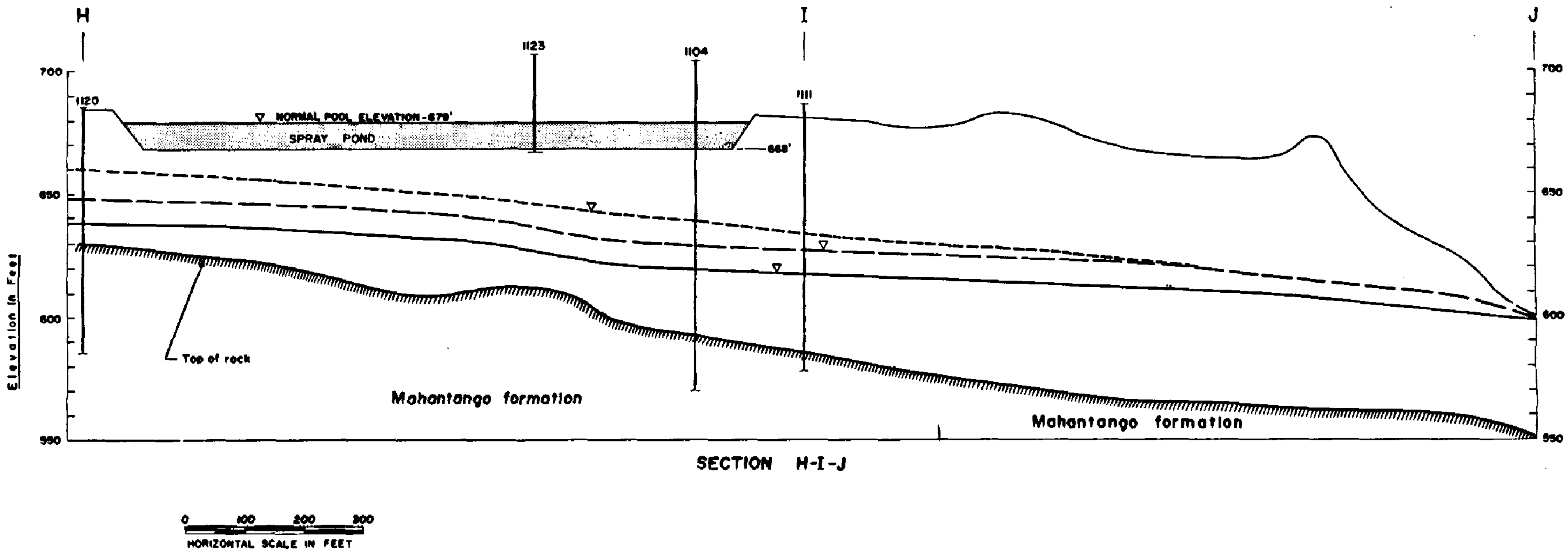
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

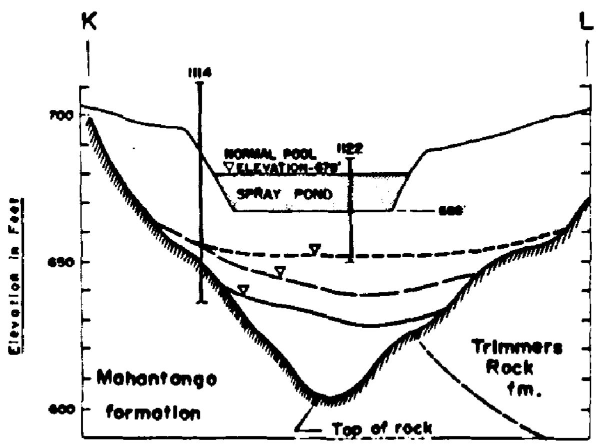
LATERAL SOIL PRESSURE DIAGRAMS

FIGURE 2.5-39, Rev 47

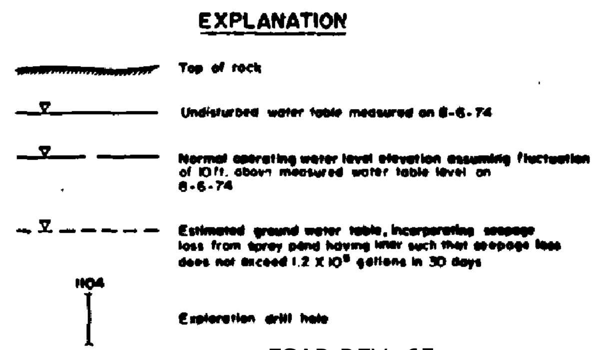
AutoCAD: Figure Fsar 2_5_39.dwg



SECTION H-I-J



SECTION K-L



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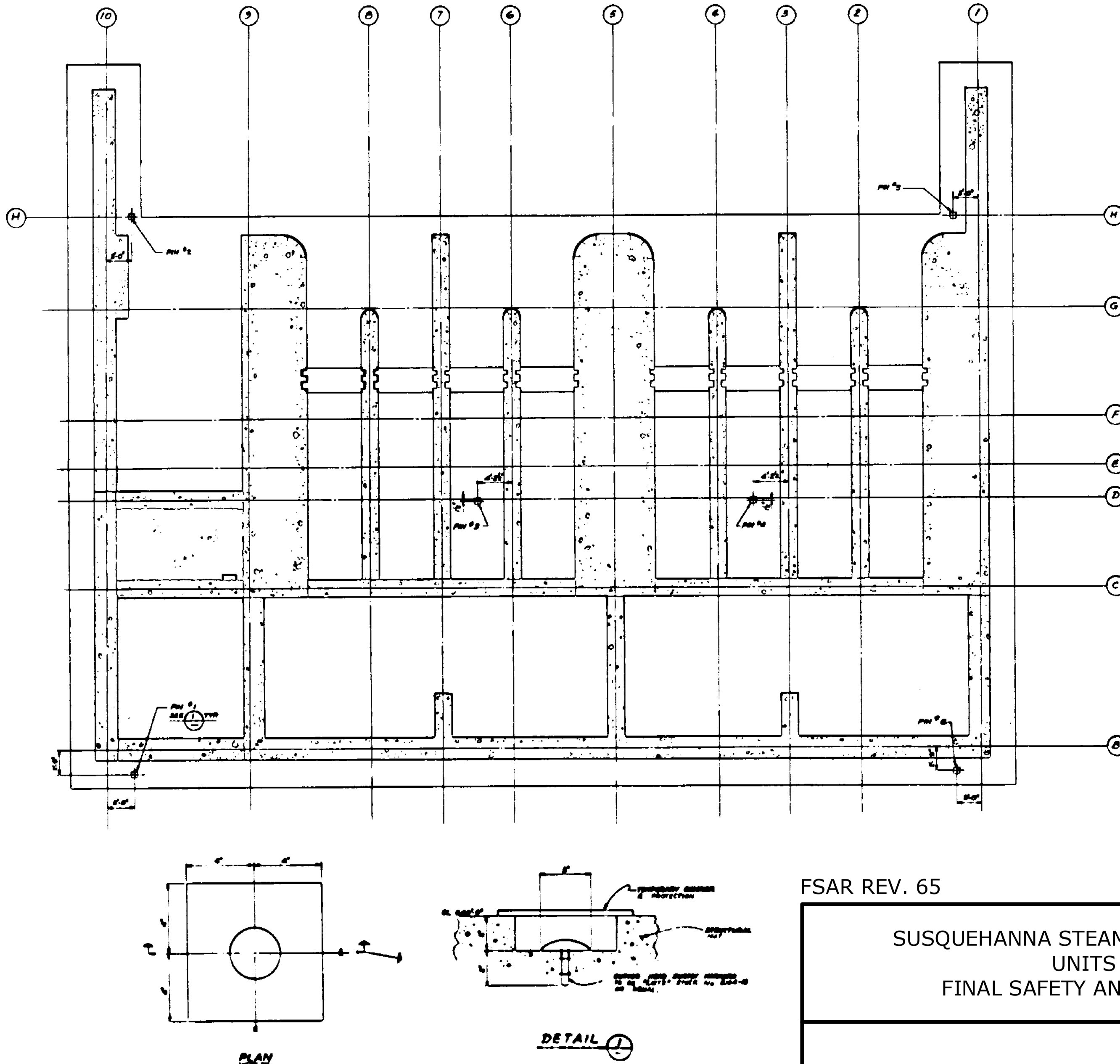
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND
MEASURED & PROJECTED
WATER LEVELS

FIGURE 2.5-40, Rev 47

AutoCAD: Figure Fsar 2_5_40.dwg



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DETAILS OF SETTLEMENT PINS
CAST IN ESSW PUMPHOUSE BASEMAT

FIGURE 2.5-41, Rev 47

AutoCAD: Figure Fsar 2_5_41.dwg

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SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
SPRAY POND, EARTHWORK PLAN
FIGURE 2.5-42

THIS FIGURE HAS BEEN
REPLACED BY DWG.
C-63, Sh. 1

FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
Figure 2.5-43 replaced by dwg. C-63, Sh. 1 FIGURE 2.5-43, Rev. 48

AutoCAD Figure 2_5_43.doc

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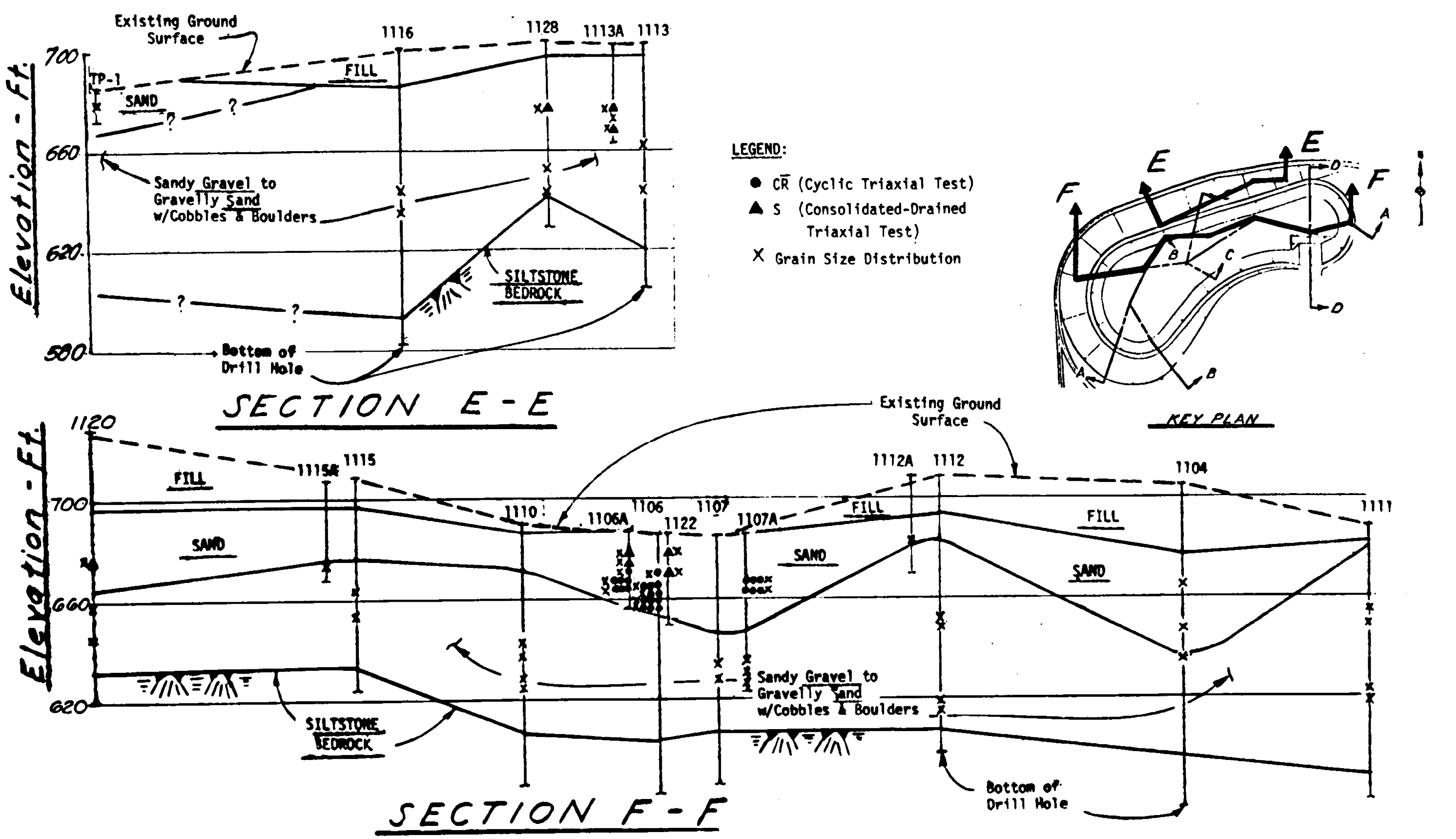
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND,
BORING AND TEST PITS
LOCATION PLAN

FIGURE 2.5-44



NOTES:

1. For details of laboratory tests refer to the Geotechnical Engineers, Inc. Report, dated Oct. 11, 1974. The locations of soil test samples are also shown on the boring logs.
2. Borings with A-designation were drilled adjacent to the borings with the same number. The locations of these borings shown on this drawing are approximate.

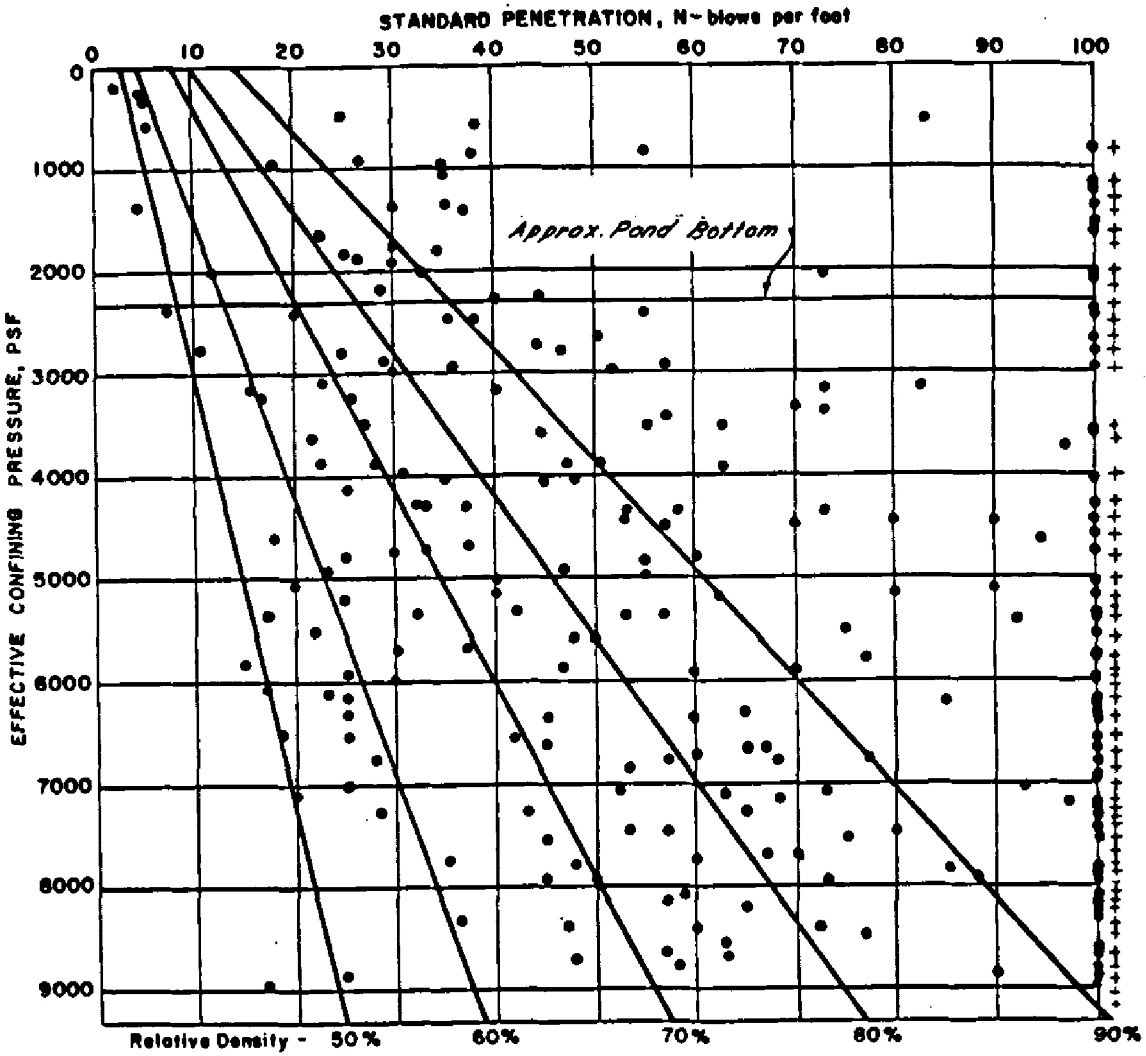
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND, GENERALIZED
CROSS SECTION WITH
LABORATORY TEST LOCATIONS

FIGURE 2.5-45, Rev 47

AutoCAD: Figure Fsar 2_5_45.dwg



NOTES:

1. Effective Confining Pressure VS. 'N' Shown by Symbol •
2. Effective Confining Pressure Based on Average Values of $\delta_t = 130$ pcf, $\delta_b = 67.5$ pcf.
3. Ref: Gibbs and Holtz (1957)

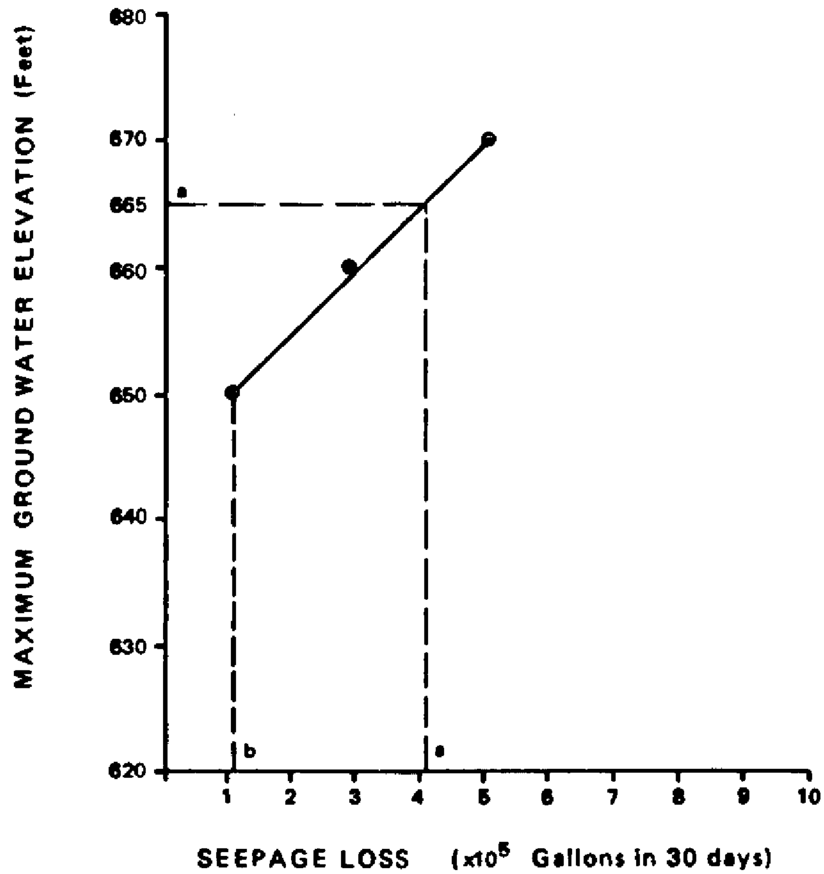
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND, RELATIVE DENSITY
RELATED TO "N" VALUE

FIGURE 2.5-46, Rev 47

AutoCAD: Figure Fsar 2_5_46.dwg



NOTES:

a. ELEVATION 665 IS THE MAXIMUM GROUND WATER LEVEL AT WHICH THE FACTOR OF SAFETY AGAINST LIQUEFACTION IS AT LEAST 1.2. THIS CORRESPONDS TO A SEEPAGE LOSS OF APPROXIMATELY 4.1×10^5 GALLONS IN 30 DAYS.

b. THE POND LINER IS DESIGNED TO LIMIT THE SEEPAGE TO LESS THAN 1.2×10^5 GALLONS IN 30 DAYS TO PROVIDE AN ADDITIONAL MARGIN OF SAFETY.

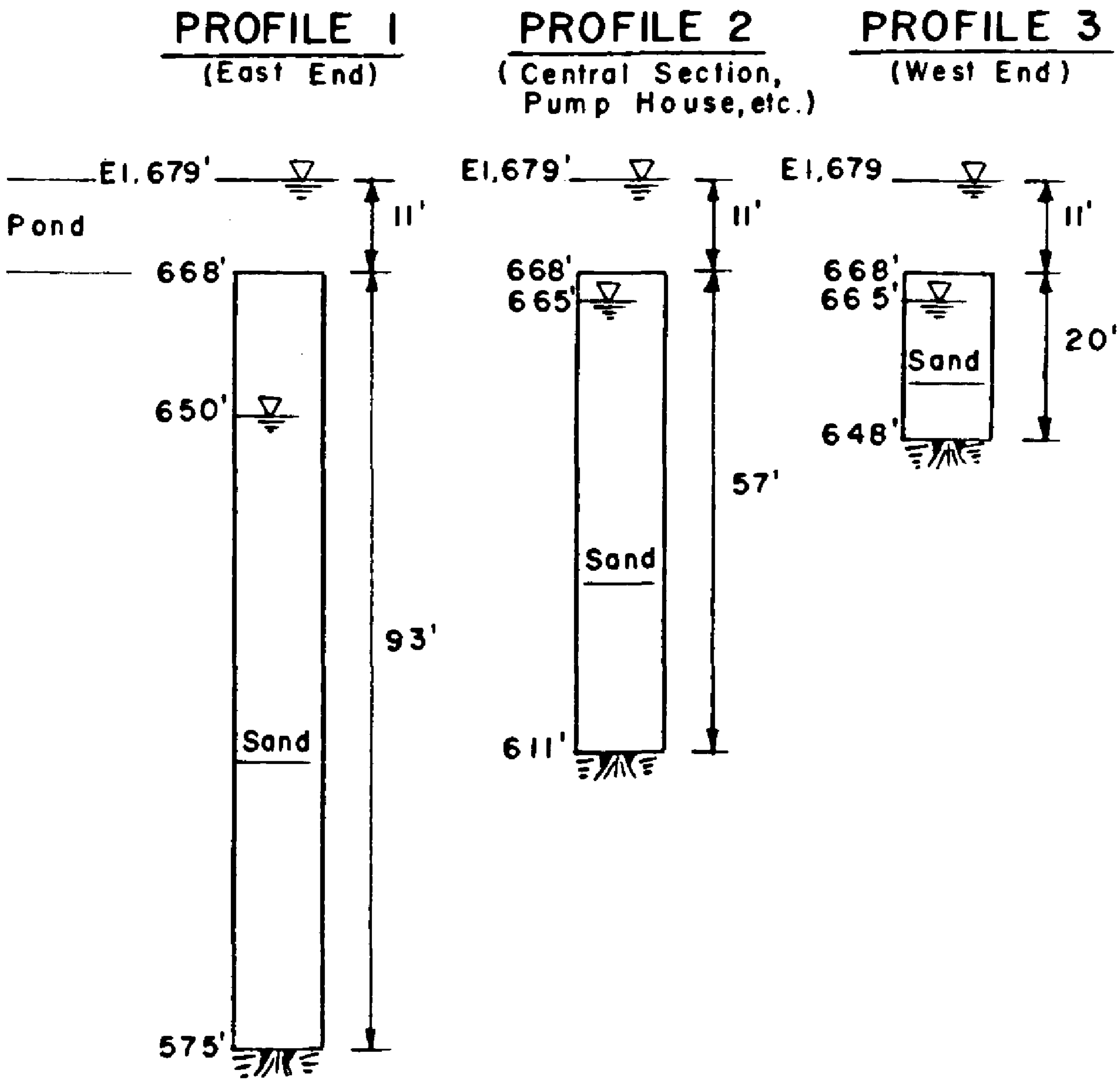
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND,
RELATIONSHIP BETWEEN SEEPAGE
LOSS & MAXIMUM GROUND
WATER ELEVATION

FIGURE 2.5-47, Rev 47

AutoCAD: Figure Fsar 2_5_47.dwg



NOTES:

1. Soil profiles were conservatively assumed to consist only of Sand.
2. Saturated Unit Weight of Sand = 130 pcf
   Buoyant Unit Weight of Sand = 67.5 pcf.
3. Ground water tables shown in the profiles are the maximum level used in liquefaction analyses.

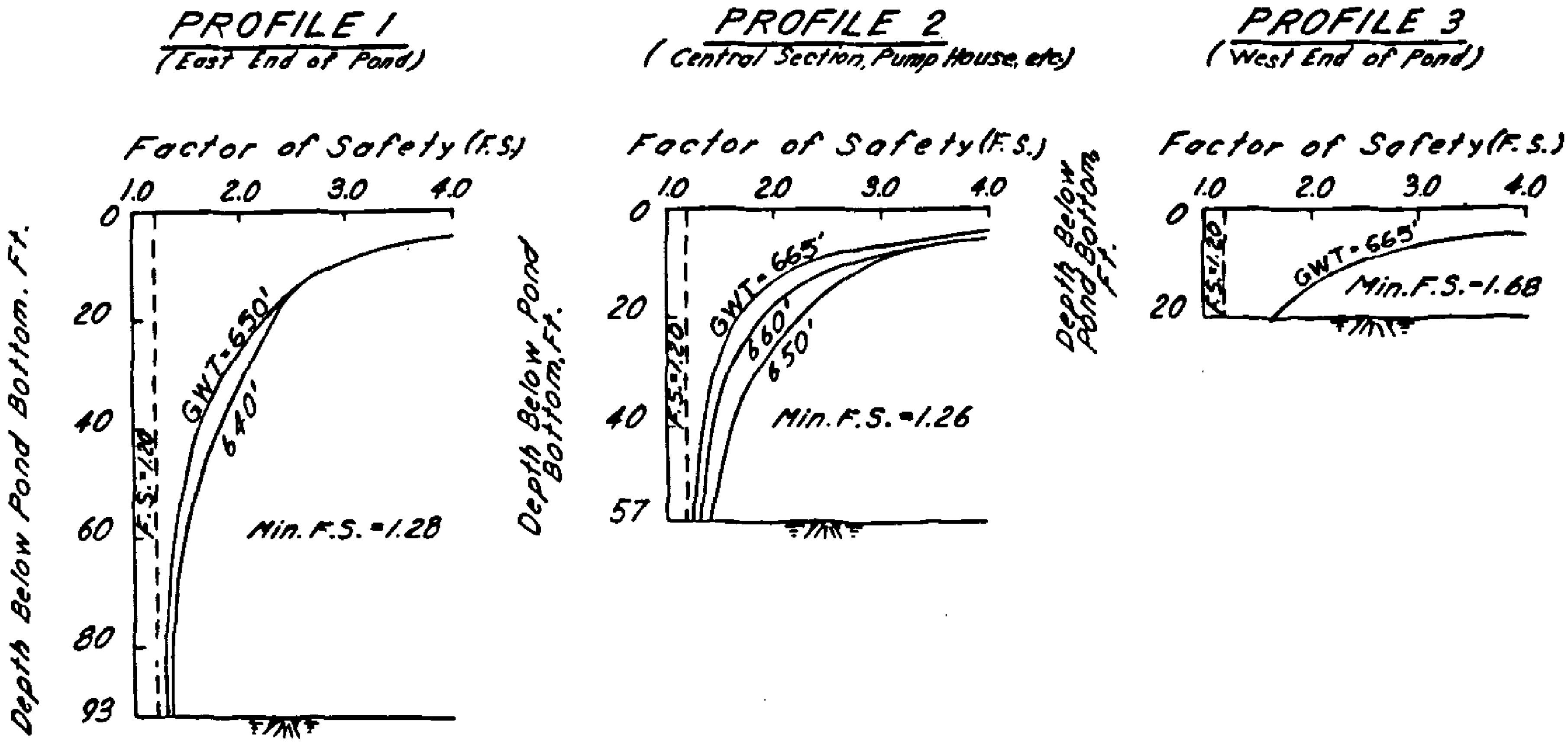
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND,
SOIL PROFILES USED IN
LIQUIFICATION ANALYSIS

FIGURE 2.5-48, Rev 47

AutoCAD: Figure Fsar 2_5_48.dwg



NOTES:

1. Earthquake – Bechtel Synthetic (Ref: FSAR, SECTION 3.7.1.2)
2. SSE of 0.15g.
3. G.W.T.-Ground Water Table
4. Pond bottom at El. 668'
5. Acceptable F.S. =1.20.

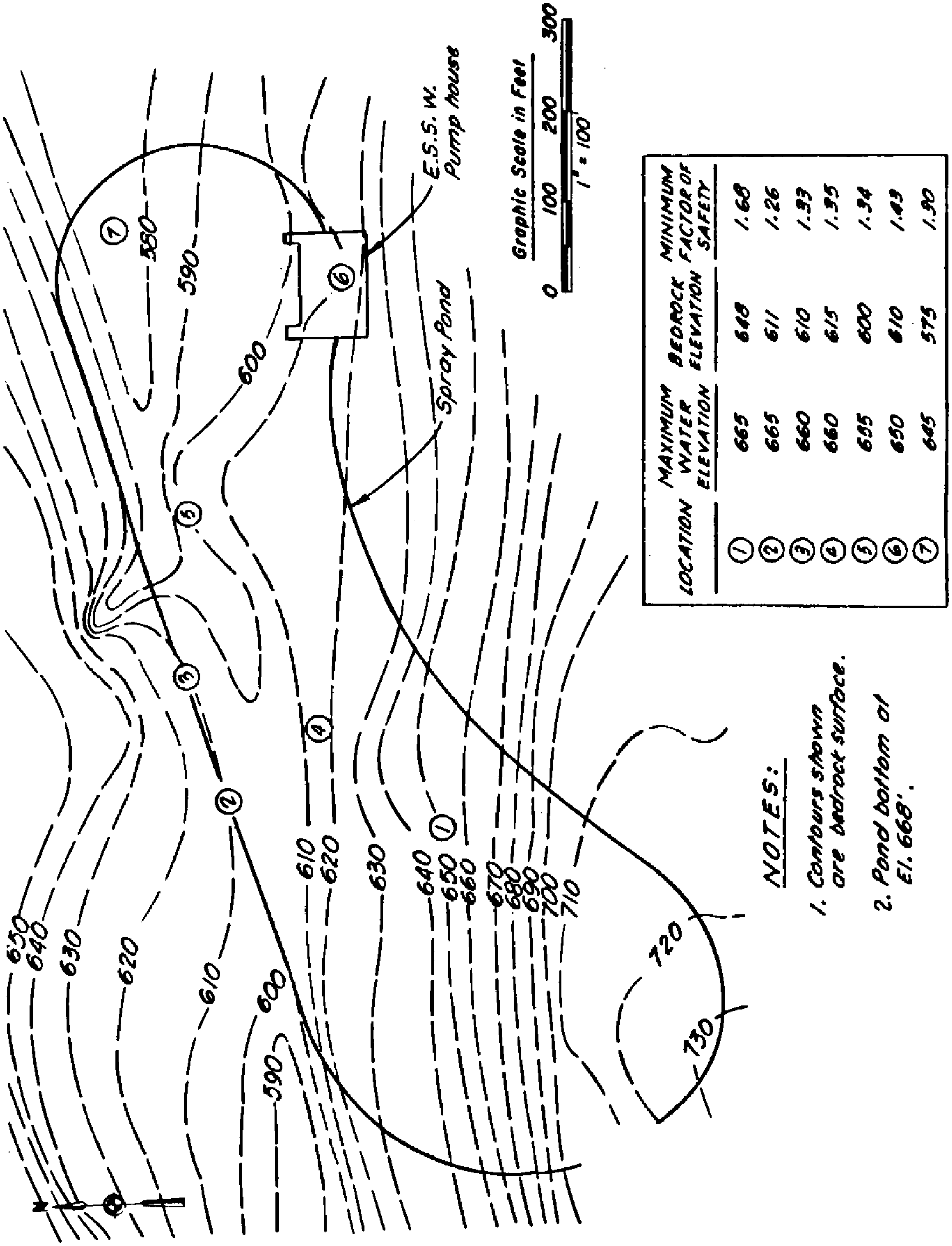
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND, FACTOR OF SAFETY VARIATION WITH GROUNDWATER TABLE & DEPTH

FIGURE 2.5-49, Rev 47

AutoCAD: Figure Fsar 2_5_49.dwg



LOCATION	MAXIMUM WATER ELEVATION	BEDROCK ELEVATION	MINIMUM FACTOR OF SAFETY
1	665	648	1.68
2	665	611	1.26
3	660	610	1.33
4	660	615	1.35
5	655	600	1.34
6	650	610	1.43
7	645	575	1.90

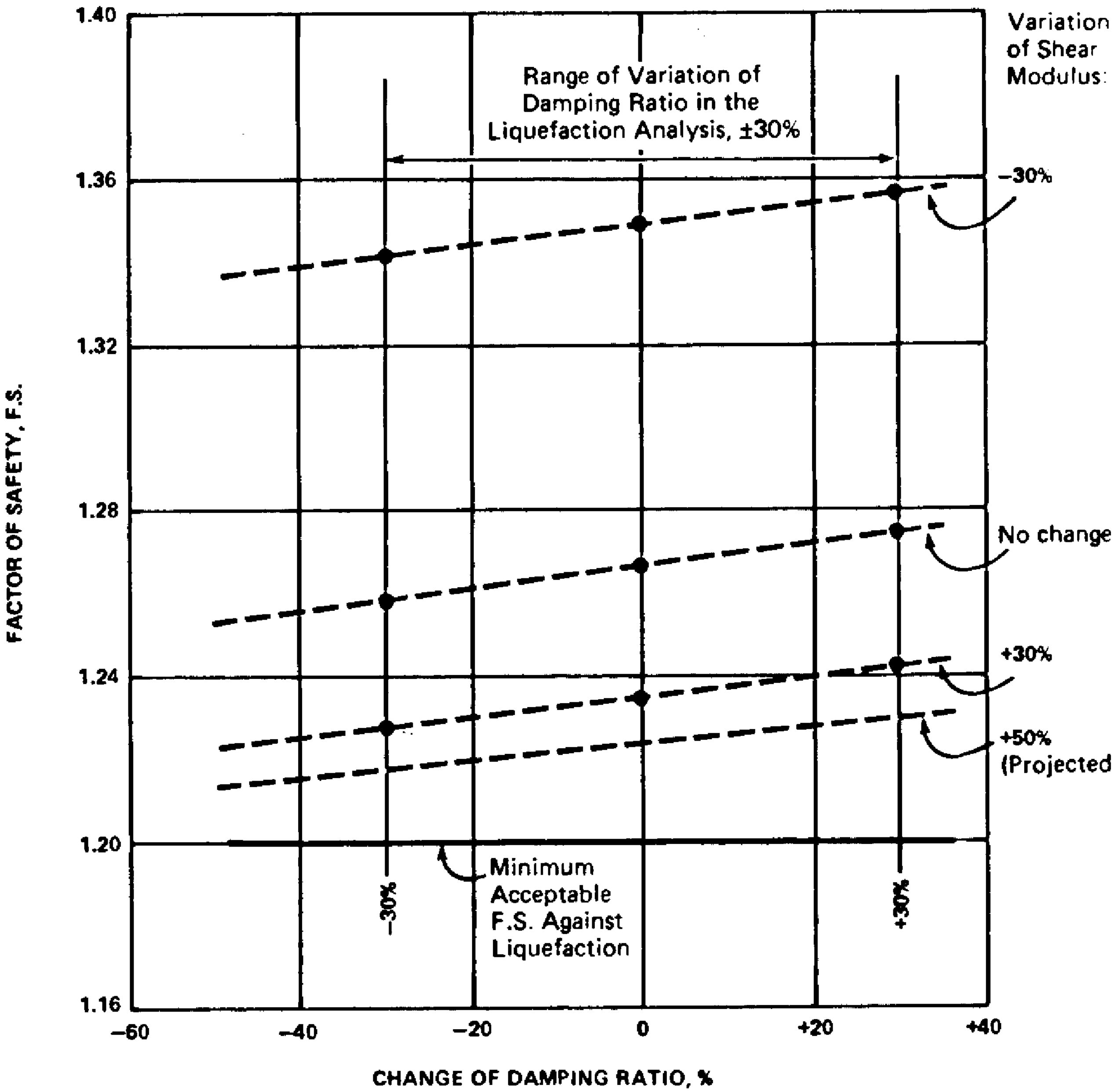
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND, MINIMUM FACTOR
OF SAFETY AGAINST
LIQUIFICATION AT SELECTED LOCATIONS

FIGURE 2.5-50, Rev 47

AutoCAD: Figure Fsar 2_5_50.dwg



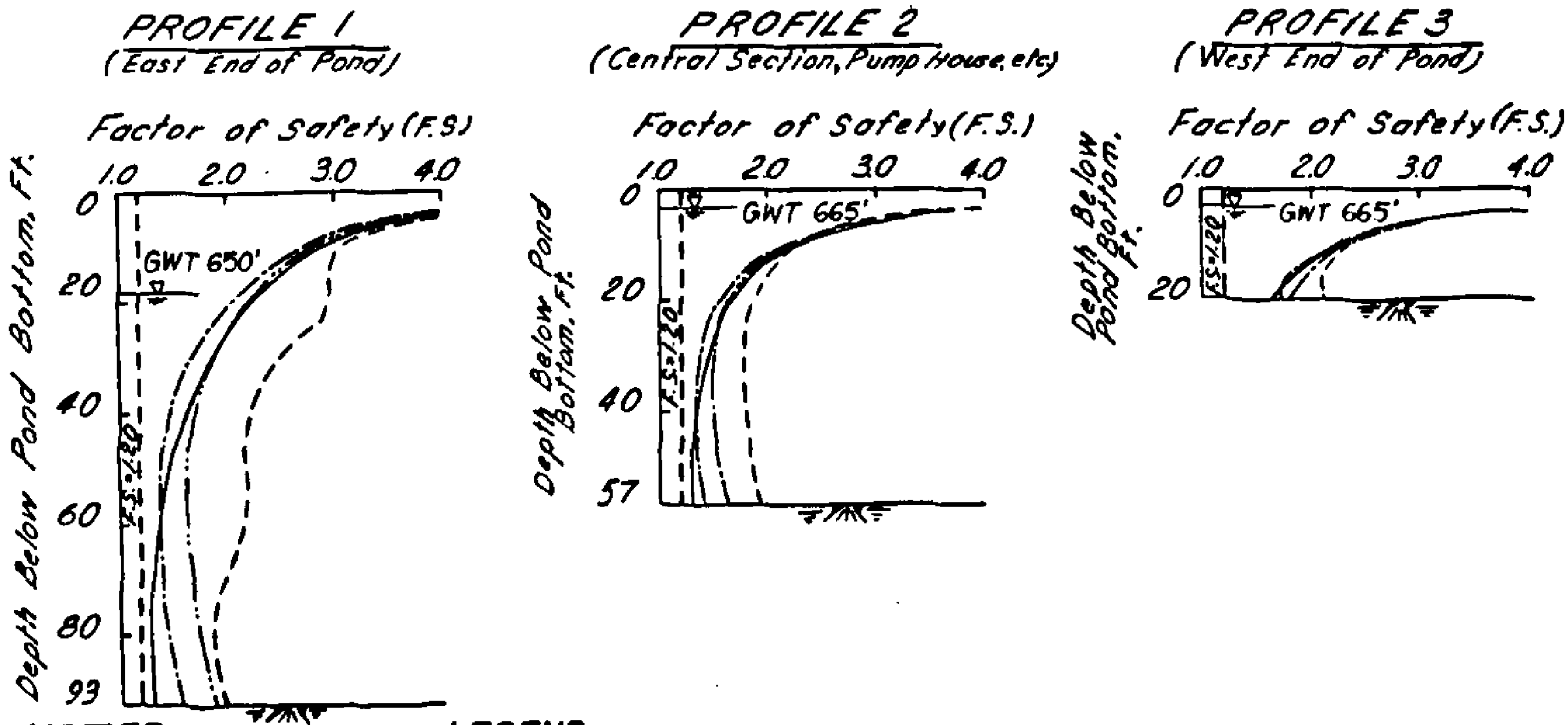
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

VARIATION FACTOR OF
SAFETY (F.S.) VERSUS
CHANGE OF DAMPING RATIO

FIGURE 2.5-50A, Rev 47

AutoCAD: Figure Fsar 2_5_50A.dwg



NOTES:

1. Comparison shown only for the profiles with the maximum ground water table (GWT). Comparison of profiles with lower ground water table, See Table 10.
2. Pond bottom at El. 668'
3. Acceptable F.S. = 1.20.

LEGEND:

Bechtel Synthetic (Design Earthquake)	————
Golden Gate Earthquake	-·-·-·-
Helena Earthquake	- - - -
Parkfield Earthquake	—·—·—

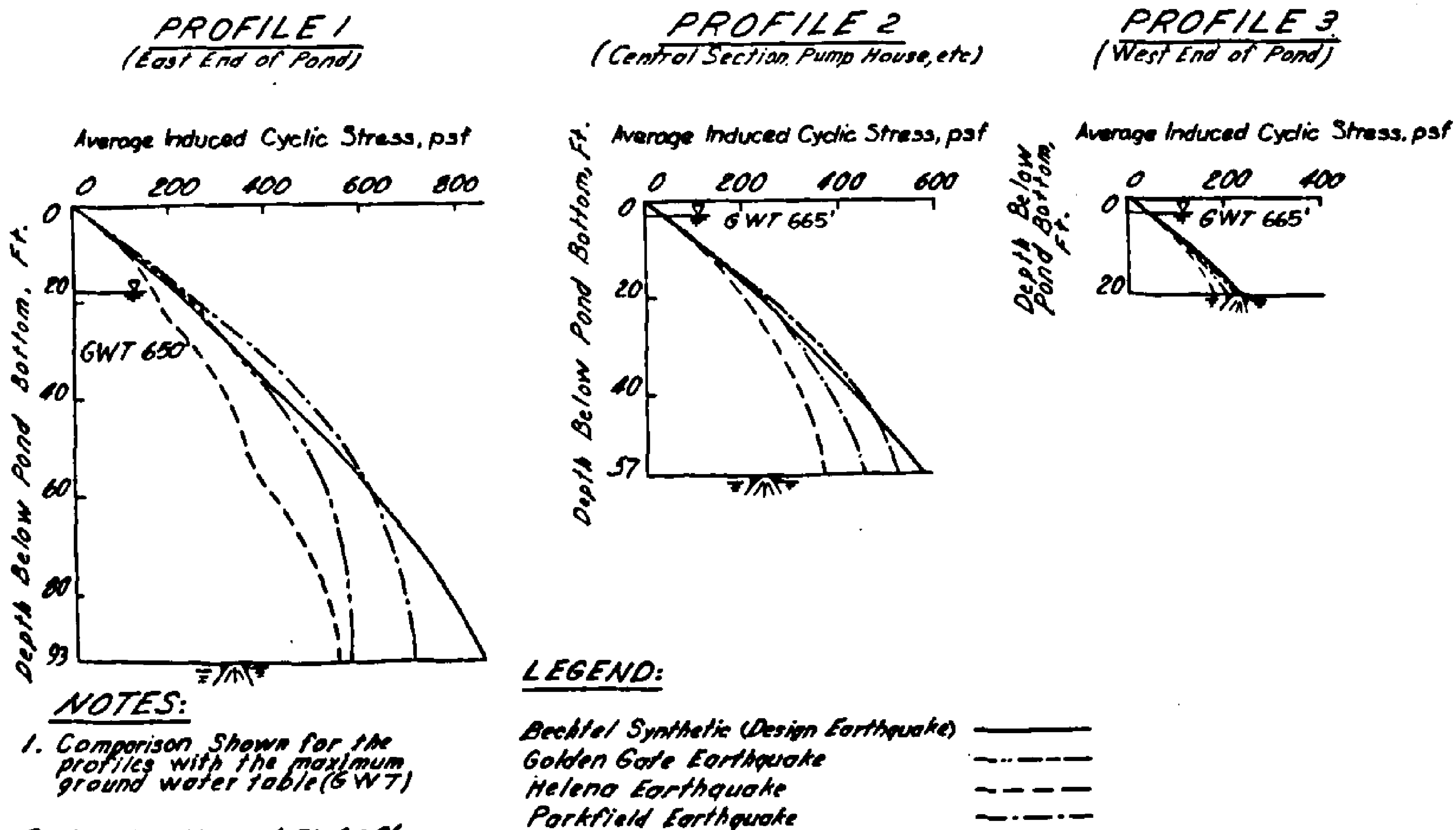
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND, COMPARISON OF FACTORS OF SAFETY FOR LIQUIFICATION FOR THE DESIGN EARTHQUAKE AND SOME REAL EARTHQUAKES

FIGURE 2.5-51, Rev 47

AutoCAD: Figure Fsar 2_5_51.dwg



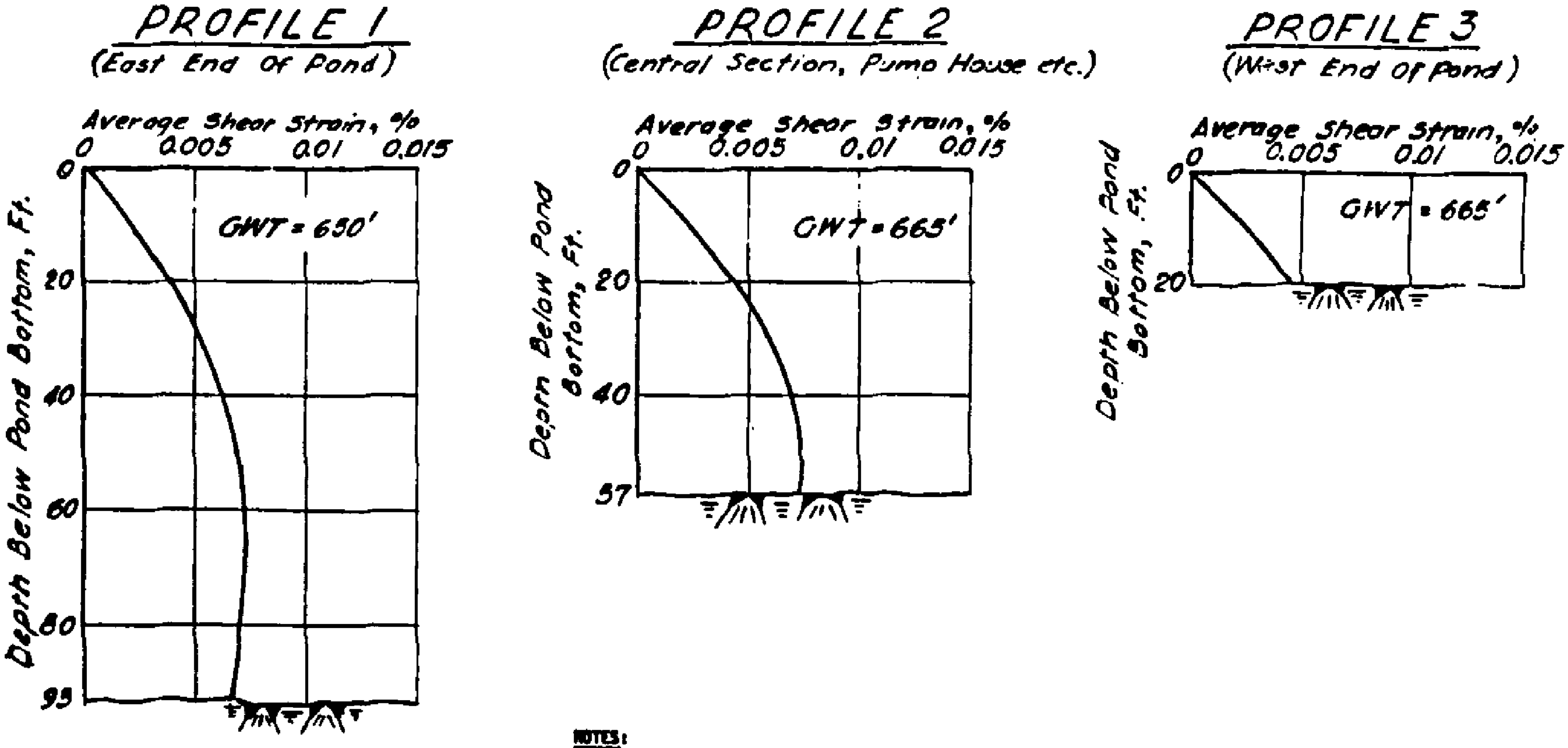
NOTES:

1. Comparison Shown for the profiles with the maximum ground water table (GWT)
2. Pond bottom at El. 668'

FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
SPRAY POND, COMPARISON OF AVERAGE CYCLE STRESS AS INDUCED BY DESIGN EARTHQUAKE AND SOME REAL EARTHQUAKES FIGURE 2.5-52, Rev 47

AutoCAD: Figure Fsar 2_5_52.dwg



NOTES:

1. Shear Strains Shown on the Drawings were induced by the SSE of 0.15g. The values were obtained from the SHAKE 3 Computer program.
2. Average Shear Strain = 0.65 Maximum Shear Strain.
3. GWT - Ground Water Table

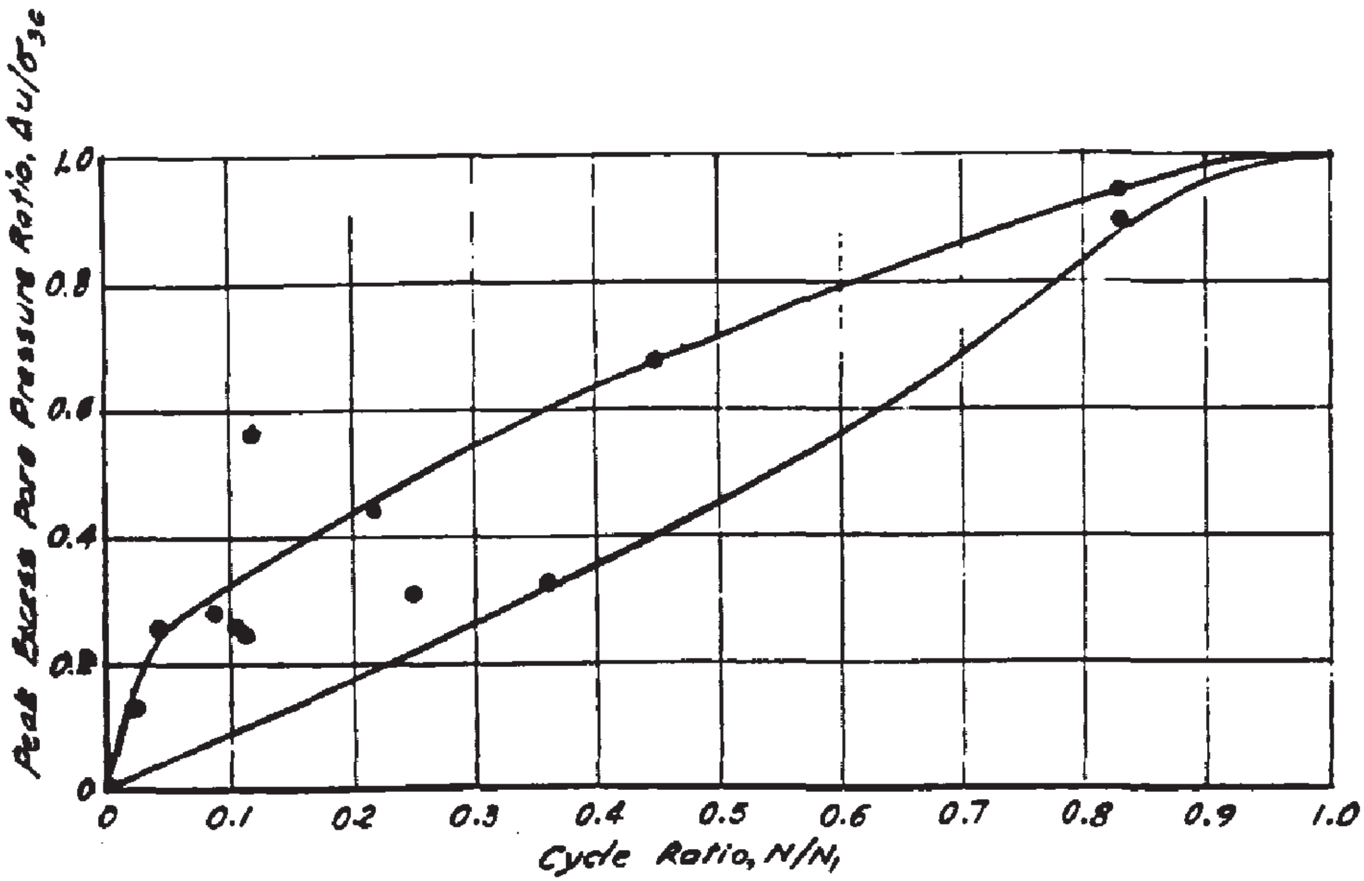
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND,
AVERAGE SHEAR VARIATION
WITH DEPTH

FIGURE 2.5-53, Rev 47

AutoCAD: Figure Fsar 2_5_53.dwg



NOTES:

1. This drawing was prepared following the procedure given in Lee, K. L., and Albaisa, A., "Earthquake Induced Settlements in Saturated Sands," Journal of the Geotechnical Engineering Div., ASCE, Vol. 100 No. GT4, pp 387-406, April 1974.

Where: N = the number of equivalent uniform cycles for design purposes (N = 5)

N_1 = the number of cycles required for the test sample to reach a total axial strain of 5%,

Δu = the induced excess pore pressure at the end of 5 cycles.

σ_{3c} = effective consolidation pressure.

2. The points shown represent the actual test data of 12 cyclic triaxial tests on undisturbed samples. (Reference - Geotechnical Engineers, Inc. Report, dated October 11, 1974.

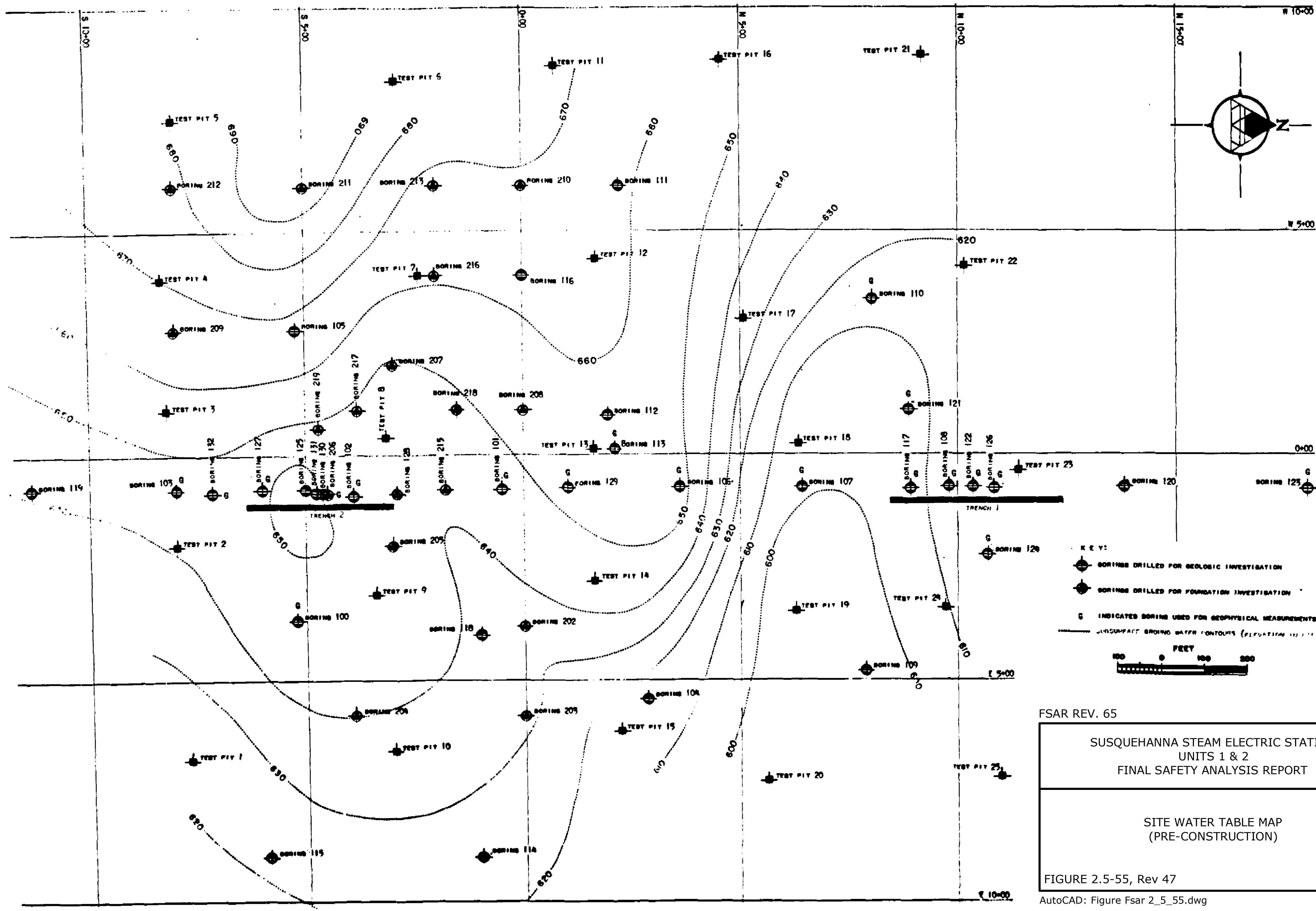
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND,
SUMMARY OF PORE
PRESSURE BUILDUP

FIGURE 2.5-54, Rev 47

AutoCAD: Figure Fsar 2_5_54.dwg



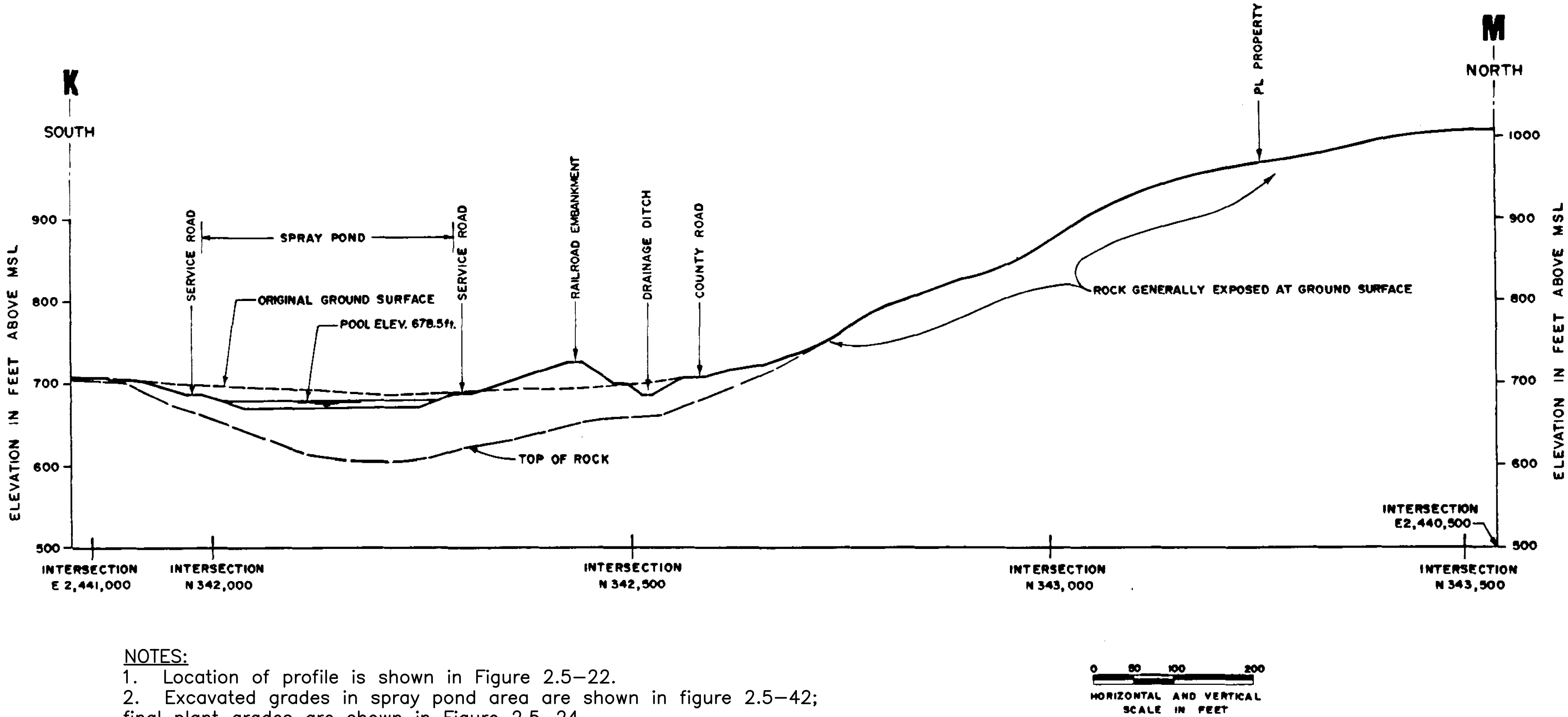
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SITE WATER TABLE MAP
(PRE-CONSTRUCTION)

FIGURE 2.5-55, Rev 47

AutoCAD: Figure Fsar 2_5_55.dwg



NOTES:
1. Location of profile is shown in Figure 2.5–22.
2. Excavated grades in spray pond area are shown in figure 2.5–42; final plant grades are shown in Figure 2.5–24.
3. Contours of slope north of county road are shown in Figure 2.1–2.

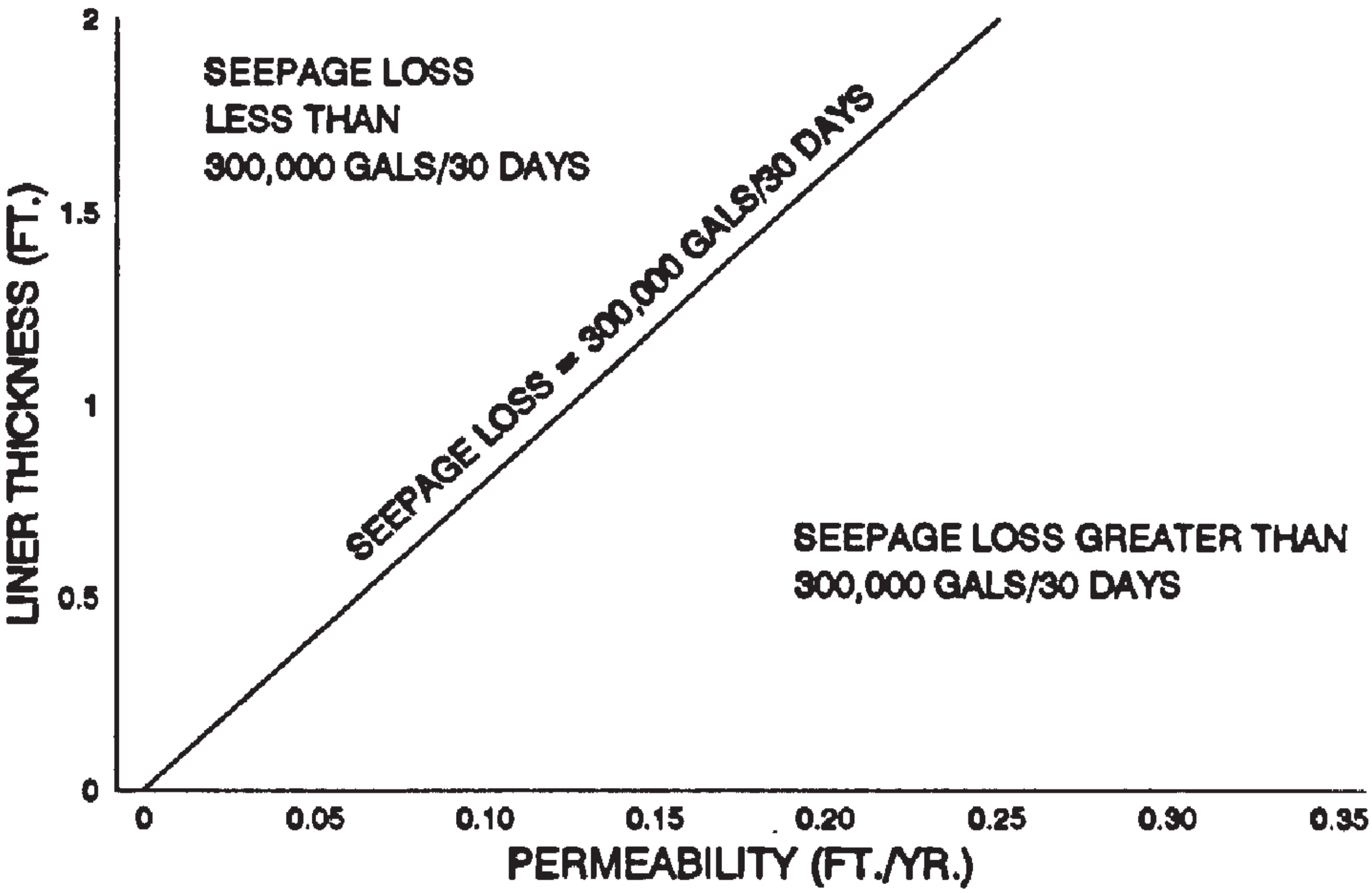
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

PROFILE OF SLOPE
NORTH OF SPRAY POND

FIGURE 2.5-56, Rev 47

AutoCAD: Figure Fsar 2_5_56.dwg



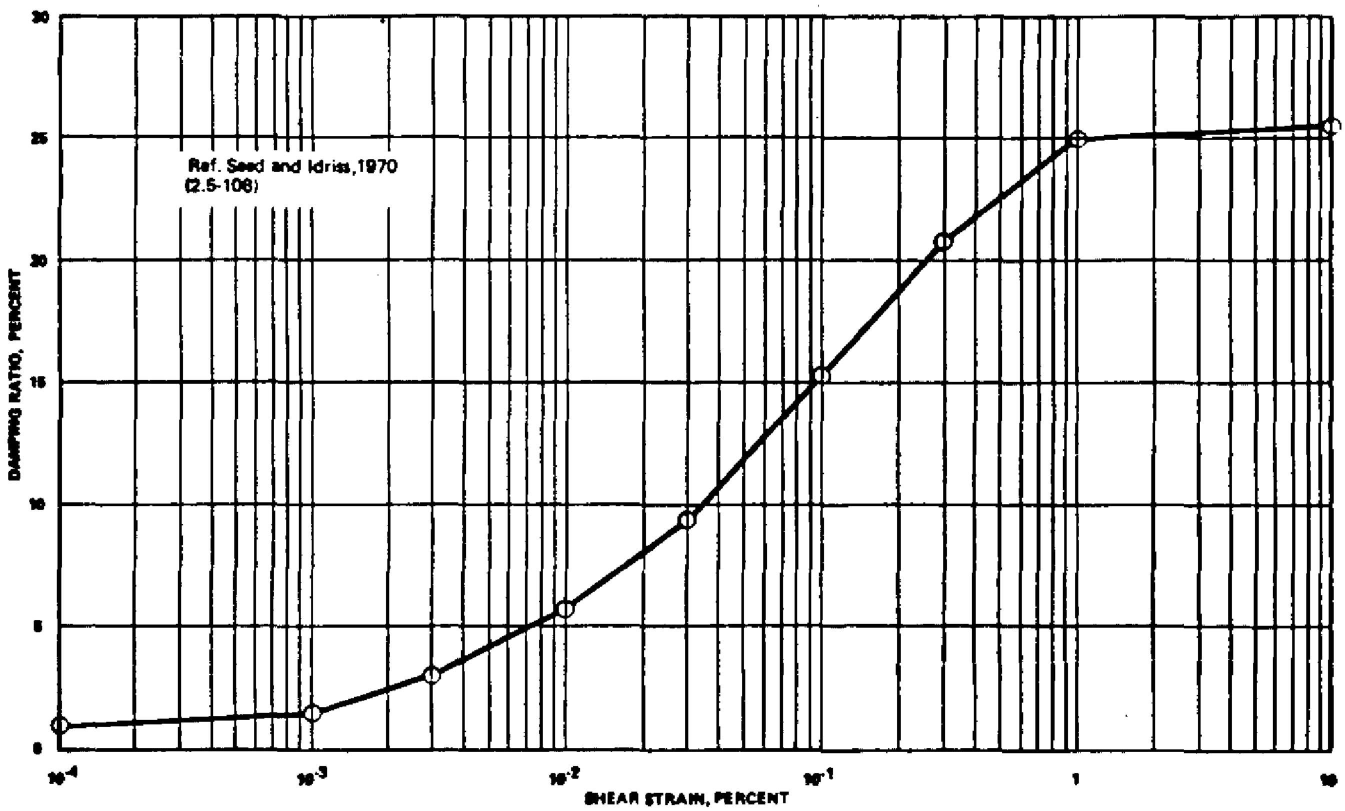
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

SPRAY POND,
RELATIONSHIP BETWEEN LINER THICKNESS,
LINER PERMEABILITY AND SEEPAGE LOSS

FIGURE 2.5-57, Rev 47

AutoCAD: Figure Fsar 2_5_57.dwg



FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

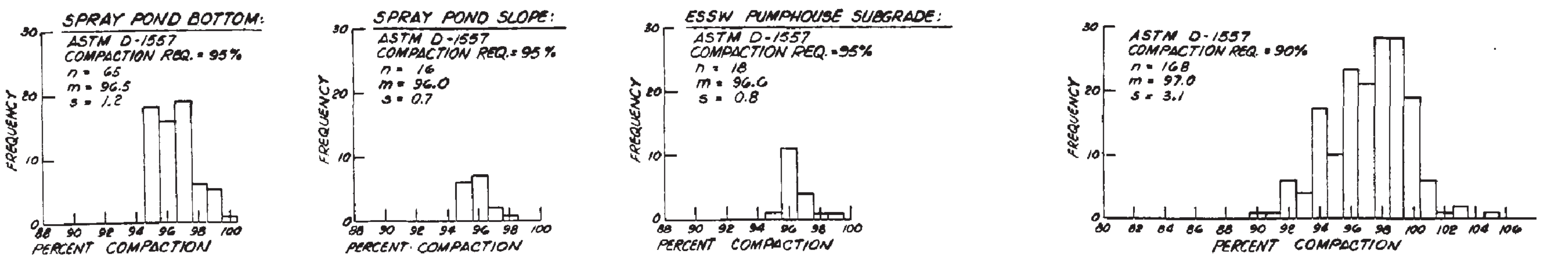
DAMPING RATION vs. SHEAR STRAIN
OF SAND IN
LIQUIFICATION ANALYSIS

FIGURE 2.5-58, Rev 47

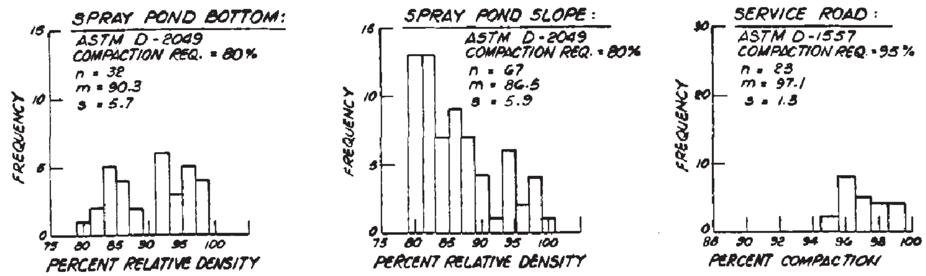
AutoCAD: Figure Fsar 2_5_58.dwg

Security-Related Information
Figure Withheld Under 10 CFR 2.390

SUSQUEHANNA STEAM ELECTRIC STATION UNITS 1 & 2 FINAL SAFETY ANALYSIS REPORT
SAMPLE LOCATIONS OF SOIL DENSITY TESTS FIGURE 2.5-59



FILL TO EAST AND SOUTH EAST OF ESSWP



HISTOGRAM FOR SUBGRADE PREPARATION

	COMPACTION CRITERIA		NUMBER OF TESTS	MEAN	STANDARD DEVIATION	(U.S. TESTING CO. REPORT NOS.)
	ASTM DESIGNATION	DENSITY REQUIREMENT				
SUBGRADE PREPARATION (REFERENCE: FSAR 2.5.4.5.2.2)						
SPRAY POND - BOTTOM	D-1557	95% OF $\delta_{d_{max}}$	65	96.5	1.2	25, 27-34, 38-40
BOTTOM	D-2049	80% RD	32	90.3	5.7	28, 34, 38-40
SLOPE	D-1557	95% OF $\delta_{d_{max}}$	16	96.0	0.7	6, 8, 11, 20, 38-40
SLOPE	D-2049	80% RD	67	86.5	5.9	6-9, 11-15, 17, 19-21, 38, 39
ESSW PUMPHOUSE	D-1557	95% OF $\delta_{d_{max}}$	18	96.6	0.8	2, 3
SERVICE ROAD AND THE SLOPES ABOVE ROAD	D-1557	95% OF $\delta_{d_{max}}$	23	97.1	1.3	1
SPILLWAY	D-1557	96% OF $\delta_{d_{max}}$	3	99.5	-	40
PIPELINE TRENCH	D-1557 D-2049	95% OF $\delta_{d_{max}}$ 80% RD	2 2	95.6 89.3	- -	16 38, 39
FILL (REFERENCE FSAR 2.5.4.5.3)						
FILL TYPE "B" TO SOUTH AND SOUTH EAST OF ESSWP	D-1557	90% OF $\delta_{d_{max}}$	168	97.0	3.1	41

TABULAR SUMMARY

NOTE:

1. FIELD DENSITY TESTS REPORTED IN U.S. TESTING CO. REPORT BUT NOT INCLUDED IN THIS STATISTICAL ANALYSIS:
   a.) IN-SITU SOIL DENSITY BEFORE COMPACTION AT ESSW PUMPHOUSE.
   b.) TESTS FOR THE EFFECTIVENESS OF COMPACTION EQUIPMENT ON SPRAY POND SLOPE.
   c.) FAILURE TESTS WHERE SOIL WAS FURTHER COMPACTED AND TESTED TO INDICATE A SATISFACTORY COMPACTION.
2. THIS DRAWING TO BE READ IN CONJUNCTION WITH FSAR QUESTION 362.18 AND SECTION 2.5.4.5.3

LEGEND

n = NUMBER OF TESTS
m = ARITHMETIC MEAN
s = STANDARD DEVIATION

FSAR REV. 65

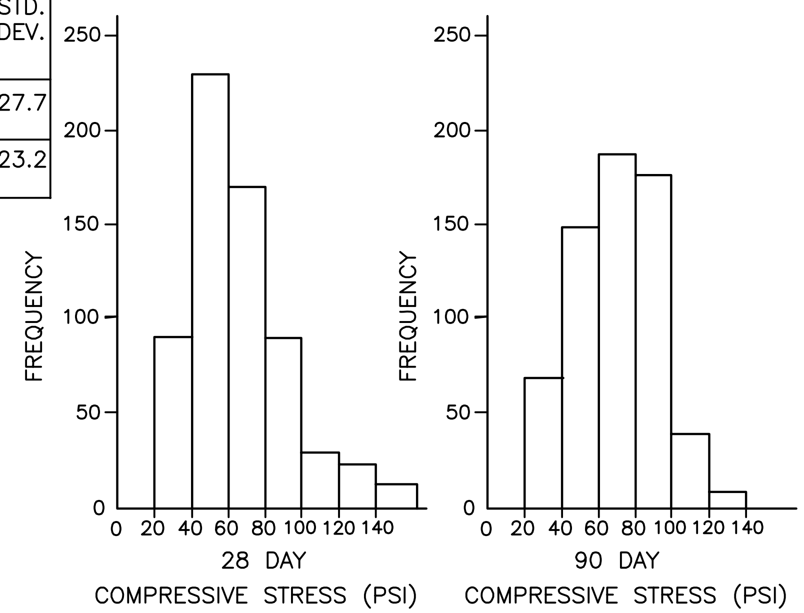
SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

STATISTICAL ANALYSIS OF
FIELD DENSITY TEST RESULTS

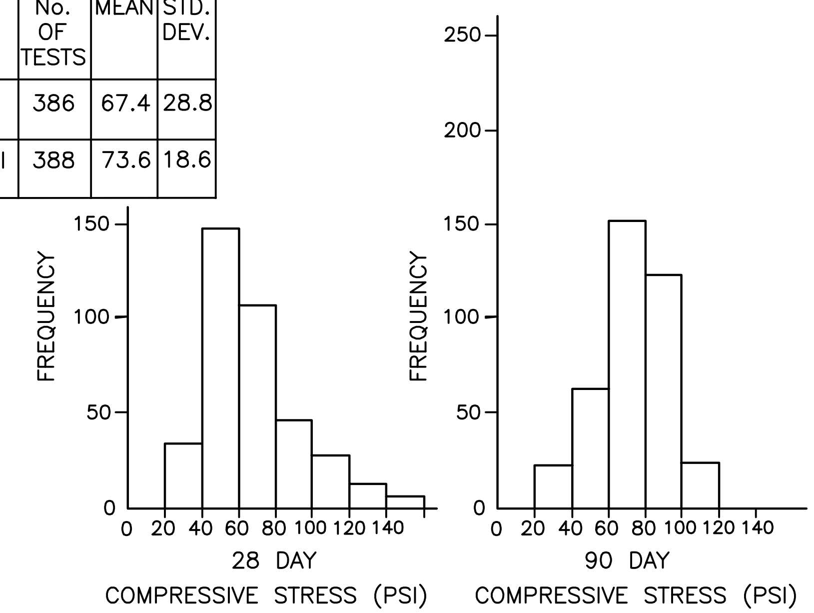
FIGURE 2.5-60, Rev 47

AutoCAD: Figure Fsar 2_5_60.dwg

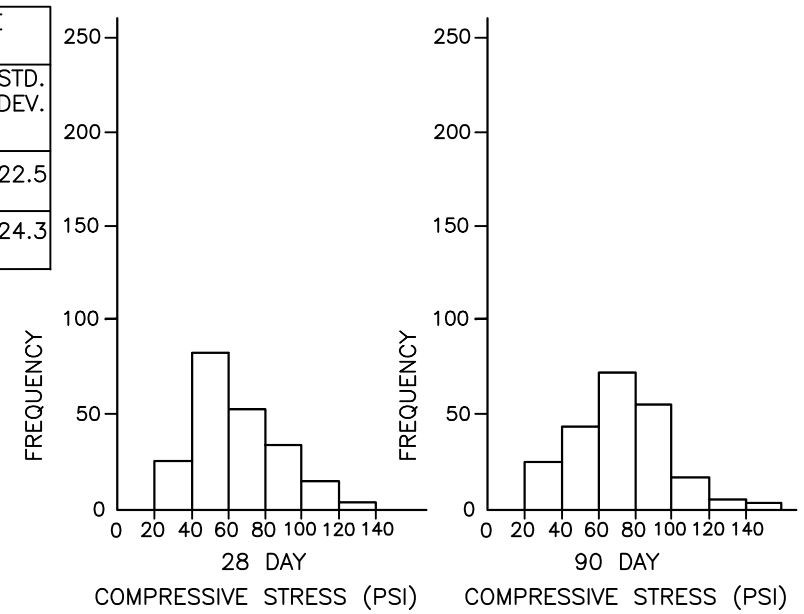
AREA EAST OF REACTOR BUILIDNG				
SPECIFICATION REQUIREMENT COMP. STRENGTH		No. OF TESTS	MEAN	STD. DEV.
28 DAY	40 PSI MIN.	637	66	27.7
90 DAY	100 PSI MAX.	635	71	23.2



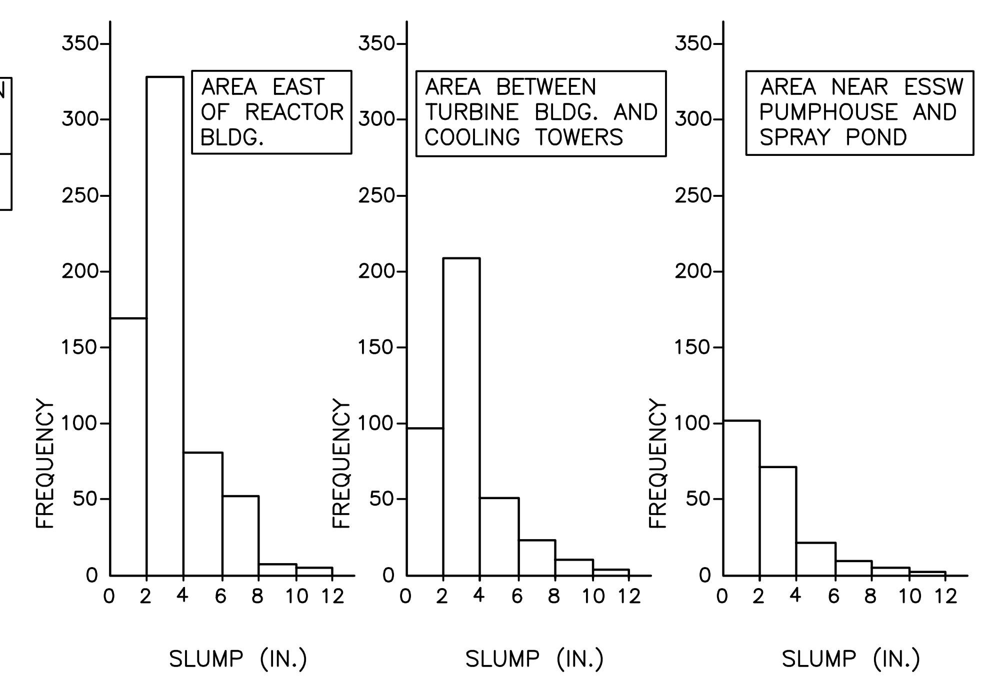
AREA BETWEEN TURBINE BUILDING AND COOLING TOWERS				
SPECIFICATION REQUIREMENT COMP. STRENGTH		No. OF TESTS	MEAN	STD. DEV.
28 DAY	40 PSI MIN.	386	67.4	28.8
90 DAY	100 PSI MAX.	388	73.6	18.6



AREA NEAR ESSW PUMPHOUSE AND SPRAY POND				
SPECIFICATION REQUIREMENT COMP. STRENGTH		No. OF TESTS	MEAN	STD. DEV.
28 DAY	40 PSI MIN.	212	64.9	22.5
90 DAY	100 PSI MAX.	212	71.8	24.3



SPECIFICATION REQUIREMENT FOR SLUMP
3 IN. MIN.
6 IN. MAX..



NOTES:

1. SLUMP - THE SLUMP REQUIREMENTS OF 3 INCH MINIMUM AND 6 INCH MAXIMUM WERE FOR REASONS OF WORKABILITY DURING FIELD POURING OPERATIONS AND AS ONE OF THE MEASUREMENTS TO ENSURE THAT A SUFFICIENT COMPRESSIVE STRENGTH WOULD BE OBTAINED. THEREFORE A DEVIATION FROM THE SPECIFICATION REQUIREMENT DOES NOT CONSTITUTE UNACCEPTABILITY SINCE THE ONLY BASIS WOULD BE AN ALTERED COMPRESSIVE STRENGTH AND THESE ITEMS HAVE THEIR OWN ACCEPTABILITY LIMITS.
2. 90 DAY COMPRESSIVE STRENGTH - THE REQUIREMENT FOR THE 100 PSI MAXIMUM STRENGTH WAS A CONVENIENCE ITEM ONLY. THE PURPOSE FOR THIS UPPER LIMIT WAS TO FACILITATE EASE OF EXCAVATION IN AREA PREVIOUSLY BACKFILLED WITH SAND-CEMENT-FLYASH MATERIAL. A DEVIATION FROM THE SPECIFICATION REQUIREMENT IN THIS AREA IN NO WAY LIMITS THE INTENDED USE OF THE MATERIAL.
3. 28 DAY COMPRESSIVE STRENGTH - THE REQUIREMENT FOR MINIMUM COMPRESSIVE STRENGTH WAS CHOSEN SUCH THAT THE COMPRESSIVE STRENGTH WOULD BE IN THE SAME RANGE AS THE BEARING VALUE OF COMPACTED GRANULAR BACKFILL. THE 40PSI STRENGTH CONVERTS TO 5,760 lb/ft² WHEREAS COMPACTED GRANULAR BACKFILL HAS BEARING VALUES ON THE ORDER OF [illegible] TO [illegible] lb/ft². HOWEVER ON SSES THE SAND-CEMENT-FLYASH MATERIAL HAS NOT BEEN USED FOR FILL BENEATH ANY STRUCTURE. THEREFORE THE HIGHER BEARING VALUE IS NOT REQUIRED. THE SAND-CEMENT-FLYASH BACKFILL HAS BEEN USED ONLY AS GENERAL BACKFILL FOR PIPES, CONDUITS, AND ALONG THE SIDES OF BUILDINGS. IT HAS BEEN DETERMINED THAT THE 20 PSI (2,880 lb/ft²) MINIMUM VALUE THAT MAY BE FOUND IN SOME AREAS IS COMPLETELY ADEQUATE TO SATISFY THE CONDITIONS FOR WHICH IT IS BEING USED.
4. THIS DRAWING TO BE READ IN CONJUNCTION WITH FSAR QUESTIONS [illegible] AND [illegible] SECTION [illegible]

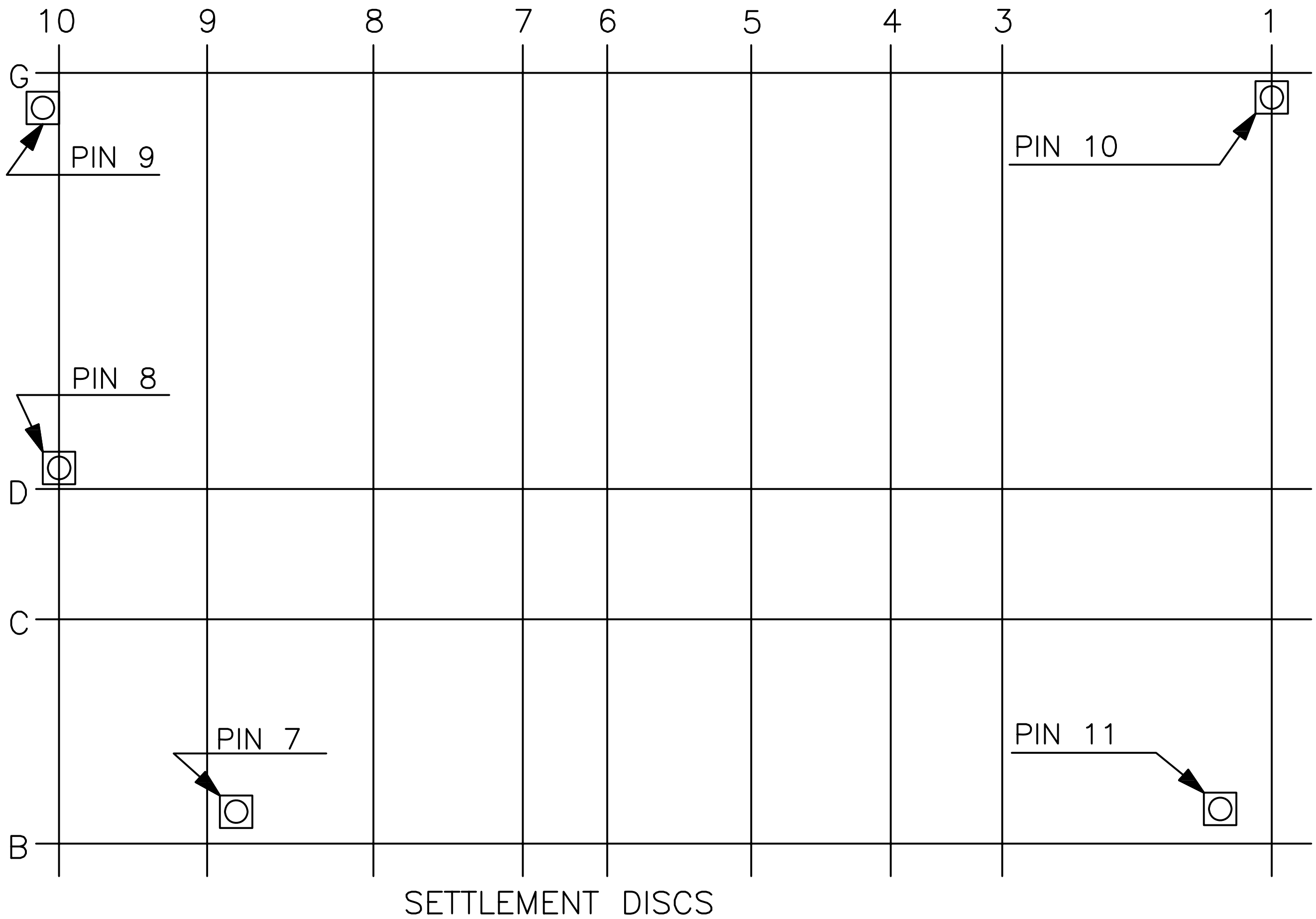
FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

STATISTICAL ANALYSIS OF
SAND-CEMENT-FLYASH
BEDDING & BACKFILL

FIGURE 2.5-61, Rev 47

AutoCAD: Figure Fsar 2_5_61.dwg



FSAR REV. 65

SUSQUEHANNA STEAM ELECTRIC STATION
UNITS 1 & 2
FINAL SAFETY ANALYSIS REPORT

ESSW PUMPHOUSE
FLOOR PLAN EL. 685'-6"

FIGURE 2.5-62, Rev 47

AutoCAD: Figure Fsar 2_5_62.dwg

APPENDIX 2.5A

U.S. DEPARTMENT OF THE INTERIOR REPORT ON INVESTIGATION OF AN EARTH DISTURBANCE AT WILKES-BARRE

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF MINES
REGION VIII

REPORT ON INVESTIGATION OF AN
EARTH DISTURBANCE AT WILKES-BARRE, LUZERNE COUNTY,
PENNSYLVANIA

February 21, 1954

By

Frank Retsel, Federal Coal-Mine Inspector, Wilkes-Barre, Pa.
W. T. Torrance, Federal Coal-Mine Inspector, Wilkes-Barre, Pa.
H. F. Weaver, Chief, Coal-Mine Inspection Branch, Washington, D.C.

Originating Office - Bureau of Mines
Room 223, Federal Building, Wilkes-Barre, Pa.
E. H. McCleary, Chief, Wilkes-Barre (Pa.) Office
Accident Prevention and Health Division

Background

About 3 p.m. on February 21, 1954, an earth disturbance started at Wilkes-Barre, Pennsylvania, and caused the upward buckling of sidewalks and roadways, the breaking of gas and water mains, and some damage to buildings in a residential portion of the city that eventually encompassed an area roughly two thousand feet square.

Inasmuch as the area involved was directly over mine workings of the Woodward colliery, Glen Alden Coal Company, personnel of the Accident Prevention and Health Division, Bureau of Mines, took a prompt and active part in the investigation of conditions therein--principally from the standpoint of safety.

The Investigation

Coal veins that had been first mined beneath the disturbed area were, starting at the top, the Abbott, Kidney, Hillman, Five Foot, and Lance, and they were overlaid with an alluvial deposit--known to many as the buried valley of the Susquehanna-

that ranged up to 165 feet in thickness. However, in the affected area, mining operations had ceased in the Abbott vein on January 14, 1954; mining in the Kidney vein consisted of driving chambers in the remining small portion of solid coal; mining in the Hillman vein was restricted to the recovery of top coal in 4 chambers and the driving of chambers in 2 blocks of coal; and first mining had long since been completed in the Five Foot and Lance veins. No second mining (pillar recovery) was done in any veins in the area involved.

Thorough inspection of the workings in the Abbott, Kidney, and Hillman veins in the affected area was delayed pending the removal of methane that had accumulated therein as a result of disrupted ventilating facilities. Reestablishment of ventilation permitted inspection of workings in the Kidney vein on February 27 and in the Abbott vein on March 13. Dangerous roof conditions in the Hillman vein have prevented persons from advancing far enough into the workings in the affected area to repair the ventilating facilities; consequently a thorough inspection of the Hillman workings was not possible at the time this report was prepared. Inspection of the Five Foot and Lance veins showed conditions therein to be normal.

Inspection of the Kidney vein workings on February 27 revealed the following significant information:

1. The vein in the affected area (known as the 38 slope section) was in the form of a trough, the main axis of which pointed east and west. The basin of the trough was beneath Lafayette Street, the crest of the trough on the south side was under and roughly paralleled Charles Street, and the crest of the trough on the north side was under and roughly paralleled Old River Road.

2. Recent spalling of coal off the pillar ribs was most pronounced near the crests of the trough on both the north and south sides.

3. Evidence of lateral movement of the strata was noted in the basin; 2" X 4" stringers installed to suspend trolley wire along gangways driven in the basin were generally bowed--some as much as 12 inches.

4. Generally, props and timber sets showed no evidence of weight, except that chambers No. 100 to No. 107, driven on the north side of the trough and the gangway (No. 573) between them, could not be inspected because of fresh, heavy roof falls and bad roof--evidence of a recent disturbance.

5. Accumulations of methane were found by means of a flame safety lamp at the faces of chambers No. 110 to No. 121 off gangway 573.

6. Mining was conducted properly, and due precautions were taken to assure adequate surface support.

Inspection of the Abbott vein workings on March 13 revealed the following significant information:

1. The Abbott vein in the affected area was in the shape of a trough, similar in shape and position to that in the Kidney vein described previously.

2. The faces of the chambers driven up the pitch on the south side of the basin of the trough had encountered solid rock, indicating a faulted area. Inspection of the maps of the mine workings showed evidence of a displacement fault in all the strata in the area--the main axis of which pointed approximately east and west--and extended through the Woodward and Lance colliery workings.

3. Only slight fresh spalling of coal off the pillars was observed in the Abbott vein, and generally the props and timbers showed no signs of weight. However, almost continuous fresh falls of roof and brows were observed along gangway No. 511, but, as stated before, little or no spalling of coal was seen.

4. Careful examination of the faces of the chambers that had encountered the rock wall at the crest of the trough on the south side showed evidence of downward slipping of the strata along the line of fault.

5. Careful examination of concrete stoppings in the affected area revealed that:

   (a) A very definite lateral movement of the strata from south to north along the south side of the trough and an upward thrust of the strata along the north side of the trough had occurred. The roof had moved recently as much as one inch away from the tops of some concrete stoppings (set parallel to the axis of the trough), and door frames set perpendicular to the axis of the trough showed evidence of lateral pressure. Concrete stoppings installed along the north side of the trough showed no indication of lateral movement of

the strata, but they all looked as though they had been struck from the bottom with a strong, sudden blow that caused the bottom portions thereof to be splintered.

6. Methane was not found by means of a flame safety lamp during this inspection of the Abbott vein.

7. Mining was conducted properly, and due precautions were taken to assure adequate surface support.

Cause of the Earth Disturbance

According to official reports made by certified fire bosses employed by the Glen Alden Coal Company, following their regular, routine examination of workings in the subsequently affected area, conditions were normal during such inspections on the morning of February 21, 1954.

The charts on the continuous pressure-recording gages at main fans at the following locations showed that an unusual and sudden movement of the recording needle had occurred around 11:30 a.m. on February 21, 1954:

		Degree of needle movement		
Colliery	Fan	Strong	Medium	Weak
Woodward	#1	X		
Woodward	#2	X		
Lance	Baltimore	X		
Lance	Red Ash	X		
Nottingham	#5			X
Nottingham	#6			X
Avondale				X
South Wilkes-Barre	#5		X	
Inman	#21			X
Loomis	#2			X
Loomis	#4			X
Loomis	#39			X
Hollenback				X
Huber	#5	X		
Huber	#8	X		

Sugar Notch	#5			X
Truesdale	#26			X
Truesdale	#1			X
Truesdale	#1 &#2			X
Truesdale	Askam		X	
Bliss	#1			X
Auchincloss	#3	X		

The chart of the cycle stormograph located at South Wilkes-Barre colliery showed an unusual and sudden movement of the recording needle at 11:30 a.m. on February 21, 1954.

This evidence, together with the upward buckling of surface sidewalks and roadways and the appearance of cracks in the ground in the disturbed area, seemed to point to the occurrence of an earth tremor. Upon further investigation of such a possibility, it was found that officials of the Seismology Branch, Geophysics Division, Coast and Geodetic Survey, Department of Commerce, Washington, D. C., had reported the occurrence of an earthquake at 11:20 a.m. on February 21, 1954. This "quake" was recorded on the seismographs at Fordham University and at Columbia University in New York City--but it was also recorded by many other seismographs located throughout the world, and, according to the official Federal report, the earthquake that was recorded on the seismographs at 11:20 a.m. on February 21, 1954, occurred definitely in the Aleutian Islands. No other evidence of earth tremors was recorded by official seismographs for the remainder of February 21, 1954.

Insofar as the seismograph records for February 21, 1954, are concerned, they did not indicate an earth tremor in the vicinity of Wilkes-Barre on that date. However, according to officials of the Seismology Branch, Department of Commerce, Washington, D. C., it is possible for an earth movement to occur at Wilkes-Barre and be too weak to be recorded on seismographs in New York City. Therefore, evidence of a localized earth movement was sought, and, we feel, was found.

Careful study of all the evidence gathered during the investigation of this earth disturbance caused the undersigned Federal investigators to deduce as follows:

1. Around 11:30 a.m. on February 21, 1954, a slight but sudden downward movement of the strata occurred along the displacement fault that exists in the strata beneath the south side of the affected area. We believe it was this sudden jolt that caused the aforementioned needles of the various gages to be moved out of normal position.

2. The powerful forces of the downward movement of the strata along the fault plane were pent up in the adjacent strata until they were suddenly released in an adjusting upward thrust of strata at the weakest point, which seemed to be along the crest of the trough on the north side directly beneath the area where the first buckling and cracking of the surface was observed.

3. Subsequent continued movement of the surface over a gradually extended area was caused by adjustment of the heavy alluvial deposits, which very likely were water logged and in a semi-fluid condition.

4. Whether downward movement of the strata along the fault plane caused failure of the coal pillars and strata under the affected area or whether failure of the coal pillars and strata under the affected area caused downward movement of the strata along the fault plane could not be definitely and positively determined.

Recommendation

Under the circumstances, we cannot offer any recommendation that may prevent recurrence of a similar earth movement.

S/ Frank Retzel
Federal Coal-Mine Inspector

S/ W. T. Torrance
Federal Coal-Mine Inspector

S/ H. F. Weaver
Chief, Coal-Mine Inspection Branch

APPENDIX 2.5B

CITY OF WILKES-BARRE
REPORT ON AN EARTH DISTURBANCE

This appendix consists of the conclusions and a discussion of the disturbances in the mined area by John C. MacCartney and F. Edgar Kudlich, Consulting Engineers, who were commissioned by the City of Wilkes-Barre, Pennsylvania to investigate the earth disturbances which occurred in the southerly section of that city on February 21 and 23, 1954.

CONCLUSIONS OF MAC CARTNEY AND KUDLICH

"Many theories have been propounded as to the cause of the surface disturbances, which exclude mining as the possible cause. These have been investigated and considered by Dr. Warren J. Mead, Consulting Geologist, and us and have been discarded as unacceptable.

Our conclusions with respect to the disturbances, supported by discussions with Dr. Mead, are as follows:

1. That downward movements occurred at certain points in the southerly section of Wilkes-Barre City on and after February 21, 1954. Reactions from the downward movements resulted in some upward movements, giving to the whole the appearance of an undulation. This combination of movements caused damage to improvements on and under the surface. The magnitude of these movements is not accurately measurable due to the normal lack of references prior to the date of the disturbances.

2. That the disturbances of the surface were caused by a vertical movement in the consolidated strata underlying the area. This movement was transmitted upward through the wash, but not necessarily vertically.

3. That the movement which had its inception in and directly above the Hillman vein would not have occurred had there been no mining.

4. That the Glen Alden Coal Company, in the extraction of coal from the Hillman vein, adopted accepted practices of mining with the intention of supporting the overlying strata, if for no other reason than to protect its mine from the hazard of inundation, but the execution of the accepted practices extended to the removal of top coal and, affected by the presence of faults and a basin, set up stresses which resulted in failure and subsequent movement."

Respectfully submitted,

(Signed) John C. MacCartney
Consulting Engineer

(Signed) F. Edgar Kudlich
Consulting Engineer

Wilkes-Barre, Pennsylvania

April 30, 1954.

(SEAL)

DISCUSSION OF DISTURBANCES IN MINED AREAS

"Our examinations of the mine workings indicated clearly that conditions in the uppermost five veins of coal that had been mined were abnormal, as evidenced by caving of the top rock and spalling of the pillars. The disturbed condition in the Five Foot and the Lance veins were resultants of movements in the upper veins rather than contributory causes to the disturbances of the surface.

Our references in this report to spalling of ribs and falls of top coal and rock are confined to recent occurrences except where specifically noted to the contrary. Naturally, we had no occasion to examine these mines prior to the February 21, 1954, disturbances. We judged the recency of the spalling and falls of top by the freshness of the exposed surfaces, which opinion was confirmed in many instances by reference to the inspection reports of those company employees whose duty it is to make daily or periodical examinations of active and abandoned workings as required by law. Furthermore, we were accompanied on our tours by individuals who had made inspections of many of the areas between February 21, 1954, and the times of our examinations and who pointed out locations where disturbances had taken place during the intervening period.

In order to portray graphically the conditions encountered in the various veins, transparent overlays were made for each, showing the areas examined and the conditions found. When superimposed in the geologic order of the occurrence of the veins, the overlays revealed a series of unusual coincidences in respect to the disturbances observed in the mines.

For example, the heavy cave previously referred to in the Hillman vein at the foot of No. 34 Slope was found to be directly beneath the heavy falls of top rock beyond which we could not penetrate in Roads 573 North and 573 South in the Kidney vein which lies 80 feet above the Hillman. Approximately at the same point and about 180 feet above the Hillman was the heaviest fall of top rock encountered and crossed in the Abbott vein in Road 511 South. This was also the point where the two ventilation walls were cracked in the Abbott.

Conversely, where the pillars in the Hillman Slope were spalled but the top coal and top rock remained intact there was spalling in the underlying Five Foot Vein.

Again, we were unable to penetrate the area in the Hillman vein lying between the Lance-Woodward barrier pillar and No. 34 Slope because of heavy falls. Although we also were unable to examine the overlying Kidney vein where worked in this vicinity, it was reported to us by both Federal and State Mine

Inspectors that there were breaks in the bottom rock of the Kidney vein and that one portion of the Kidney near the Hillman No. 34 Slope was completely caved. Our own inspection of the Abbott vein in this same area revealed heavy spalling and some caving of top rock and a crack in the roof which was admitting water at the time of our visit.

We were stopped in the Hillman vein at the perimeter of the easterly surface disturbance by heavy falls of top coal and rock. In the Kidney vein above, just west of our point of penetration in the Hillman, we found spalling of the ribs and top falls. In the Abbott vein above and in the general line of the disturbances in the Hillman, there were spalling of the ribs and falls of top.

We noted also that near the center of the eastern disturbances, about at Academy Street, there were heavy spalling of the ribs and falls of top in the Kidney vein and lesser spalling and falls of top in the Abbott. The Hillman in this area was impenetrable.

In the Kidney vein, under the Lafayette Gardens area, there were spalled ribs and one top fall and a reputedly caved area which we could not examine, and spalling and heavy top falls in the overlying Abbott. The Hillman vein workings here also were impenetrable.

From the foregoing discussion it is evident that we were unable to examine the greater portion of the Hillman vein under the disturbed area. However, in several instances where we found caving and spalling in the Hillman vein we were able to observe similar conditions in the overlying veins. In other instances, where we were able to observe the overlying veins, we were unable to examine the Hillman vein. A comparison of the conditions at the latter points in the overlying veins with those where we were able to examine all three leads to a proper assumption that the conditions in the Hillman where not penetrated were just as bad as those observed where penetrated.

By measuring the mine workings as portrayed by the maps, we have found the normal extraction of coal in the Abbott vein had allowed 46.65% of the original vein area to remain as pillars, even after reserve pillars had been chambered; 49.49% remained in the Kidney vein and 51.35% in the Hillman vein.

No more than a cursory investigation was made of the size and strength of the pillars system in the Abbott and Kidney veins as the height of either vein, as well as the burden upon the vein, was less than the height of and burden upon the Hillman vein.

A computational investigation of the strength of the pillar system in the Hillman disclosed that the weight on the pillars, before the taking of the top coal, was slightly below the minimum calculated strength of the pillar under normal conditions.

By reference to the stratigraphic plates, it will be noted that the Hillman vein carries a bench of top coal which in ordinary practice is not mined during the first extractive operation unless it is structurally so weak that it fails during the first mining. It has been found that the top bench of coal is more stable than the immediate roof of the vein and therefore reduces the cost of maintaining the mine openings.

In the greater part of this area the operator has removed the top coal prior to the abandonment of the area, and accepted practice in the region, thereby increasing the height of the original pillars.

The ability of a coal pillar to support overlying strata is affected by the height of the vein. As a pillar increases in height its ability to support decreases. Consequently, the removal of the top coal of the Hillman vein tends to weaken the pillar system.

In this instance, the increased height of the pillars and the consequent weakening of the pillar system creates a condition which is further compounded by the presence of a series of faults. The geological definition of a fault is "a break in the continuity of a body of rock." Here, however, we use the word in the generally accepted application in the Anthracite district, e.g., an abrupt change in the plane of either the top or bottom rock, or of both, accompanied by a change in the thickness of the vein. The word also is commonly applied to structural irregularities in the rock above or below the vein. Where these faults occur the rock and vein strata are weaker than normal.

In addition to the removal of the top coal and the presence of faults in this area, there is a third condition which tends to rob the pillars of their normal efficiency and that is the occurrence of a syncline or basin centered beneath the disturbed area. The decrease in pillar efficiency is brought about by the fact that the force of gravity is not perpendicular to the plane of the vein.

In spite of the presence of faults and a basin, the strata were in equilibrium before mining. Had it not been for mining, the strata would have remained in equilibrium. It is a fact that the creation of voids, regardless of extent, sets up stresses in the surrounding strata. When the stresses created exceed the strength of the strata, failure must occur and it follows

that the equilibrium is disturbed. Movement and redistribution of the stresses follow until equilibrium is re-established.

Many theories have been advanced as to the causes of the surface disturbances. No matter how vague or tenuous these theories may have been, we have investigated them and brought them to the attention of Dr. Warren J. Mead and discussed them with him.

Dr. Mead, former head of the Department of Geology, Massachusetts Institute of Technology, and now Professor Emeritus of the same department, is an internationally known geologist and a consultant in economic engineering and geology. Dr. Mead made a personal inspection of surface of the disturbed area, has reviewed carefully the reports of our findings in the mines, has examined the mine maps and cross sections of the Glen Alden Coal Company and has studied the overlays and other exhibits prepared by us."

APPENDIX 2.5C

BORING LOGS

2.5C.1 Summary of Field Density Test Results

This summary is divided into four parts as follows:

1) Introduction
2) Specification Requirement
3) Explanation of Results
4) Conclusion

1) INTRODUCTION

The terminology used by U.S. Testing to identify the soils does not agree exactly with specification wording and the wording used in Subsection 2.5.4. Further, there are some inconsistencies on the "U.S. Testing Co." reports regarding elevations and/or coordinates. This condition is clarified later in this summary.

As explained in Subsection 2.5.4 the basic intent of the soil testing was to check the recompaction of the natural soil beneath the concrete liner and check the compaction of all soil fill used.

2) SPECIFICATION REQUIREMENT

The specification governing the work in the spray pond and vicinity addressed two types of soil fill (as well as recompaction of the existing soil under the concrete liner addressed in Section 2.5.4.5.2.2), classified as follows:

a) USAGE:

Fill Type A

This will be used for backfill behind concrete retaining walls, concrete cut-off walls and backfill where any make-up is required in soil beneath the concrete liner, spillway or service road.

Fill Type B

This will be used for embankment fill in the immediate vicinity of the spray-pond and ESSW Pumphouse.

b) MATERIAL:

General

All fill shall be well graded, sound, dense, durable material. It shall not contain any top soil, roots, brush, logs, trash or waste material, ice or snow.

Fill Type A

The maximum size of this material shall be 4 inches and no more than 5 percent by dry weight shall pass the No. 200 sieve.

Fill Type B

The maximum size of this material shall be 12 inches and no more than 35 percent by dry weight shall pass the No. 200 sieve.

c) PLACEMENT:

Material shall be placed in uniform horizontal layers so that when compacted it is free from lenses, pockets and layers of material differing substantially in grading from surrounding material. Fill shall not be placed on frozen ground. Placing of fill for which moisture conditioning is required shall be suspended whenever the ambient temperature reaches 34 degrees Fahrenheit and is falling.

The finished grade of all fills shall be within $\pm$ 2 inches of elevation specified or shown.

Fill Type A

Fill Type A shall be placed in a 6 inch maximum uncompacted layer thickness, moisture conditioned to obtain the required compaction, and compacted to at least 80 percent relative density as determined by ASTM D2049.

Fill material placed within 2 feet of structures and in areas where large construction equipment cannot be used shall be compacted to the specified density by hand operated equipment.

Fill Type B

Fill Type B shall be placed in a 15 inch maximum uncompacted layer thickness, moisture conditioned to obtain the required compaction, and compacted to satisfy both of the following requirements:

a) At least 80% relative density as determined by ASTM D2049 for material having not more than 12% passing the No. 200 sieve or 90% of maximum dry density as determined by ASTM D1557 for all other material.

b) Irrespective of the compacting effort to satisfy part a) above, the fill shall be compacted in one of the following manners as a minimum effort:

   i) Using a crawler tractor having a weight at least equal to that of a D8 Caterpillar tractor with bulldozer blade. Each track shall overlap the preceding track by not less than four inches. When the tractor has made one entire coverage of an area in this manner, it will be considered to have made one pass. Each fill lift shall be compacted with four passes.

   ii) Using a vibratory roller of minimum weight 20,000 pounds having a roller width of approximately 78 inches and a diameter of approximately 60 inches. The roller shall have a vibrator frequency range of between 1100 and 1600 vibrations per minute and have a minimum vibratory dynamic force of 40,000 pounds. The roller speed shall not exceed 3 mph and each track shall overlap the preceding one by at least 4 inches. When the roller has made one entire coverage of an area in this manner, it shall be considered to have made one pass. Each fill lift shall be compacted with four complete passes.

   iii) Using a hand controlled vibratory compactor in locations inaccessible by tractor or vibratory compactors will be

on the basis of the demonstrated ability of the compactor to compact the material to the same density as the contiguous backfill.

d) TESTING

The Subcontractor shall engage the services of an independent soils laboratory to carry out the testing specified herein. Reports of results shall be made available to the Contractor immediately they are available, but not more than 7 working days following completion of the test. Whenever doubt exists as to whether the specified degree of compaction has been attained, density and compaction tests shall be promptly arranged by the Contractor to verify conformance with this specification. The Subcontractor will be responsible for the cost of this testing.

The Subcontractor shall be held responsible to remove and recompact, at his own expense, any fill that fails to meet the specified degree of compaction.

The in-situ density of Fill Types A and B shall be determined in accordance with ASTM D1556 and performed at a frequency of at least one test per lift and every 1000 square feet on plan for Fill Type A and every 10,000 square feet on plan for Fill Type B.

Tests in accordance with ASTM D2049 or ASTM D1557, as appropriate, shall be carried out on the same material extracted for the ASTM D1556 test. The frequency of this testing shall be carried out at least twice in each 8 hours during placing operations.

Gradation tests in accordance with ASTM D422 shall be carried out at least twice in each 8 hours during placing operations.

3) <u>EXPLANATION OF RESULTS</u>

Generally, limits of excavation, fill and backfill for Seismic Category I structures are shown on Figure 2.5-37. However, the test data reports furnished by the "U.S. Testing Company"

and included in the appendix are applicable to the spray pond area only (sand-cement-flyash mixture was used elsewhere). In order to clarify the locations of the samples identified in the "U.S. Testing Company" Reports, a new Figure 2.5-59 has been prepared showing wherever possible the distribution within the spray pond. Some traceability has been lost and not every test is plotted on this figure. It should be noted that the south west section of the pond was excavated in rock; soil sampling was therefore not applicable. Fill Type B was placed predominantly in the area to the south and south-east of the ESSW Pumphouse. However, there is a good distribution of tests overall for the soil areas; the results are discussed more closely under a following section dealing with statistical analysis. The correlation testing was carried out at a lesser frequency than specified. The good consistency of the material used, as indicated by the results of the compaction tests, permitted this to be acceptable.

In some cases, the test reports show that "failures" occurred. Whenever this was the case, the soil was recompacted until satisfactory compaction was achieved. Accordingly, amended test reports #'s 7, 12, 17, 18, 20, 23, 32 and 33 are submitted. A copy of the SDDR (Supplier #46, Bechtel #52) is also submitted. A summary of the amendments is as follows:

a) U.S. Testing Company Report No. 7 dated September 27, 1977 shows Test No. 1 failed with a relative density of 63.3% on August 29, 1977. After recompaction, Test Nos. 2, 3 and 4 were retested on August 30, 1979, for this same area, as per the field notes dated August 29, 1977, and August 30, 1977.

b) U.S. Testing Company Report No. 12 dated October 4, 1978 shows Test No. 1 STA. 0 +90 failed with a relative density of 74.0%. Test No. 2 STA. 0 + 110 and No. 3 STA. 1 + 130 were retaken for this location with relative density results of 94.4% and 105.4% respectively, as per the field notes dated August 19, 1977, to August 24, 1977.

c) U.S. Testing Company Report No. 17 dated October 5, 1978 shows Test No. 3 was retaken (Relative Density 82.4%) because of failure of Test No. 2 (Relative Density 13.1%). See field notes dated September 12, 1977.

d) U.S. Testing Company Report No. 18 dated October 5, 1977 shows Test No. 1 (Relative Density 65.7%) and Test No. 2 (Relative Density 39.8%) taken on September 15, 1977, and Test No. 3 (Relative Density 74.0%) taken September 19, 1977, failed. Retest on September 30, 1977, shows satisfactory results, as per field notes dated September 15, 1977, September 19, 1977, and September 30, 1977.

e) U.S. Testing Company Report No. 20 dated October 6, 1977 shows Test No. 2 (Relative Density 13.6%) was an error in calculation. Should be 82.6%.

Test No. 4 (Relative Density 32.1%) was retaken after a recompaction and shown in Test No. 5 (Relative Density 83.4%). See field notes dated September 21, 1977.

f) U.S. Testing Company Report No. 22 dated October 7, 1977 shows retest done August 25, 1978, and reported on Test Report No. 43 dated September 7, 1978. See field notes dated September 15, 1977, and August 23, 1978.

g) U.S. Testing Company Report No. 23 dated October 7, 1977 shows retest taken September 21, 1977.

Note: Identification sheet attached to original report incorrectly shows S.16. Actual tests were done on S.14.

See field notes dated September 19, 1977, attached to Report No. 18 and dated September 21, 1977.

h) U.S. Testing Company Report No. 32 dated December 7, 1977 shows retest taken September 15, 1977, and reported on U.S. Testing Company Report No. 33. See field notes dated November 14, 1977, and November 15, 1977.

Statistical Distribution of Field Density Test Results

The field density test results given in the test reports were grouped for statistical study according to the test location and the compaction category (subgrade preparation or fill/backfill). The subgrade preparation for the spray pond liner was further divided into two groups -- pond bottom and pond slope.

The mean and standard deviation were computed for each group of data. Where additional compaction and density tests were made at the location where an initial test indicated too low a density, then only the final density result is included in the statistical study.

The results of the statistical distribution of percent compaction and relative density for all groups of data are summarized in Figure 2.5-60. For each group of data, the frequency histogram, number of tests, mean, standard deviation, and compaction criterion are shown. However, for small groups when the number of tests is only 2 or 3, only the mean is given.

Summary of Statistical Results

The statistical evaluation indicated the following:

a) The distribution of relative compaction and relative density shown in the frequency histograms are generally skewed distributions with a concentration of test results close to the minimum compaction required by the specification.

b) The exceptions to the above are exhibited in two groups of data:

   i) Subgrade Preparation for spray pond liner at the pond bottom, when 80 percent relative density was required.

   ii) Fill to the south and southeast of the ESSW Pump house.

Both sets of data show a mean much higher than the required compaction. The mean of the first group is 90.3 with minimum requirement of 80.0 (relative density). The mean of the second group is 97.0 with minimum requirement of 90.0 (percent of maximum dry density).

c) Although standard deviation was computed as a matter of interest, it is not significant in this case because the specification <u>minimum</u> requirements were met.

4) <u>CONCLUSION</u>

The test results and statistical analysis demonstrate that the specification compaction requirement was met. The spray pond was constructed upon a sound foundation and will adequately serve its intended function.

An amendment is made to Subsections 2.5.4.5.2.2 and 2.5.4.5.3 to address the "filled" area to the south and south-east of the ESSW pumphouse, since some of the "U.S. Testing Co." reports cover the compaction of this area. Amended test reports are also included in this appendix.

2.5C.2 Listings of Boring Logs

Item	Sheets
Key to Soil Boring Log Test Abbreviations For Borings 1101 through 1128	1
General Notes for Borings B1 through B-11	1
Boring Log 1101	1
Boring Log 1102	1
Boring Log 1103	1
Boring Log 1104	2
Boring Log 1105	1
Boring Log 1106	2
Boring Log 1106-A	1
Boring Log 1107	2
Boring Log 1107-A	1
Boring Log 1108	1
Boring Log 1109	1
Boring Log 1110	2
Boring Log 1110-A	1
Boring Log 1111	2
Boring Log 1112	2
Boring Log 1112-A	1
Boring Log 1113	2
Boring Log 1113-A	1
Boring Log 1114	2
Boring Log 1115	2
Boring Log 1115-A	1

Item	Sheets
Boring Log 1116	2
Boring Log 1117	1
Boring Log 1120	2
Boring Log 1122	1
Boring Log 1123	1
Boring Log 1124	1
Boring Log 1125	1
Boring Log 1126	2
Boring Log 1127	2
Boring Log 1128	2
Boring Log B1	1
Boring Log B2	2
Boring Log B3	1
Boring Log B4	1
Boring Log B5	2
Boring log B6	1
Boring Log B7	1
Boring Log B8	2
Boring Log B9	1
Boring Log B10	1
Boring Log B11	1
Boring Log 1	2
Boring Log 2	2
Boring Log 3	2

Item	Sheets
Boring Log 4	2
Boring Log 5	2
Boring Log 6	2
Boring Log 7	2
Seismic Velocity and Elastic Modul Measurements, Spray Pond, by Weston Geophysical Engineers, Inc.	11

NOTE: Holes 1118, 1119, and 1121 were planned but not drilled due to the bedrock out-cropping at borings 1118 and 1121, and heavy rock fill at boring 1119.

KEY TO SOIL BORING LOG TEST ABBREVIATIONS

FOR BORINGS 1101 THROUGH 1128

CR - Cyclic Consolidated - Undrained Triaxial Test

S - Consolidated - Drained Triaxial Test

Gs - Specific Gravity Determination

Grain Size - Grain Size Determination

GENERAL NOTES FOR BORINGS B1 THROUGH B11

1950 Chicago Building Code Soil Classifications are Used Except Where Noted

DRILLING & SAMPLING SYMBOLS

SS : Split-Spoon - 1-3/8" I.D., 2" O.D. except where noted
ST : Shelby Tube - 2" O.D. except where noted
PA : Power Auger Sample
DB : Diamond Bit - NX: BX: AX:
CB : Carboloy Bit-NX: BX: AX:
OS : Osterberg Sampler - 3" Shelby Tube
HS : Housel Sampler
WS : Wash Sample
FT : Fish Tail
RB : Rock Bit
WO : Wash Out

Standard "N" Penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2 inch OD split spoon, except where noted.

WATER LEVEL MEASUREMENT SYMBOLS

WL : Water Level
WCI : Wet Cave In
DCI : Dry Cave In
WS : While Sampling
WD : While Drilling
BCR : Before Casing Removal
ACR : After Casing Removal
AB : After Boring

Water levels indicated on the boring logs are the levels measured in the boring at the times indicated. In previous soils, the indicated elevations are considered reliable ground water levels. In impervious soils, the accurate determination of ground water elevations is not possible in even several days observation, and additional evidence on ground water elevations must be sought.

CLASSIFICATION

COHESIVE SOILS

If clay content is sufficient so that clay dominates soil properties, then clay becomes the principles noun with the other major soil constituent as modifier; i.e., silty clay. Other minor soil constituents may be added according to classification breakdown for cohesionless soils; i.e., silty clay, trace to some sand, trace gravel.

Soft	:	0.00 - 0.59 tons/ft^2
Stiff	:	0.60 - 0.99 tons/ft^2
Tough	:	1.00 - 1.99 tons/ft^2
Very tough	:	2.00 - 3.99 tons/ft^2
Hard	:	4.00 tons/ft^2

COHESIONLESS SOILS

"Trace"	:	1% to 10%	
"Trace to some"	:	10% to 20%	
"Some	:	20% to 35%	
"And"	:	35% to 50%	
Loose	:	0 to 9 Blows	
Medium Dense	:	10 to 29 Blows	or
Dense	:	30 to 59 Blows	equivalent
Very Dense	:	60 Blows	