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Chapter 2

SITE CHARACTERISTICS

This section provides information on the DAEC site and environs and summarizes site evaluation efforts performed. This information was used to demonstrate the acceptability of the DAEC location. Information related to Emergency Planning will be updated as required per 10CFR50 Appendix E.

2.1 GEOGRAPHY AND DEMOGRAPHY

2.1.1 SITE LOCATION AND DESCRIPTION

2.1.1.1 Specification of Location

The DAEC site is located on the western side of a north-south reach of the Cedar River, approximately 2.5 miles north-northeast of the Village of Palo, Iowa, in Linn County (T-84N, R-8W, Sections 9 and 10). The closest city is Cedar Rapids with its outer boundary being 8 miles to the southeast.

The site is approximately 500 acres on a flat strip of land running northeast and parallel to the Cedar River. [redacted] Site boundaries are shown in Figures 2.1-1 and 2.1-2. The site location is shown in Figure 2.1-3.

[redacted]

2.1.1.2 Site Area Map

A paved county highway provides access to the site. Figures 2.1-2 and 2.1-3 indicate the access route to the site.

The topography of the site varies as indicated in Figure 2.1-2. A relatively flat plain at approximate elevation 750 ft above mean sea level (msl) extends from the site toward the Village of Palo on the southwest, and most of this land is now being farmed. At Palo, the elevation is 747 to 750 ft.

Across the river from the site, the land rises from an elevation of 750 ft to an elevation of about 900 ft within a horizontal distance of approximately 2000 ft. These slopes are rather heavily wooded with only an occasional field or pasture dotting the landscape. Beyond this rise, the land is gently rolling farmland.

To the northwest, the land rises to an elevation of 850 ft.

Immediately adjacent to the east is another heavily wooded low area that constitutes the current flood plain. This area is very flat and extends approximately 1500 ft to the west bank of the river.

The general topographical features in this portion of the Cedar River consist of broad valleys with relatively narrow flood plains. In many places, these broad valleys merge almost imperceptibly into the adjacent uplands. Away from the immediate vicinity of the river, the land is gently rolling farmland.

2.1.1.3 Boundaries for Establishing Effluent Release Limits

[redacted] Figure 1.2-1 is not a topographic map; however, Figure 2.1-2 shows the same boundary on a topographic map. [redacted]

2.1.2 EXCLUSION AREA AUTHORITY AND CONTROL

2.1.2.1 Authority

[redacted]

2.1.2.2 Control of Activities Unrelated to Plant Operation

The site, which consists of approximately 500 acres owned byNextEra Energy Duane Arnold, LLC, is bordered on the east by the Cedar River. The land immediately adjacent to the river is forest covered, whereas the remainder of the area is divided between flood plain and land under cultivation. All site activities are under the control of NextEra Energy Duane Arnold, LLC.

[redacted] These farmsteads were subsequently acquired by Iowa Electric and a formal vacation of this road was effected by the Linn County Board of Supervisors after public hearings held July 21, 1970, and October 12, 1971.

2.1.2.3 Arrangements for Traffic Control

[redacted]

2.1.2.4 Abandonment or Relocation of Roads

See Section 2.1.2.3.

2.1.3 POPULATION DISTRIBUTION AND LAND USE

This section presents the results of a population and land-use study performed by Dames & Moore for the area surrounding the DAEC. The plant is near the Village of Palo in Linn County, Iowa. The site is adjacent to and west of the Cedar River, and is located between two metropolitan centers; Waterloo, 40 miles to the northwest, and Cedar Rapids, 8 miles to the southeast. The location of the site is shown in relation to surrounding cultural features in Figure 2.1-4.

The Dames and Moore study provided data for the year 1970 and projected data for the year 2010. Population data to support the Dames and Moore study is provided in Figures 2.1-5 through 2.1-8 and in Tables 2.1-1 through 2.1-5.

The population and land-use program included the following:

1. A review of available pertinent literature and applicable census data.

2. Interviews with local, county, and state officials and discussions with local residents.

3. A review of maps and aerial photographs and a reconnaissance of the site and surrounding area.

The population estimates were based on data available from the 1970 Census. The method of projecting the population growth was to superimpose a radial coordinate system on an assumed area distribution of the 1970 population of townships and cities. Statistical analysis of past population growth was coupled with various population forecasts issued by Federal, state, and county agencies. An appraisal and review of the principal factors and conditions that have induced past population growth and those deemed most likely to affect future trends were assimilated into the study. Two separate projections were selected to reflect the expected differential rural and urban growth. These projections were linearly extrapolated to develop population estimates through the year 2010.

Continuing urban and industrial expansion is expected to increase both the population density and the amount of land used for residential and commercial purposes in the area. It is anticipated that the urban population in the region will double by the year 2014 and that the rural population will experience only a small increase. Although the amount of land devoted to farming is decreasing, agriculture is expected to continue to be the primary land use in the area during the life of the plant.

Based on data collected to renew the operating license to 2034, the urban population near Cedar Rapids did not double, as originally predicted, but only increased by about 12%. During the period of extended operation the population of Linn County is expected to increase by about 20%.

The greatest concentration of population, within 50 miles of the site, will continue to be along an axis connecting Waterloo, Cedar Rapids, and Iowa City. It is estimated that virtually two-thirds of the almost 1 million residents of the 50-mile region will be located in the southeast and northwest sectors by the year 2010.

The future population estimates indicate that the greatest concentrations of population will continue to be in the southeast and south-southeast sectors, at distances of 5 to 10 miles from the site. Approximately three-quarters of the total population within 10 miles of the site will be located within this area in the year 2010. This area is principally northern and northwestern suburban Cedar Rapids.

There is no reason to believe the present rural character of the area within 5 miles of the site will change during the life of the plant. In general, rural and farm population has declined both in magnitude and in proportion to the total population in recent years. Only a small increase is envisioned in this segment of the population by the year 2010.

The Cedar Rapids Municipal Airport is on the south side of Cedar Rapids, 15 miles from the plant.

2.1.3.1 Population Within 10 Miles

Population estimates for the DAEC emergency planning zone (EPZ) were updated as part of Evacuation Time Estimate (ETE) studies prepared by KLD Associates, Inc. in 1992 and by TOM COD Data Systems in 2003.

The estimates in both the 1992 and 2003 ETE updates were based on U.S. Census data, data from the Greater Cedar Rapids Chamber of Commerce, and from telephone surveys. Census block and tract data was superimposed on maps of the EPZ and estimates were developed for the 2-mile, 5-mile, and 10-mile segments as well as for the entire EPZ.

The population estimates included the permanent residents within the EPZ and the transient population, which was comprised of employees commuting to work from outside the EPZ and other visitors from outside the EPZ.

The projected population for 2010 for all increments from zero to 10 miles was extrapolated from the population forecasts available for the area. The termination dates for these population forecasts varied; however, the longest forecast had a termination date of 1999.

Figure 2.1-5 is a site vicinity map showing the 1970 and the estimated 2010 population distributions between zero and 10 miles. 1970 population data and projected population data within 10 miles of the DAEC are given in Tables 2.1-1 through 2.1-5. A discussion of the method used to interpolate beyond the 1970 data is given in Section 2.1.3.2.

The population data indicated that the area within 5 miles of the site was sparsely populated. The 1990 census showed a very slight increase in population. There are three farm dwellings within 1 mile of the plant. The closest farm dwelling is 2900 ft from the plant. The area between 5 and 10 miles of the site is also sparsely populated with the exception of the southeast sector, which includes Cedar Rapids (estimate from 1994 census update: 113,438) and Marion (estimate from 1995 special census: 23,105). The boundaries of metropolitan Cedar Rapids range from approximately 8 to 14 miles from the site. Marion is 10 miles from the site. Sixty-nine percent of the total population between 5 and 10 miles of the site occurs in southeast and south-southeast sectors.

Land-use data for the counties and townships in the area were developed from the 1964 Census of Agriculture and from data provided by the Iowa State Department of Agriculture and the Bureau of Statistics, respectively. Projections of future land use were predicated on the rate of change from 1959 to 1964.

References 1 through 10 are the publications reviewed. Table 2.1-6 presents the organizations and individuals interviewed to obtain the information presented here.

2.1.3.2 Population Between 10 and 50 Miles

Independence (population 5972), which is 27 miles from the site, and Vinton (population 5103), approximately 13 miles from the site, are the only population centers other than Cedar Rapids within 30 miles of the site. Other more distant population centers, between 30 and 50 miles of the site, are Iowa City (population 59,700), Evansdale (population 4638), Waterloo (population 65,800), Oelwein (population 6493), and Cedar Falls (population 34,900). These population figures are based on the 1990 Census.

The basis and references for the 50-mile radial population projections for the year 2010 are discussed in this section. Elements used in the projection technique were as follows:

1. Graphical analysis of the past population growth for the encompassed counties and large cities.

2. Graphical extensions of the available future population estimates.[1-5]

3. Discussion with local officials and authors of the population forecasts in regard to the principal factors and conditions that had or would affect population changes.

4. Selection and extrapolation of the appropriate population projections to the year 2010.

5. Determination of the ratio of 2010 to 1970 population numbers per county and city. (The Linn County Regional Planning Commission provided separate population estimates for approximately 100 areas within the Cedar Rapids metropolitan area to the year 1999.)

6. Multiplication of the 1970 population polar coordinate grid numbers by the above-mentioned appropriate county and city ratios.

The method of interpolating the 1980, 1990, and 2000 population numbers presented in Tables 2.1-2, 2.1-3, and 2.1-4 was based on the following:

1. Selection of Series I-C, U.S. Department of Commerce, Bureau of Census Population Projection for Iowa[11] as a representative indicator of the rate of population change.

2. Extrapolation of the Iowa Population Projection I-C to the year 2010 by determining the rate of population change, that is, the first- and second-order finite differences were determined from the provided 5-year interval projections and then the 1990 population estimate was extrapolated to the years 2000 and 2010.

3. The rate of population increase was determined by decade and applied to the base 1970 polar coordinate segmented population grid by the following equations (S.P. is the specific segmented population grid number):

   1980 S.P. = (2010 S.P. - 1970 S.P.) 17.35% + 1970 S.P.
   1990 S.P. = (2010 S.P. - 1970 S.P.) 43.75% + 1970 S.P.
   2000 S.P. = (2010 S.P. - 1970 S.P.) 71.98% + 1970 S.P.
   2010 S.P. = (2010 S.P. - 1970 S.P.) 100.00% + 1970 S.P.

Figure 2.1-6 is a site vicinity map showing the 1970 and the estimated 2010 population distributions between 10 and 50 miles. 1970 population data and projected population data from 10 to 50 miles from the DAEC are given in Tables 2.1-1 through 2.1-5. A discussion of the method used to interpolate beyond the 1970 data was given above.

2.1.3.3 Transient Population

There is no indication that the present or future population of the region will be influenced by seasonal variations. The regional population is stable.

Any noticeable population variations in the region will be of very short duration and can be associated primarily with recreational activities. Typical recreational activities that may temporarily influence the regional population are the following:

1. Athletic activities at the University of Iowa stadium in Iowa City. The stadium has a seating capacity of 89,000 and is 31 miles southeast of the site.

2. The All Iowa Fair in Cedar Rapids during July of each year. Approximately 75,0000-85,000 people per year visit the fair (approximately 13,000 per day).

3. Attendance at the various county parks within the region, as described in Section 2.1.3.4.3.

4. The Cedar Rapids Freedom Festival running from the last week in June through July 4 annually. Total festival attendance in 1996 was estimated to be 341,000, averaging a daily attendance of 46,625. In 1996, many of the major activities were moved to Kirkwood Community College. The fireworks display was held in downtown Cedar Rapids with an estimated attendance of 100,000 people.

5. The Taste of Iowa festival is held on the Labor Day weekend in downtown Cedar Rapids. Estimated total attendance of 55,000 averaging a daily attendance of 13,750.

2.1.3.4 Low-Population Zone

The nearest population center is the city of Cedar Rapids, 8 miles from the site (Section 2.1.3.5). Since the population center must be at least 1-1/3 times the distance to the outer boundary of the low-population zone (LPZ), the LPZ outer boundary was initially chosen to be 6 miles. The LPZ boundary distance was based on the DAEC population analysis (Section 2.1.3), the required distance relationship between the nearest population center and the LPZ (the 1-1/3 relationship described above), and the possibility of warning and evacuation within the LPZ. The LPZ boundary distance and the Cedar Rapids population center distance are shown in Figure 2.1-7. In 1984 the offsite radiological consequences of the loss-of-coolant accident and the control rod drop accident were calculated to evaluate the radiological effects of the power uprate. The outer boundary was chosen to be 2 miles for these calculations to coincide with the 2-mile radius zone used for emergency planning for the DAEC. In 1987 the EPZ was expanded from a 10 mile radius zone to a 10 mile radius zone plus the part of the Cedar Rapids/Marion metropolitan area that is beyond that radius. In 1992, the DAEC EPZ was redefined to utilize landmarks (roads, highways, river) as boundary descriptors for individual emergency planning subareas. The DAEC EPZ includes the area within these boundaries around an approximate 10-mile radius from the DAEC. This area includes Cedar Rapids, Marion, Hiawatha, Center Point, Alburnett, Palo, and Robins in Linn County, and Shellsburg, Atkins, and Urbana in Benton County. The DAEC EPZ is shown in Figure 2.1-7a.

In 2000, the DAEC reperformed a radiological consequences analysis for all design basis accidents using the methods of RG 1.183 as part of adopting the Alternative Source Term under

10 CFR 50.67. This analysis determined the offsite dose consequences for the limiting 2-hr dose at the Exclusion Area Boundary (936 meters NW of the offgas stack) and at the 2 mile LPZ (9218 meters). This analysis is described in Chapter 15 and was adopted as Amendments 237 and 240. This analysis was updated later and approved as Amendment 261.

2.1.3.4.1 Public Facilities

There are no hospitals located in the low-population zone.

The emergency plans for the DAEC include the notification of local law enforcement agencies and schools when any conditions exist at the plant that could endanger the health and safety of the faculty and students at any of the schools. There is one school in the low-population zone: Shellsburg – Kindergarten through 6th grade, approximately 340 students, 5 miles west of the DAEC.

The enrollment at the remainder of the schools and the number of people using the public facilities within the Emergency Planning Zone, which includes the 6-mile LPZ, are included in either the permanent population count or the transient population count, as shown in the chart below.

The 10-mile Emergency Planning Zone was expanded in 1987 to include the entire Cedar Rapids/Marion metropolitan area; the population figures in the chart below reflect that expansion.

ESTIMATED 2000 POPULATION WITHIN 10 MILE RADIUS
(Based on 2003 Evacuation Time Estimate Study)

EPZ Subarea		Permanent Population	Transient Population
1	(0-2 miles)	1134	313
2	(0-5 miles)	316	25
3	(0-5 miles)	703	25
4	(0-5 miles)	2504	
5	(0-5 miles)	3953	397
6	(0-5 miles)	70	
7	(0-5 miles)	1468	
8	(0-5 miles)	243	
9	(0-10 miles)	2260	35
10	(0-10 miles)	405	25
11	(0-10 miles)	132	
12	(0-10 miles)	854	25
13	(0-10 miles)	706	25
14	(0-10 miles)	34960	188
15	(0-10 miles)	29820	3558
16	(0-10 miles)	31540	652
17	(0-10 miles)	1991	100
18	(0-10 miles)	1488	10
19	(0-10 miles)	218	
20	(0-10 miles)	368	
21	(0-10 miles)	740	105
22	(0-10 miles)	1353	25
23	Beyond 10 miles	35074	993
24	Beyond 10 miles	22140	3878
Totals		174,440	10,378

Note: For subareas, please refer to Figure 2.1-7a.

2.1.3.4.2 Agriculture

The area within 10 miles of the site is in two counties: two-thirds in Linn County and one-third in Benton County. Linn County, although it includes the Cedar Rapids metropolitan area, is predominantly rural. Benton County remains a typical agricultural area. A trend toward fewer farms of increasing size is evident in both Linn and Benton Counties.

Available statistics indicate that the area surrounding the site is used primarily for agricultural purposes. The major harvested crop is corn, with secondary crops of oats and soybeans. The major livestock animals are cattle and hogs. Poultry is also a significant farm product.

Pertinent data relative to the area surrounding the plant are presented in Table 2.1-7. Data pertaining to the agricultural uses of the land surrounding the site are presented in Table 2.1-8.

The distribution and use of the land area by township are shown in Figure 2.1-8.

Table 2.1-9 provides the acreage, yields per acre, and production of farm crops for Benton and Linn Counties during the year 1970.[16] The two counties represent an area in excess of the 10-mile radius around the plant site.

2.1.3.4.3 Recreational Activities

Recreational activities within the vicinity are associated primarily with parks and similar public property. Principal conservation and recreational areas are described below and are shown in Figure 2.1-8.

Directly east of the site and adjacent to the eastern bank of the Cedar River lies the Wickiup Hills Natural Area. This is a 563-acre unit that is generally undeveloped and used principally for hiking, wilderness camping, nature study, and hunting. The Waltonian Archery League has developed a range in this area complete with lodge and pavilion.

The Palo Marsh Natural Area is 2 miles south of the site in the Cedar River floodplain. It is a migratory bird refuge and game preserve 144 acres in area.

Chain Lakes Natural Area is 5 miles south of the site on an island and south bank of the Cedar River. This is a popular 373-acre picnic and boat-launching area.

Morgan Creek Park consists of a 230-acre tract about 10 miles south of the site along the western boundary of Cedar Rapids. It is not adjacent to the river and is largely undeveloped.

Seminole Valley Park and Campground is a large, well-developed park of 409 acres located on the north bank of the Cedar River 2 miles northwest of Cedar Rapids. It has facilities for camping and picnicking and contains playground equipment.

Ellis Park is 396 acres in area and is located on the Cedar River along the northwestern edge of Cedar Rapids. It has wading and swimming pools, picnic areas, pavilions, a duck pond, a golf course, softball, ice skating, and fishing.

Twin Pines Park has an 18-hole golf course and consists of 150 acres located 1 mile north of Ellis Park.

There are two nearby parks in Benton County. Wildcat Bluff is 119 acres of parkland situated 2 miles south of Urbana, and the Benton City-Fry Access is 40 acres located 5 miles east of Vinton.

The Sac and Fox Trail is located approximately 14 miles southwest of the site, bordering the Cedar River on both north and south banks. The trail is primarily used for biking and walking.

Squaw Creek Park is located approximately 12 miles east-southeast of the site. This is a 663 acre park used primarily for camping, picnicking, hiking, and cross country skiing.

The Cedar Valley Nature Trail is a multipurpose trail used for biking, walking, and running. It originates in Hiawatha, approximately 5 miles east of the site and angles north and west through Center Point and Urbana.

Pleasant Creek Recreation Area is a state park located 1 mile northwest of the site. Pleasant Creek was developed for use by the DAEC if river flow was not sufficient for the operation of the plant. The 1927 acre area is used for camping, boating, fishing and hiking.

The North Cedar Area is located approximately 4 miles northwest of the site. The 56 acre area includes a boat ramp.

Wakema Park is located in northwest Center Point and is approximately 6.5 miles north of the site. This 5 acre area is used primarily for picnicking and recreation.

The Coralville Dam and Reservoir Project is located along the Iowa River 26 to 30 miles southeast of the site. Within this region are found the Hawkeye Wildlife Area and Lake Macbride State Park. The reservoir conservation pool at the summer elevation of 680 ft covers an area of 4900 acres. The reservoir project boundaries enclose approximately 25,000 acres.

Recreational activities include fishing, swimming, picnicking, camping, hunting, and boating.

2.1.3.5 Population Center

Metropolitan centers closest to the site are Cedar Rapids, approximately 8 miles to the southeast; Waterloo, approximately 40 miles to the northwest; Iowa City, approximately 35 miles to the southeast; and Davenport, approximately 75 miles to the southeast. Aside from these urban centers, the area is sparsely populated and used primarily for agricultural purposes.

The nearest boundary of the Cedar Rapids metropolitan area, which is the nearest boundary of a population center containing more than approximately 25,000 persons, is about 8 miles from the plant site. Therefore, the population center distance for the DAEC is considered to be 8 miles.

2.1.3.6 Population Density

Figure 2.1-9 is a site vicinity map giving the 1970 and projected 2010 population densities within 10 miles of the DAEC. Figure 2.1-10 is a regional map providing similar information out to 50 miles.

REFERENCES FOR SECTION 2.1

1. Department of Commerce, Current Population Reports, United States Census of Population, 1970, Iowa, Bureau of Census, 1970.

2. Doerflinger, J., Iowa's Population: Recent Trends, Future Prospects, Iowa State University, Special Report No. 47, 1966.

3. Hartman, J., Characteristics and Structure of Iowa's Population, Iowa State University, Special Report No. 57, 1968.

4. Hartman, J., Trends and Changing Structure of Iowa's Population, reprint from Iowa Farm Science, Vol. 22, No. 8, pp. 7-10, 1968.

5. Howard, Needles, Tammen, and Bergendorff, Linn County Regional Planning Commission, Transportation Study, Part 1, Population and Part 2, Economy, 1967.

6. Office for Planning and Programming, Iowa Trend, Actual and Projected, 1960-1980, State Capital, Des Moines, Iowa, 1968.

7. Cedar Rapids Chamber of Commerce, Liveability Data and Information on Cedar Rapids, Iowa, Research Department, Iowa.

8. Cedar Rapids Chamber of Commerce, Manufacturing Data for Cedar Rapids, Research Department, Iowa.

9. State Conservation Commission, Outdoor Recreation in Iowa, Planning and Coordination Section, 1968.

10. Cedar Rapids Chamber of Commerce, Population Project and Economic Growth Indicators, Research Department, Iowa.

11. Department of Commerce, Population Estimates and Projections, Series P-25, No. 477, Social and Economic Statistics Administration, U.S. Bureau of Census, 1972.

12. Discussion with representative of St. Lukes Hospital, April 1972.

13. Discussion with representatives of:

    Mt. Mercy College
    Coe College
    Benton County School Districts
    Linn County School Districts
    Catholic School District
    Private schools
    Cedar Rapids School District

14. Discussion with the Commissioner of Public Parks for Cedar Rapids.

15. Modified list of parks from FSAR Figure 2.1-9.

16. Iowa Annual Farm Census 1970, Iowa Department of Agricultural Statistics Bulletin No. 92-AF.

17. Final Report for the Duane Arnold Energy Center Evacuation Time Estimate, December, 1992.

18. Final Report for the DAEC ETE, 2003.

Table 2.1-1 Sheet 1 of 2

1970 POPULATION

Direction	Miles from Plant Site 0-1	12	2-3	3-4	4-5	5-6	6-10
N	0	5	35	55	50	330	1,560
NNE	0	15	30	45	40	40	260
NE	0	15	30	40	40	40	250
ENE	0	15	35	50	50	110	560
E	0	20	65	40	60	60	350
ESE	0	15	20	40	65	1,180	5,400
SE	0	50	50	50	90	5,120	31,700
SSE	0	25	85	20	60	3,410	22,050
S	0	20	20	35	15	100	570
SSW	0	10	240	185	20	130	790
SW	3	10	20	25	25	115	200
WSW	4	15	15	20	90	40	250
W	2	20	30	40	330	90	390
WNW	0	20	20	25	35	40	250
NW	5	5	35	25	20	40	260
NNW	1	0	0	20	25	125	770
Totals	15	260	730	715	1,015	10,970	65,610
Cummulative totals	15	275	1,005	1,720	2,735	13,705	79,315

Table 2.1-1 Sheet 2 of 2

1970 POPULATION

Direction	Miles from Plant Site 10-20	20-30	30-40	40-50	Totals
N	1,366	2,360	1,488	13,069	20,318
NNE	1,429	2,018	7,260	5,895	17,032
NE	1,492	1,676	3,750	7,331	14,664
ENE	2,913	1,846	6,770	7,876	20,225
E	2,102	6,405	2,772	3,914	15,788
ESE	10,741	2,384	5,439	4,338	29,622
SE	45,361	7,099	9,338	7,639	106,497
SSE	15,481	2,936	51,958	5,149	101,174
S	2,020	3,120	2,110	5,536	13,546
SSW	805	1,964	4,092	4,283	12,519
SW	1,486	3,043	2,986	3,030	10,943
WSW	962	4,214	2,380	6,130	14,120
W	438	2,000	1,774	5,591	10,705
WNW	6,027	1,836	3,475	6,978	18,706
NW	1,926	3,928	27,443	91,987	125,674
NNW	1,646	7,926	1,650	6,319	18,482
Totals	96,195	54,755	134,685	185,065	550,015
Cummulative totals	175,510	230,265	364,950	550,015	550,015

Table 2.1-2 Sheet 1 of 2

1980 POPULATION

Direction	0-1	1-2	2-3	3-4	4-5	5-6	6-10
	Miles from Plant Site						
N	0	5	38	61	55	368	1,649
NNE	0	17	33	49	44	44	282
NE	0	17	33	44	44	44	273
ENE	0	17	38	55	55	121	613
E	0	22	72	44	73	73	419
ESE	0	17	30	61	100	2,060	9,017
SE	0	55	76	85	126	6,339	37,557
SSE	0	28	114	30	102	4,790	28,328
S	0	22	22	38	18	287	1,394
SSW	0	11	268	204	22	146	869
SW	3	11	22	28	28	127	216
WSW	4	16	16	22	106	44	273
W	3	22	33	47	386	96	428
WNW	1	22	22	28	45	44	273
NW	5	6	38	28	23	44	283
NNW	2	0	1	22	28	141	844
Totals	18	285	858	846	1,253	14,770	82,720
Cummulative totals	18	303	1,161	2,007	3,260	18,029	100,749

Table 2.1-2 Sheet 2 of 2

1980 POPULATION

Direction	10-20	20-30	30-40	40-50	Totals
	Miles from Plant Site				
N	1,395	3,072	2,052	12,412	21,109
NNE	1,626	2,438	7,050	6,454	18,037
NE	1,856	1,803	4,375	7,613	16,104
ENE	3,510	2,257	6,822	7,929	21,418
E	3,320	6,338	3,469	4,519	18,348
ESE	15,801	3,115	8,026	4,870	43,097
SE	49,755	7,113	13,602	7,599	122,308
SSE	19,340	3,563	47,239	5,475	109,009
S	2,495	3,605	4,451	5,729	18,061
SSW	1,228	2,509	5,046	4,398	14,700
SW	1,528	3,259	3,090	3,068	11,380
WSW	1,205	4,060	2,523	5,900	14,169
W	882	2,064	1,957	5,724	11,641
WNW	5,417	2,002	5,244	14,274	27,371
NW	1,944	3,804	26,935	91,937	125,046
NNW	1,670	7,390	3,902	13,982	27,980
Totals	112,971	58,392	145,783	201,882	619,778
Cummulative totals	213,720	272,113	417,896	619,778	619,778

Table 2.1-3 Sheet 1 of 2

1990 POPULATION

Direction	Miles from Plant Site 0-1	1-2	2-3	3-4	4-5	5-6	6-10
N	0	5	44	70	63	426	1,785
NNE	0	19	39	56	51	51	315
NE	0	19	39	51	51	51	309
ENE	0	19	44	63	63	138	693
E	0	24	82	51	93	93	523
ESE	0	19	46	92	152	3,398	14,522
SE	0	63	116	137	182	8,193	46,470
SSE	0	32	159	46	165	6,890	37,881
S	0	24	24	44	22	572	2,648
SSW	0	12	310	233	24	169	989
SW	4	12	24	32	32	146	242
WSW	4	17	17	24	129	51	309
W	3	24	39	57	470	105	486
WNW	2	24	24	32	59	51	309
NW	5	7	44	32	27	51	317
NNW	3	0	2	24	32	164	956
Totals	22	323	1,054	1,045	1,614	20,551	108,754
Cummulative totals	22	345	1,399	2,444	4,058	24,610	133,364

Table 2.1-3 Sheet 2 of 2

1990 POPULATION

Direction	10-20	20-30	30-40	40-50	Totals
	Miles from Plant Site				
N	1,440	4,155	2,911	11,411	22,312
NNE	1,925	3,076	6,730	7,305	19,567
NE	2,409	1,997	5,327	8,043	18,296
ENE	4,419	2,882	6,902	8,009	23,233
E	5,174	6,236	4,529	5,439	22,244
ESE	23,500	4,228	11,963	5,679	63,602
SE	56,441	7,134	20,091	7,538	146,365
SSE	25,211	4,517	40,058	5,971	120,930
S	3,217	4,344	8,013	6,022	24,931
SSW	1,872	3,337	6,499	4,573	18,019
SW	1,593	3,588	3,248	3,125	12,044
WSW	1,575	3,826	2,741	5,549	14,244
W	1,557	2,161	2,234	5,927	13,064
WNW	4,489	2,254	7,935	25,375	40,555
NW	1,971	3,616	26,162	91,861	124,090
NNW	1,705	6,575	7,327	25,643	42,433
Totals	138,498	63,927	162,670	227,471	725,930
Cummulative totals	271,862	335,789	498,459	725,930	725,930

Table 2.1-4 Sheet 1 of 2

2000 POPULATION

	Miles from Plant Site						
Direction	0-1	1-2	2-3	3-4	4-5	5-6	6-10
N	0	5	49	80	72	488	1,931
NNE	0	22	44	63	58	58	350
NE	0	22	44	58	58	58	347
ENE	0	22	49	72	72	157	780
E	0	27	94	58	114	114	634
ESE	0	22	63	126	209	4,829	20,408
SE	0	72	158	194	241	10,177	56,000
SSE	0	36	207	63	233	9,136	48,096
S	0	27	27	49	26	877	3,989
SSW	0	14	355	264	27	195	1,118
SW	4	14	27	36	36	165	268
WSW	5	19	19	27	155	58	347
W	4	27	44	69	560	115	548
WNW	4	27	27	36	75	58	347
NW	5	9	49	36	31	58	354
NNW	4	0	4	27	36	190	1,076
Totals	26	364	1,263	1,258	2,001	26,734	136,593
Cumulative totals	26	390	1,653	2,911	4,912	31,646	168,239

Table 2.1-4 Sheet 2 of 2

2000 POPULATION

Direction	10-20	20-30	30-40	40-50	Totals
	Miles from Plant Site				
N	1,488	5,314	3,829	10,342	23,598
NNE	2,245	3,759	6,388	8,216	21,203
NE	3,001	2,204	6,345	8,502	20,640
ENE	5,391	3,550	6,987	8,094	25,174
E	7,156	6,126	5,663	6,423	26,410
ESE	31,733	5,418	16,173	6,545	85,527
SE	63,590	7,157	27,029	7,473	172,091
SSE	31,489	5,537	32,380	6,501	133,678
S	3,989	5,134	11,822	6,335	32,277
SSW	2,560	4,223	8,052	4,761	21,568
SW	1,662	3,939	3,416	3,187	12,755
WSW	1,970	3,576	2,974	5,175	14,324
W	2,279	2,265	2,531	6,143	14,587
WNW	3,497	2,523	10,813	37,246	54,653
NW	1,999	3,414	25,335	91,779	123,068
NNW	1,744	5,704	10,991	38,113	57,887
Totals	165,795	69,846	180,728	254,833	839,440
Cummulative totals	334,034	403,879	584,607	839,440	839,440

Table 2.1-5 Sheet 1 of 2

2010 POPULATION

Direction	Miles from Plant Site 0-1	1-2	2-3	3-4	4-5	5-6	6-10
N	0	5	55	90	80	550	2,075
NNE	0	25	50	70	65	65	385
NE	0	25	50	65	65	65	385
ENE	0	25	55	80	80	175	865
E	0	30	105	65	135	135	745
ESE	0	25	80	160	265	6,250	26,250
SE	0	80	200	250	300	12,145	65,460
SSE	0	40	255	80	300	11,365	58,235
S	0	30	30	55	30	1,180	5,320
SSW	0	15	400	295	30	220	1,245
SW	5	15	30	40	40	185	295
WSW	5	20	20	30	180	65	385
W	5	30	50	80	650	125	610
WNW	5	30	30	40	90	65	385
NW	5	10	55	40	35	65	390
NNW	5	0	5	30	40	215	1,195
Totals	30	405	1,470	1,470	2,385	32,870	164,225
Cummulative totals	30	35	1,905	3,375	5,760	38,630	202,855

Table 2.1-5 Sheet 2 of 2

2010 POPULATION

Direction	10-20	20-30	30-40	40-50	Totals
	Miles from Plant Site				
N	1,536	6,464	4,740	9,280	24,875
NNE	2,562	4,437	6,048	9,119	22,826
NE	3,588	2,410	7,355	8,958	22,966
ENE	6,356	4,214	7,071	8,179	27,100
E	9,124	6,018	6,788	7,400	30,545
ESE	39,905	6,599	20,352	7,404	107,290
SE	70,686	7,180	33,916	7,408	197,625
SSE	37,721	6,549	24,759	7,027	146,331
S	4,756	5,918	15,603	6,646	39,568
SSW	3,243	5,103	9,593	4,947	25,091
SW	1,730	4,288	3,584	3,248	13,460
WSW	2,363	3,328	3,205	4,803	14,404
W	2,996	2,368	2,826	6,358	16,098
WNW	2,512	2,791	13,670	49,028	68,646
NW	2,028	3,214	24,514	91,698	122,054
NNW	1,782	4,839	14,627	50,489	73,227
Totals	192,888	75,720	198,651	281,992	952,106
Cummulative totals	395,743	471,463	670,114	952,106	952,106

Table 2.1-6

PRINCIPLE SOURCES OF DATA FOR GEOGRAPHY AND DEMOGRAPHY

A bibliography is included a the end of Section 2.1, which lists the publications and reference material. The individuals and agencies interviewed are as follows:

Mr. Harold F. Ewoldt, Cedar Rapids Chamber of Commerce, Iowa.

Mr. C. Miller, Linn County Regional Planning Office, Iowa.

Mr. David Hammond, Iowa State University, Extension Office.

Messrs. W. Harrington and W. Bethrani, Linn County Engineering Office, Iowa.

Mr. Forest Holveck, Linn County Assessors Office, Iowa.

Mr. Charles Mullenix, Green Engineering.

Mr. James Mollingshead, Iowa State Planning Department, Des Moines, Iowa.

Mr. John Flaming, Iowa State Conservation Department.

Mr. G. Anderson, Iowa State Highway Commission, Ames, Iowa.

Dr. John Hartman, Assistant Professor of Sociology, Iowa State University, Iowa.

Mr. D. Lunberg, Iowa Urban Planning Department, Iowa City, Iowa.

Dr. Orville Van Eck, Assistant State Geologist, Iowa Geological Survey, Iowa City, Iowa.

Mr. Rex Myres, Farmer, Palo, Iowa.

Mr. Norman Pint, Waltonian Archery League, Cedar Rapids, Iowa.

Mr. Robert Brooks, KCRG Radio, Sports Department, Cedar Rapids, Iowa.

Cedar Rapids Parks Department and Water Department, Iowa.

Iowa City Chamber of Commerce, County Assessor, County Engineer, and Recreation Department, Iowa.

Iowa State Department of Agriculture, Highway Commission, and Natural Resources Council.

Linn County Chamber of Commerce, Conservation Board and Bureau, Iowa.

U. S. Department of Agriculture.

U. S. Corps of Engineers, Coralville Dam Project.

Table 2.1-7

LOCAL LAND USE

	Linn County		Benton County	
	1964	1969	1964	1969
Total area				
Square miles	713	717	718	718
Acres	456,320	458,752	459,520	459,264
Farm area				
Square miles	622	596	700	699
Acres	398,080	391,732	488,485	447,198
Cedar Rapids metropolitan Area				
Square miles	50	50	- -	- -
Acres	32,000	32,000	- -	- -
Area remaining (forests, water urban, industrial, commercial)				
Square miles	41	71	18	19
Acres	25,725	45,020	11,035	12,066
Area devoted to Farming, %	87.3	83.2	97.5	97.4
Total Farms	2343	2118	1991	1827
Average farm size, acres	170.1	180.2	225.3	244.7

Table 2.1-8

AGRICULTURAL DATA

Farm Types	Linn County		Benton County	
Dairy	206 (8.8%)[a]	189 (8.8%)	94 (4.7%)	82 (4.7%)
Poultry	7 (0.3%)	6 (0.3%)	11 (0.5%)	9 (0.5%)
Livestock other than dairy and poultry (beef cattle, hogs, sheep, etc.)	913 (38.9%)	834 (38.9%)	1193 (59.9%)	1049 (59.9%)
Vegetable	5 (0.2%)	4 (0.2%)	- -	- -
Fruit and nut	4 (0.2%)	4 (0.2%)	- -	- -
Other field crops (corn, soybeans, oats, etc.)	560 (23.9%)	513 (23.9%)	402 (20.2%)	354 (20.2%)
General farms	201 (8.5%)	182 (8.5%)	146 (7.3%)	128 (7.3%)
Unclassified	447 (19.2%)	412 (19.2%)	145 (7.3%)	128 (7.3%)

[a] Percentage of total farms.

Table 2.1-9

1970 IOWA FARM CENSUS

Crop	Linn County	Benton County
Corn		
Field corn harvested for all purposes, acres	152,050	135,403
Harvested for grain		
Acres	144,910	127,931
Yield per acre, bushel	94.1	92.5
Production, bushel	13,632,306	11,833,381
Harvested for silage		
Acres	6,936	4,088
Yield per acres, ton	15.6	15.1
Production, ton	108,363	61,541
Harvested for all other purposes, acres	204	384
Oats		
Acres harvested	30,480	25,167
Yield per acre, bushel	59.0	51.7
Production, bushel	1,798,370	1,302,357
Soybeans for beans		
Acres harvested	76,096	53,469
Yield per acre, bushel	37.7	35.0
Production, bushel	2,865,102	1,872,072
Sorghum		
Acres harvested for grain	46	7
Yield per acres, bushel	68.0	42.9
Production, bushel	3,130	300
Acres harvested for all other purposes	171	1,624
All wheat		
Acres harvested	- -	13
Yield per acre, bushel	- -	23.1
Production, bushel	- -	300

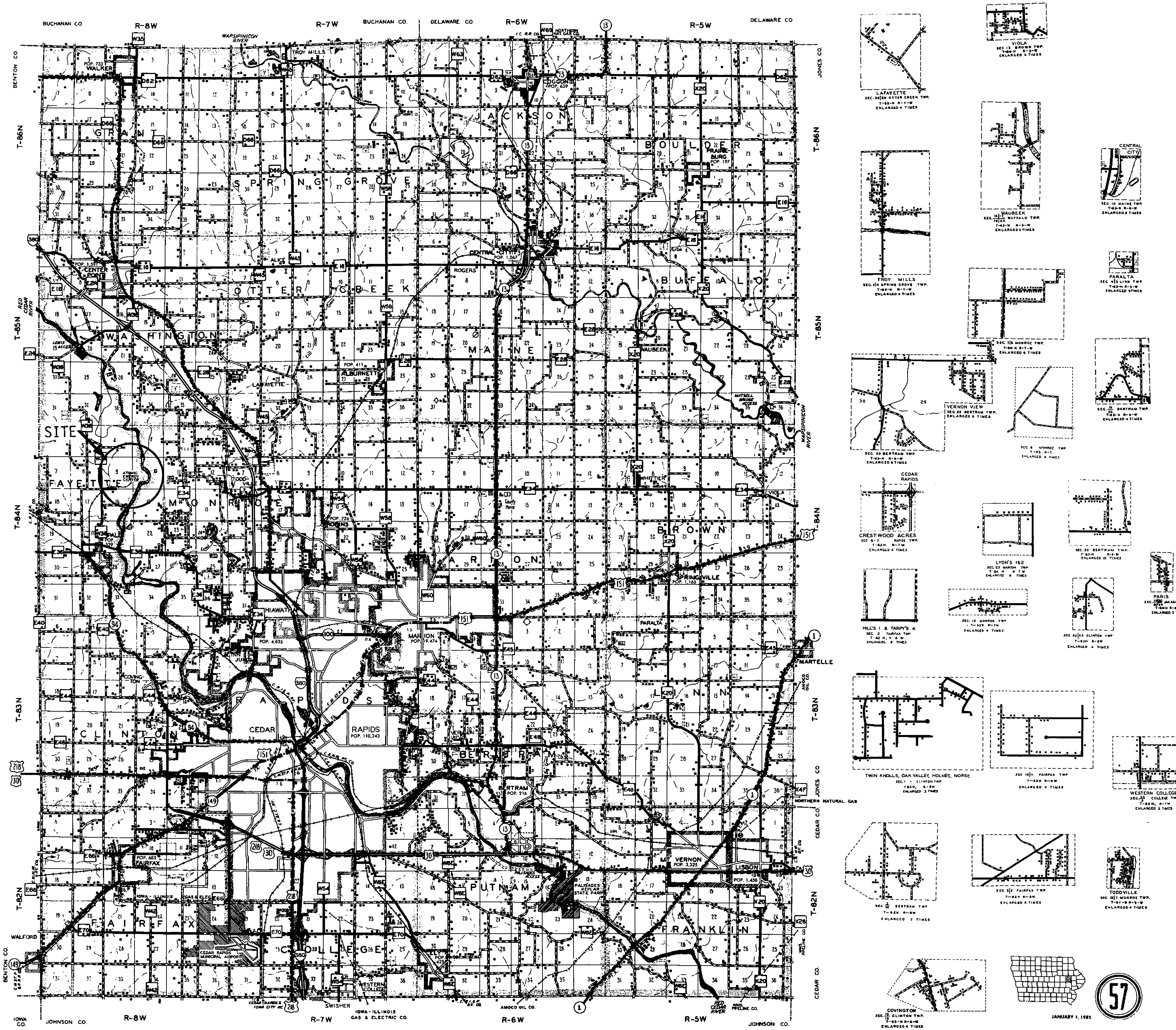
1970 IOWA FARM CENSUS

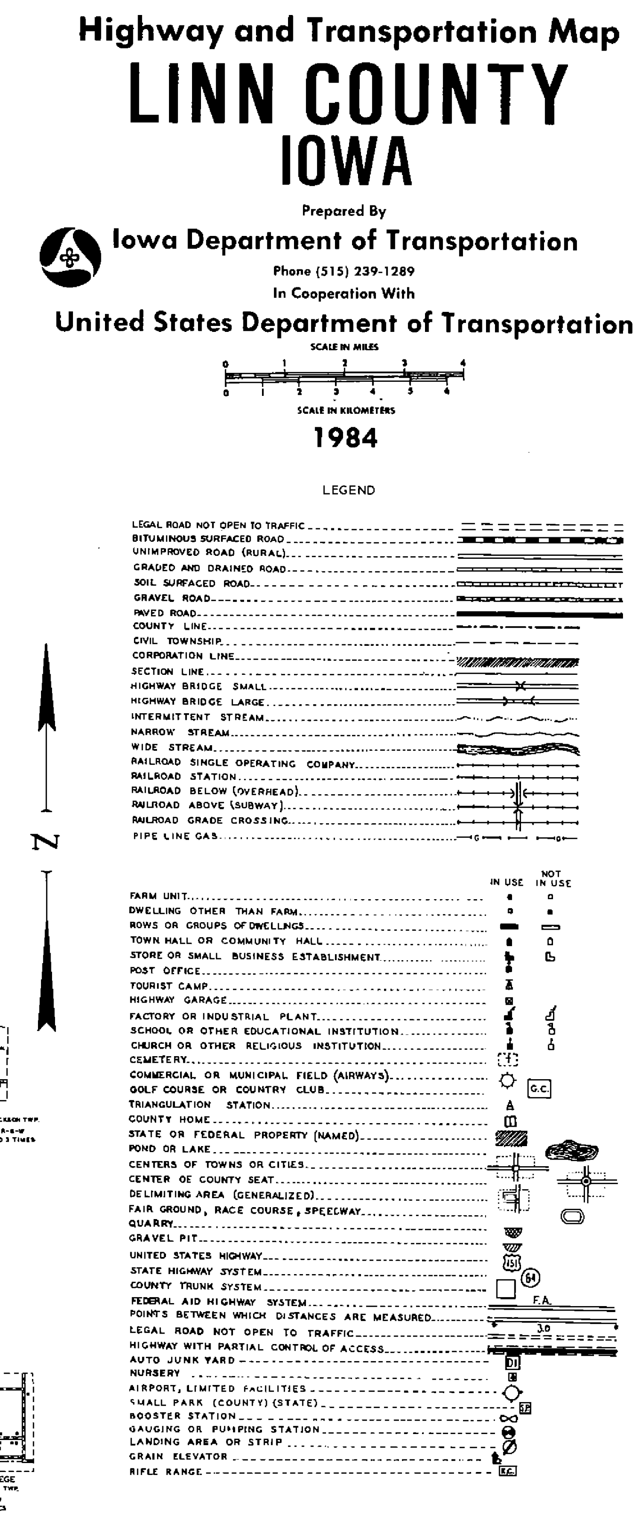
Crop	Linn County	Benton County
Barely		
Acres harvested	- -	52
Yield per acre, bushel	- -	40.0
Production, bushel	- -	2,080
Rye		
Acres harvested	27	26
Yield per acre, bushel	33.3	34.5
Production, bushel	900	898
Timothy seed		
Acres harvested	- -	7
Yield per acre, lb	- -	157
Production, lb	- -	1,100
Red clover seed		
Acres harvested	4	40
Yield per acre, lb	52	73
Production, lb	210	2,940
Popcorn		
Acres harvested	88	1
Yield per acres, lb	3,379	1,000
Production, lb	297,360	1,000

DUANE ARNOLD ENERGY CENTER
IOWA ELECTRIC LIGHT & POWER COMPANY
UPDATED FINAL SAFETY ANALYSIS REPORT

Site Aerial Photo
Figure 2.1-1

[redacted]





DUANE ARNOLD ENERGY CENTER
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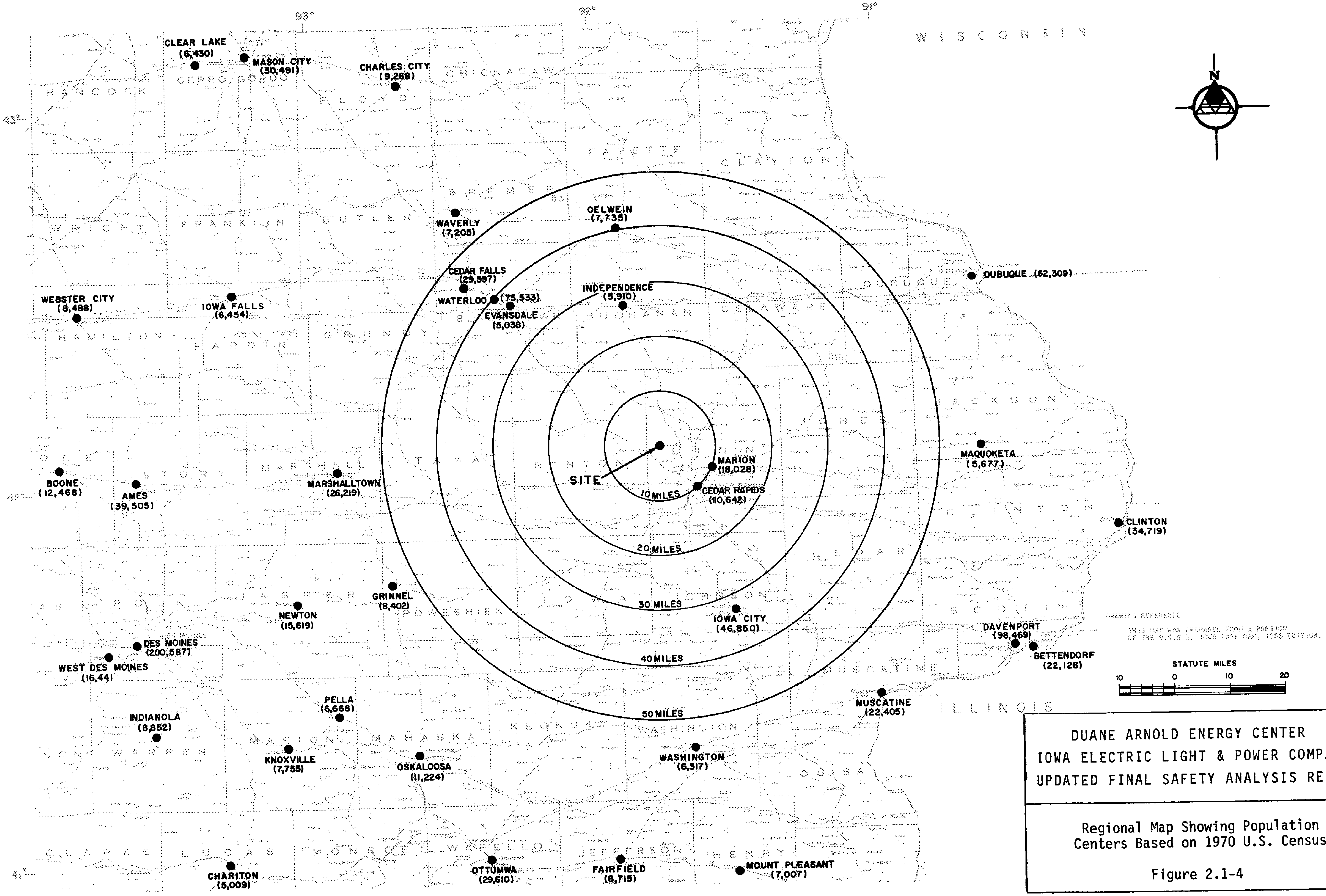
Site Location
Figure 2.1-3



DUANE ARNOLD ENERGY CENTER
IOWA ELECTRIC LIGHT & POWER COMPANY
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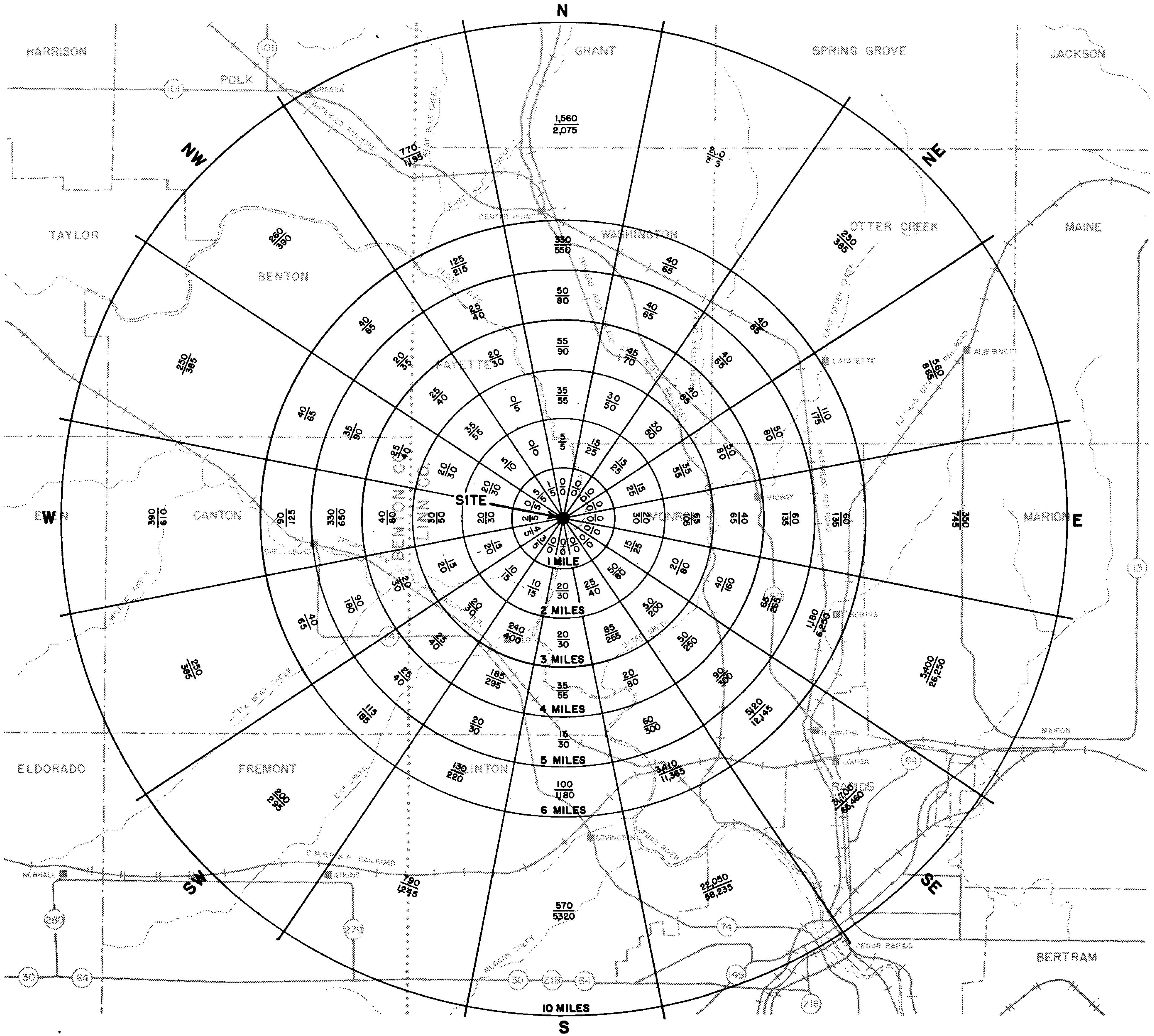
Regional Map Showing Population Centers Based on 1970 U.S. Census

Figure 2.1-4



RADIUS IN MILES	CUMULATIVE TOTAL	
	1970	2010
1	15	35
2	275	440
3	1,005	1,910
4	1,720	3,380
5	2,735	5,765
6	13,705	38,635
10	79,315	202,860

SECTOR DISTRIBUTION	1970	2010
N	2035	2,855
NNE	430	660
NE	415	655
ENE	820	1,280
E	595	1,215
ESE	6,720	33,030
SE	37,060	78,435
SSE	25,650	70,275
S	760	6,645
SSW	1,375	2,210
SW	398	610
WSW	434	705
W	902	1,550
WNW	390	645
NW	390	600
NNW	941	1,490

KEY:

$\frac{400}{800} = \frac{\text{1970 POPULATION}}{\text{ESTIMATED 2010 POPULATION}}$

STATUTE MILES

1 0 1 2 3

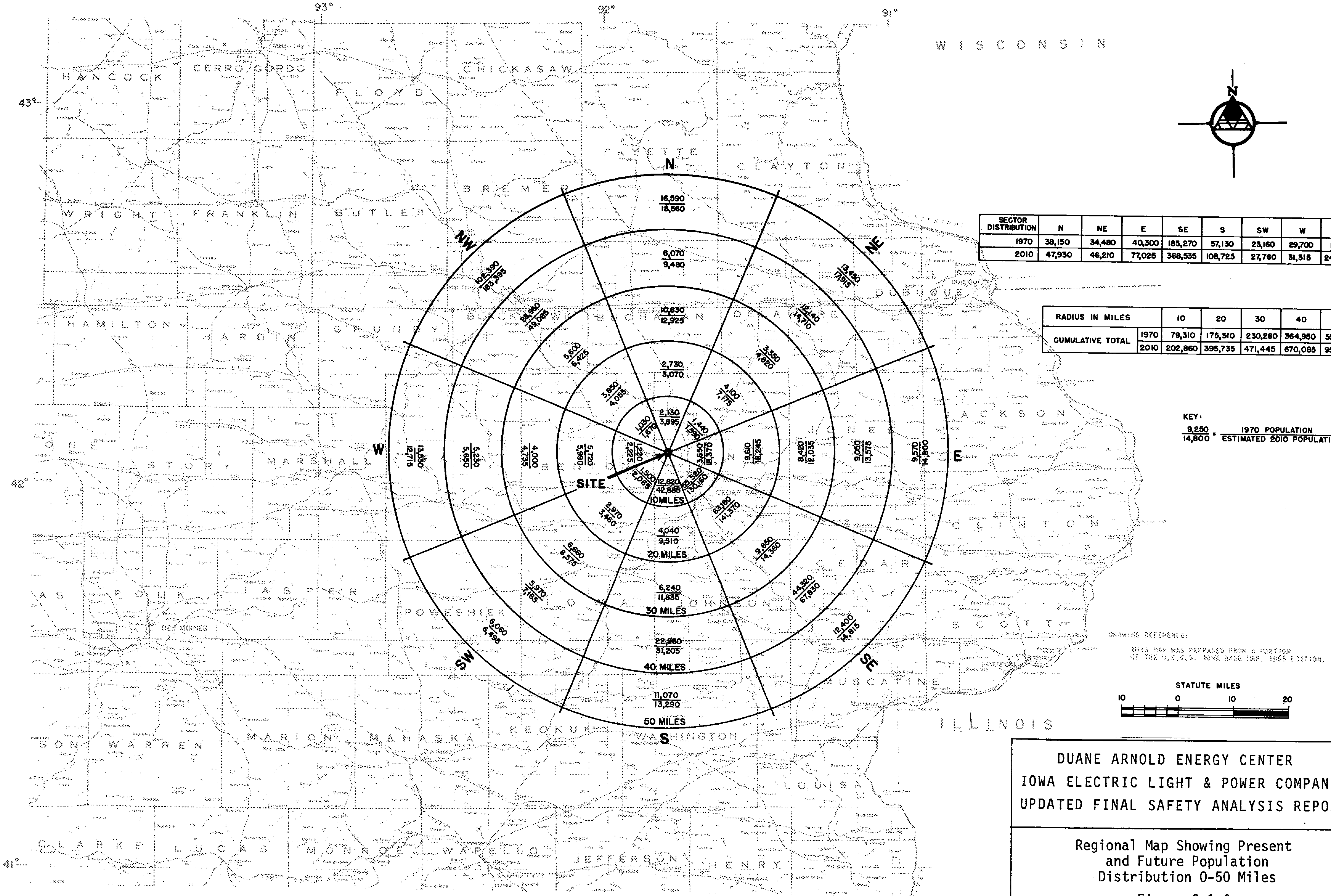
DUANE ARNOLD ENERGY CENTER

IOWA ELECTRIC LIGHT & POWER COMPANY

UPDATED FINAL SAFETY ANALYSIS REPORT

Site Vicinity Map Showing Present and Future Population Distribution 0-10 Miles

Figure 2.1-5



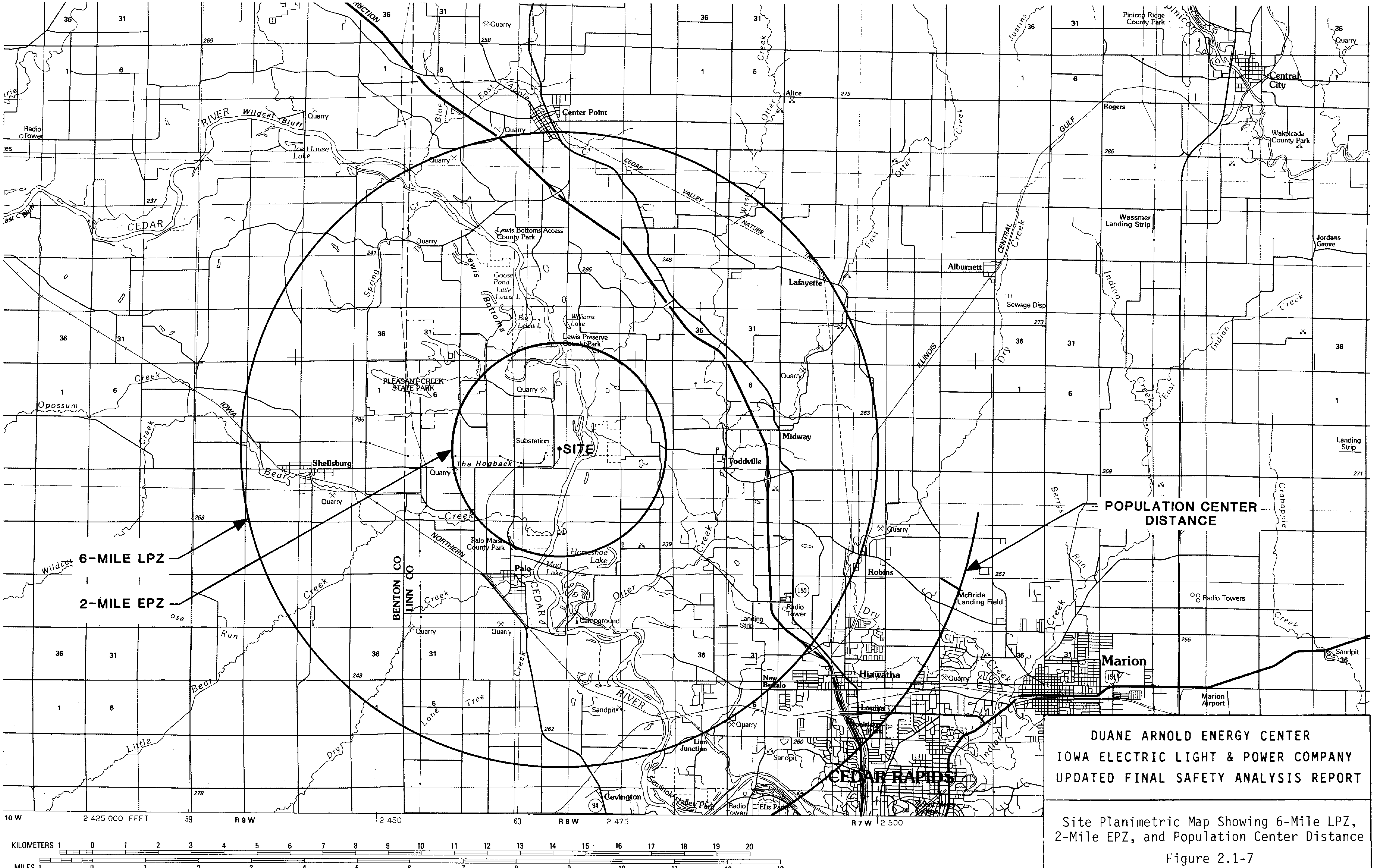
SECTOR DISTRIBUTION	N	NE	E	SE	S	SW	W	NW
1970	38,150	34,480	40,300	185,270	57,130	23,160	29,700	141,820
2010	47,930	46,210	77,025	368,535	108,725	27,760	31,315	244,570

RADIUS IN MILES		10	20	30	40	50
CUMULATIVE TOTAL	1970	79,310	175,510	230,260	364,950	550,010
	2010	202,860	395,735	471,445	670,085	952,070

DUANE ARNOLD ENERGY CENTER
IOWA ELECTRIC LIGHT & POWER COMPANY
UPDATED FINAL SAFETY ANALYSIS REPORT

Regional Map Showing Present and Future Population Distribution 0-50 Miles

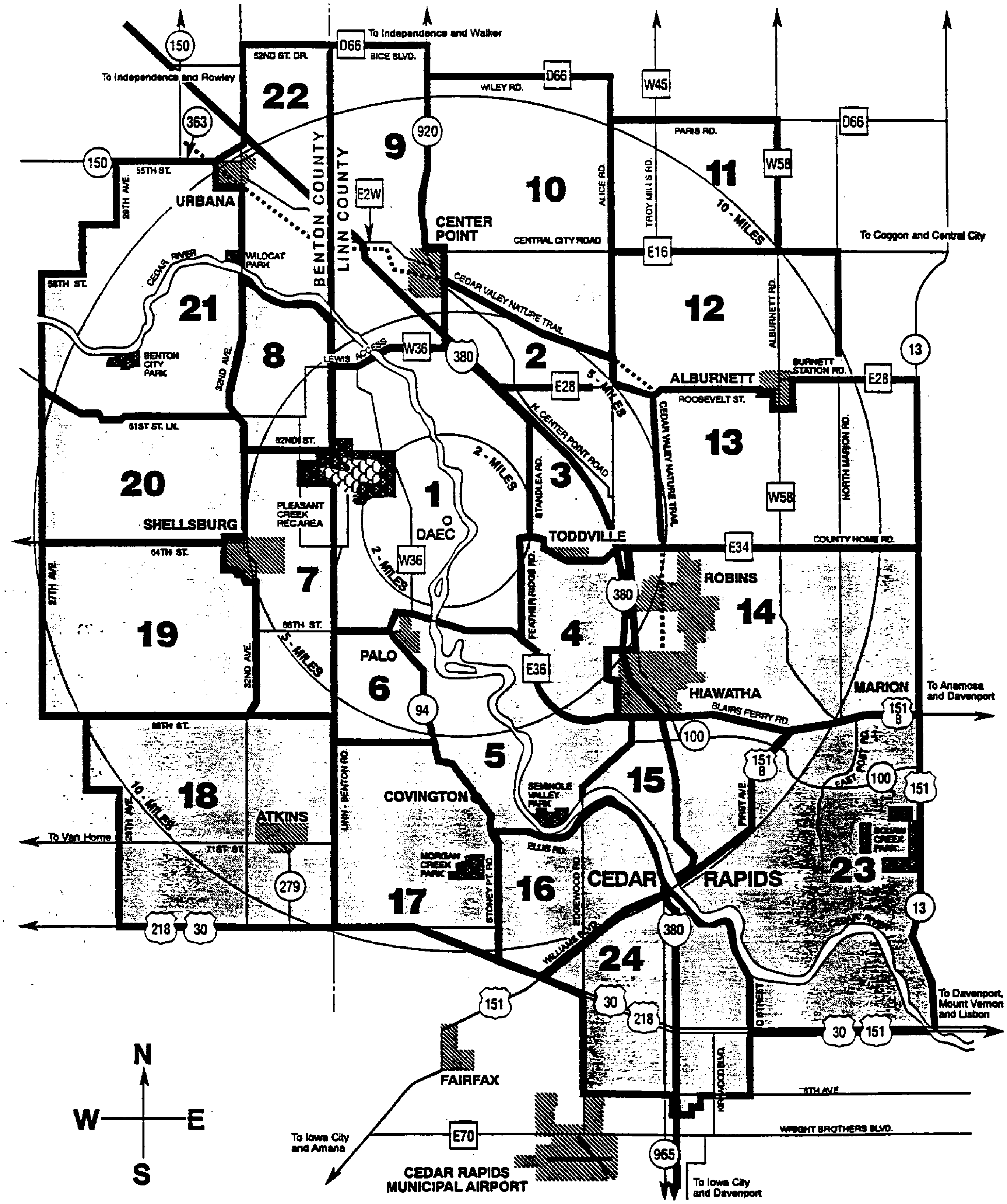
Figure 2.1-6



DUANE ARNOLD ENERGY CENTER
IOWA ELECTRIC LIGHT & POWER COMPANY
UPDATED FINAL SAFETY ANALYSIS REPORT

Site Planimetric Map Showing 6-Mile LPZ, 2-Mile EPZ, and Population Center Distance

Figure 2.1-7



DUANE ARNOLD ENERGY CENTER

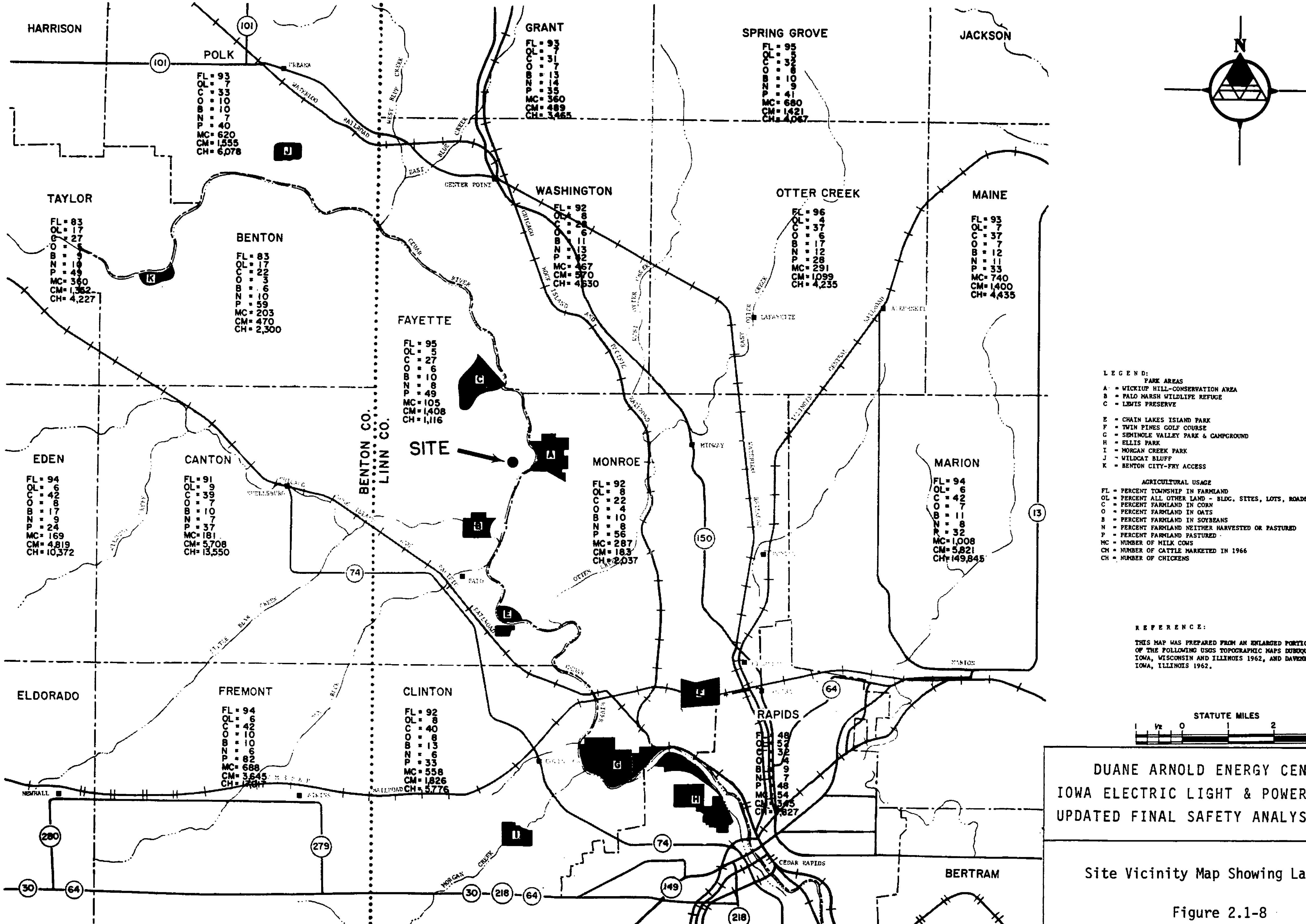
IES UTILITIES, INC.

UPDATED FINAL SAFETY ANALYSIS REPORT

DAEC Plume Exposure

Emergency Planning Zone

Figure 2.1-7a



LEGEND:

PARK AREAS

A = WICKIUP HILL-CONSERVATION AREA
B = PALO MARSH WILDLIFE REFUGE
C = LEWIS PRESERVE

E = CHAIN LAKES ISLAND PARK
F = TWIN PINES GOLF COURSE
G = SEMINOLE VALLEY PARK & CAMPGROUND
H = ELLIS PARK
I = MORGAN CREEK PARK
J = WILDCAT BLUFF
K = BENTON CITY-FRY ACCESS

AGRICULTURAL USAGE

FL = PERCENT TOWNSHIP IN FARMLAND
OL = PERCENT ALL OTHER LAND - BLDG. SITES, LOTS, ROADS, WOODS, ETC.
C = PERCENT FARMLAND IN CORN
O = PERCENT FARMLAND IN OATS
B = PERCENT FARMLAND IN SOYBEANS
N = PERCENT FARMLAND NEITHER HARVESTED OR PASTURED
P = PERCENT FARMLAND PASTURED
MC = NUMBER OF MILK COWS
CM = NUMBER OF CATTLE MARKETED IN 1966
CH = NUMBER OF CHICKENS

REFERENCE:

THIS MAP WAS PREPARED FROM AN ENLARGED PORTION OF THE FOLLOWING USGS TOPOGRAPHIC MAPS DUBUQUE, IOWA, WISCONSIN AND ILLINOIS 1962, AND DAVENPORT IOWA, ILLINOIS 1962.

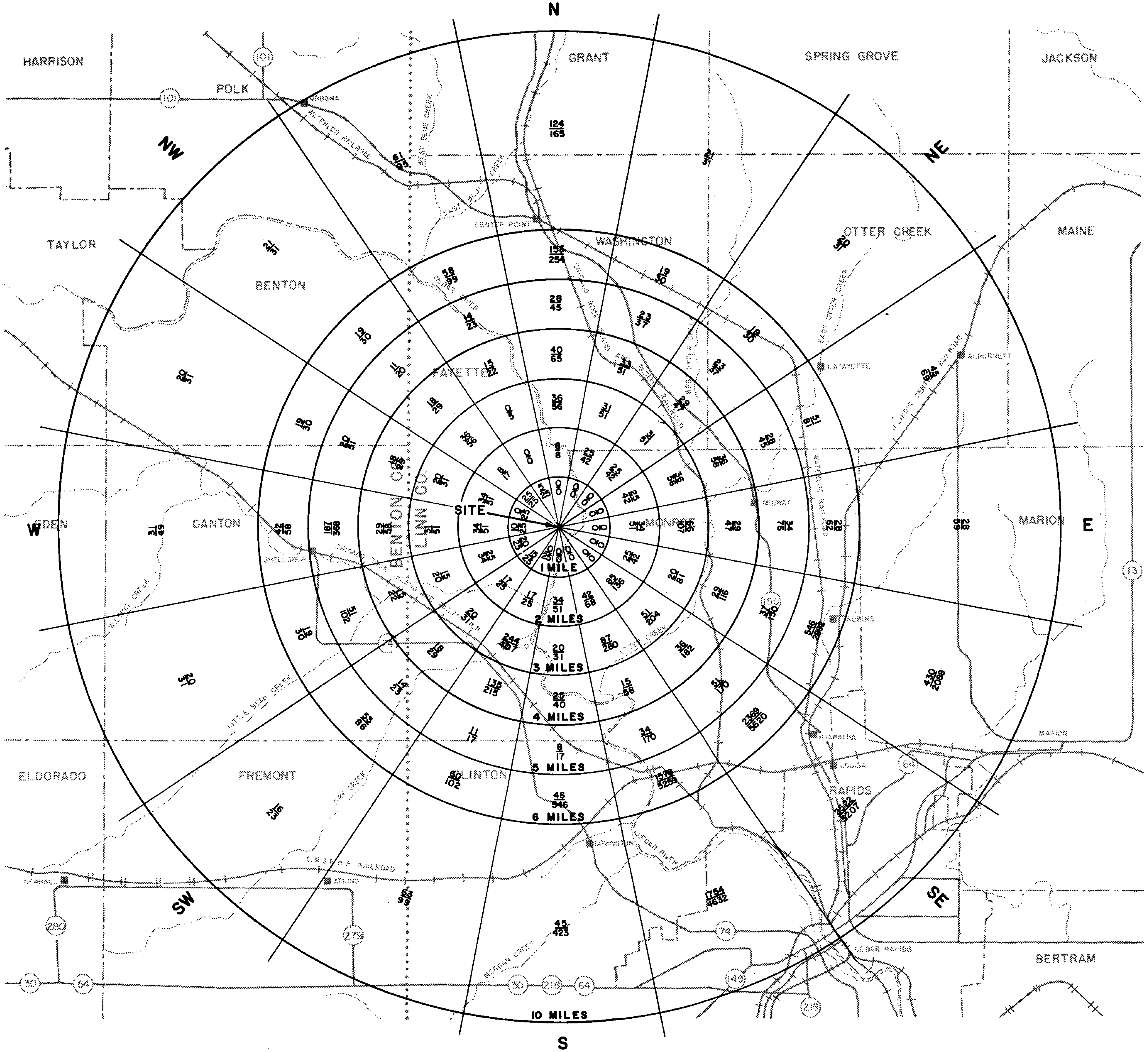
STATUTE MILES

1 ½ 0 1 2 3

DUANE ARNOLD ENERGY CENTER
IOWA ELECTRIC LIGHT & POWER COMPANY
UPDATED FINAL SAFETY ANALYSIS REPORT

Site Vicinity Map Showing Land Use

Figure 2.1-8



RADIUS IN MILES	CUMULATIVE TOTAL 1970	2010
1	5	11
2	22	35
3	36	68
4	34	67
5	35	73
6	121	342
10	252	639

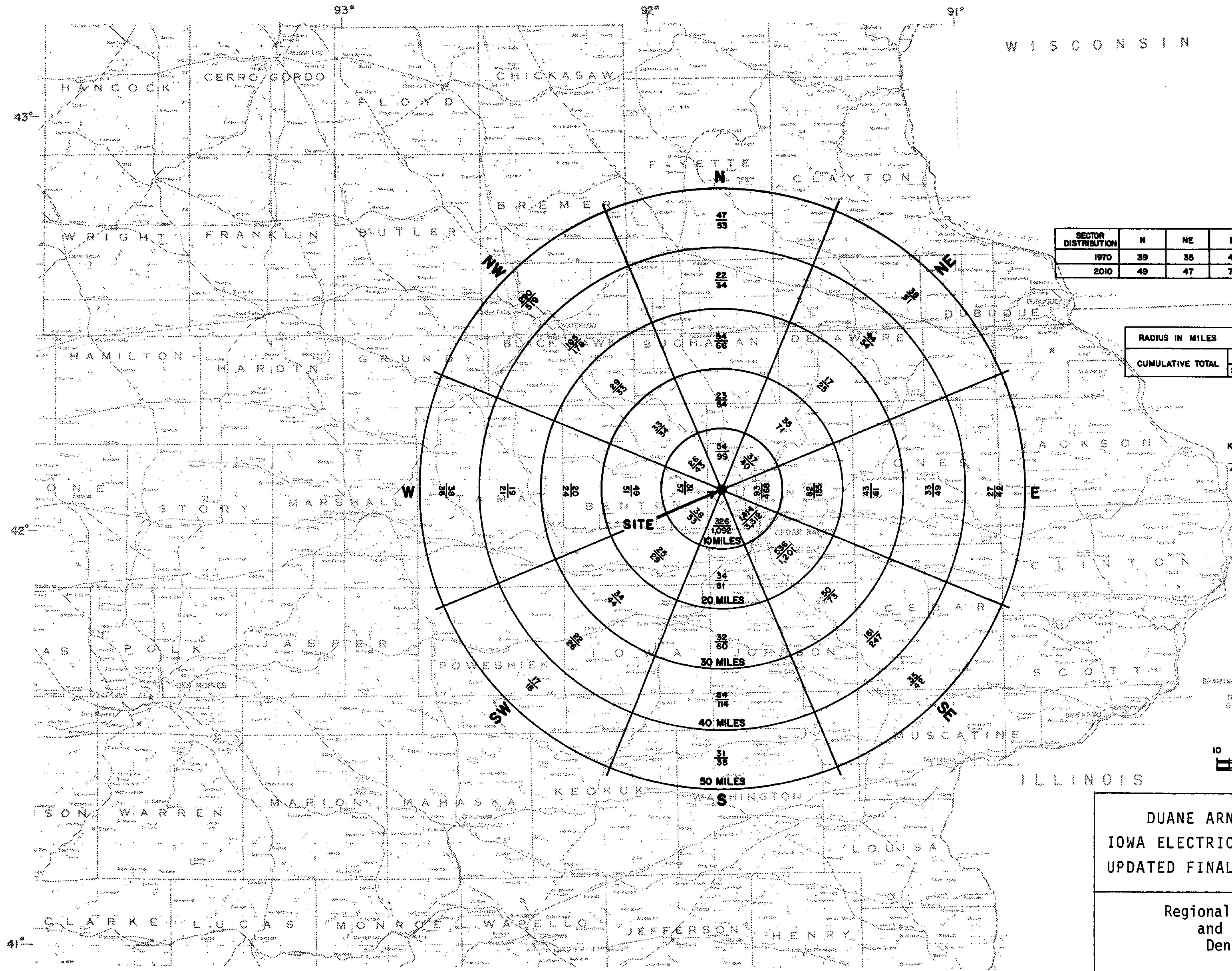
SECTOR DENSITY	1970	2010
N	104	145
NNE	22	30
NE	21	33
ENE	42	65
E	30	62
ESE	342	1,682
SE	1,887	3,993
SSE	1,306	3,578
S	39	338
SSW	70	113
SW	20	31
WSW	22	36
W	46	79
WNW	20	33
NW	20	31
NNW	48	76

KEY: $\frac{27}{70}$ = $\frac{\text{1970 POPULATION}}{\text{ESTIMATED 2010 POPULATION}}$

STATUTE MILES
1 0 1 2 3

DUANE ARNOLD ENERGY CENTER
IOWA ELECTRIC LIGHT & POWER COMPANY
UPDATED FINAL SAFETY ANALYSIS REPORT

Site Vicinity Map Showing
Present and Future Population
Density 0-10 Miles
Figure 2.1-9



SECTOR DISTRIBUTION	N	NE	E	SE	S	SW	W	NW
1970	39	35	41	189	58	24	30	144
2010	49	47	78	375	111	28	32	249

RADIUS IN MILES		10	20	30	40	50
CUMULATIVE TOTAL	1970	252	140	81	73	70
	2010	639	313	166	133	121

KEY:

$\frac{26}{42} = \frac{\text{1970 POPULATION PER SQUARE MILE}}{\text{ESTIMATED 2010 POPULATION PER SQUARE MILE}}$

DRAWING REFERENCE:

THIS MAP WAS PREPARED FROM A PORTION OF THE U.S.G.S. IOWA BASE MAP, 1966 EDITION.

STATUTE MILES

10 0 10 20

DUANE ARNOLD ENERGY CENTER
IOWA ELECTRIC LIGHT & POWER COMPANY
UPDATED FINAL SAFETY ANALYSIS REPORT

Regional Map Showing Present and Future Population Density 0-50 Miles

Figure 2.1-10

2.2 NEARBY INDUSTRIAL, TRANSPORTATION, AND MILITARY FACILITIES

2.2.1 LOCATIONS AND ROUTES

General information on the locations and routes of nearby industrial and transportation facilities is given below. There are no nearby military facilities.

2.2.2 DESCRIPTIONS

The industrial activities within 10 miles of the site are confined principally to the Cedar Rapids metropolitan area. The smaller communities in the vicinity of the site have little or no industry and consist of small retail business establishments.

2012-018 Immediately to the south of the DAEC site, is the Croell Sand Extraction Plant which began operation in 2012. Areas not under sand excavation will remain in crop production.

There are several quarries within 5 miles of the DAEC site as shown on the General Highway and Transportation Maps for Linn and Benton Counties. However, discussions with Linn and Benton County Engineers indicated that only one of these quarries was operational at the time of the initial FSAR. The other quarries had been inoperative for a number of years.

2.2.2.1 Description of Facilities

Facility descriptions are included in Section 2.2.2.2 below.

2.2.2.2 Description of Products and Materials

Industry in Cedar Rapids is typical of a progressive midwestern city of its size. Industrial density is greatest in the central and southern metropolitan areas. Manufacturing is the largest employer in Linn County. Manufacturing firms account approximately 17% of the total employment in the county.

The operational quarry currently is owned and operated by Aggregates, Inc., and is approximately 3 miles southwest of the DAEC site.

Communication with Aggregates revealed that quarry operations may involve some blasting with dynamite. Although Aggregates personnel are responsible for blasting operations, all dynamite associated with such blasting is individually ordered and is delivered to the quarry by the dynamite company. This method of operation eliminates onsite storage of dynamite at the quarry except for small quantities (5 to 10 lb) that are kept on hand for routine daily use (fracturing large boulders, etc). Although the frequency of blasting at the quarry varies considerably depending on business factors, the maximum frequency is estimated to be four times per week with an average frequency of once or twice per week.

The maximum weight of dynamite used is approximately 18,000 lb, whereas the average weight of dynamite used is approximately 10,000 lb.

John Blume & Associates, Iowa Electric's seismic consultant, evaluated the potential impact of these offsite blasting operations on the DAEC. Blume's analysis made assumptions regarding shot emplacement and soil and foundation conditions that would produce the maximum possible effect at the plant site. Based on the data presented by Johnson,[1] it was determined that the maximum particle acceleration of seismic waves experienced at the site would not exceed 0.002g. The capability exists to confirm the conservatism of the Blume analysis as the containment base slab strong-motion accelerograph is triggered at a vertical acceleration of 0.01g. The absence of SMA-2 triggering due to blasting operations at the Aggregates quarry will in effect confirm the insignificance of such operations relative to plant safety considerations.

The nature of the operation at this quarry together with Blume's analysis of effects at the DAEC site and the fact that the quarry had operated for approximately 25 years without incident (at the time of the initial FSAR) would indicate that the potential hazard from this source is insignificant.

2.2.2.3 Pipelines

There are no known mineral mines or petroleum wells located within 5 miles of the plant site.

There are two 4” diameter natural gas mains belonging to Alliant Energy located no less than 2 miles from DAEC at their closest points. A 12” diameter natural gas main belonging to MidAmerican Energy Company is located approximately 8 miles from the DAEC at its closest point. The hazard analysis contained in Reference 2 determined the gas pipelines around the DAEC site present negligible risk to the safe operation of DAEC.

2.2.2.4 Waterways

In the vicinity of the site, the Cedar River is not navigable. The closest boat landing is at the Chain Lakes public access, approximately 3 miles downstream of the site.

2.2.2.5 Airports

The nearest airfield used for commercial traffic is the Cedar Rapids Municipal Airport located 15 miles south-southeast of the site, in accordance with Standard Review Plan Section 3.5.1.6, airports which have fewer than $1000d^2$ flight operations per year, where d is the distance in miles from the site, are screened from further analysis. At a distance of 15 miles this equates to 225,000 flight operations per year. This number is more than twice the estimate of annual flight operations from the Cedar Rapids Municipal Airport. Therefore, the Cedar Rapids Municipal Airport is justifiably screened from further analysis.

A small private landing strip located 4.5 miles southeast of the site is no longer active. Other airports in the vicinity can also be screened from further analysis as they are encompassed by the potential hazard posed by the Cedar Rapids Municipal Airport.

At the time of the initial FSAR, the private airport near Shellsburg was designated on the most recent edition of the Dubuque Sectional Aeronautical Chart and also on the 1968 USGS topographical map, Shellsburg Quadrangle. Investigation and discussion with Cedar Rapids area aviation organizations indicated that a private restricted airstrip was operated at one time at the above mentioned location by an individual who owned a single light airplane, but the owner had dismantled the hangar and later died. The airstrip was no longer in use, but was still designated on the aeronautical charts because apparently no cancellation had been initiated. During the course of the investigation, it was determined that another small landing strip not shown on the most recent aeronautical charts existed at a location approximately four miles southeast of the plant. Discussion with the airfield manager indicated that approximately 10 single engine light airplanes were stationed at the strip. The maximum gross weight of these aircraft was estimated to be 3000 lb. Runway orientation was north-south (3000 ft turf) and east-west (2600 ft turf) according to the "Iowa Airport Directory 1972-73," published by the Iowa Aeronautical Commission. The facility was an uncontrolled field and accordingly there was no record kept of the number of takeoffs and landings. An estimate of the number of takeoffs and landings was made based upon the assumption that between May 1 and November 1 each of the 10 planes averaged four movements per weekend. During the winter months it was estimated that four of the planes were active on six weekends, again at an average of four movements per weekend. These assumptions were felt to be conservative and were based upon discussions held with the owner. Accordingly, it was conservatively estimated that there were less than 1500 movements per year from this airfield. The airfield owner indicated that there were no plans for significantly expanding the scope of operations at this facility. At the time of initial FSAR, no other airfields were known to exist within 5 miles of the DAEC.

2.2.2.5.1 Military Aviation

There are no nearby military facilities. In addition, the closest military operations areas is more than 100 miles distant from the site. Military air activity does not represent a significant hazard to the plant.

[redacted]

2.2.2.6 Projections of Industrial Growth

See Section 2.4.11.

2.2.3 EVALUATION OF POTENTIAL ACCIDENTS

2.2.3.1 Determination of Design Basis Events

Design basis events are discussed in Section 2.2.3.2 below.

2.2.3.2 Effects of Design Basis Events

Rail, chlorine, river, and aircraft hazards are discussed in the following subsections.

2.2.3.2.1 Commercial Rail Line

The closest commercial rail line to the DAEC that could conceivably carry hazardous material on a routine basis is approximately 3.5 miles west of the DAEC site. The potential consequences of incidents involving such material are not felt to be significant in terms of plant safety. No such hazardous material is manufactured in the vicinity of the site.

2.2.3.2.2 LPG Distribution Facility

An LPG storage and distribution facility is located on the west edge of the town of Palo approximately 3.5 miles from the site. Potential consequences of incidents at this facility are felt to be insignificant in terms of plant safety.

2.2.3.2.3 Chlorine

Chlorine, primarily in liquid form, was used in the condenser cooling system and was stored in twelve 1-ton containers in the onsite pump house. In 1982, this chlorine was removed from the site in order to eliminate the potential hazard of a chlorine spill to control room personnel. Sodium hypochlorite was substituted for circulating water treatment. See Sections 10.4.5.2 and 9.2.4.

2.2.3.2.4 Liquid Spills

The nearest location upstream that could be the source of significant amounts of corrosive liquids or oil is the City of Vinton, which is about 30 river miles upstream. The City of Vinton stores diesel oil for its electric power generating station in three 15,000-gal tanks. A dike surrounds these three tanks and is designed to retain the contents of any one tank. Therefore, for oil to be released into the river, more than one tank or one tank and a fault to the dike would be required.

An interview with local personnel from the U.S. Geological Survey indicates that the time required for material to transit the river from Vinton to the plant site would not be less than 24 hr. Therefore, any significant discharge into the river would be accompanied by a significant amount of time in which to take necessary action to mitigate the consequences of any discharge. Discharges upstream of Vinton would be accompanied by more time in which to take action.

If an upstream discharge were to take place, it can be assumed that cognizant authorities would be mobilized to take action to mitigate the ecological consequences. These actions would tend to lessen the concentrations in the site vicinity.

The action to be taken on the receipt of warning that an upstream discharge had taken place would be to reduce plant power to hot standby. This would reduce evaporative losses to essentially zero deleting the requirement for makeup or blowdown. With no river input, there will be no effects to the plant cooling systems. This condition can be maintained until long after river water with contaminants has passed the intake structure.

2.2.3.2.5 Aircraft Accidents

At the time of the initial FSAR the potential for aircraft accidents at the facility was considered extremely improbable due to the relatively small number of airplane movements at the small landing strip described in Section 2.2.2.5.

[redacted]

[redacted]

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1. U. S. Army Engineer Nuclear Cratering Group, NCG Technical Report No. 31, Explosive Excavation Technology, TID-4500, UC-35, Livermore, California, 1971.

2. IES Utilities Inc., Individual Plant Examination of External Events, Transportation and Nearby Facility Hazards Evaluation of the DAEC Site, December 1995.

2.3 METEOROLOGY

2.3.1 REGIONAL CLIMATOLOGY

General climatology for the area has been evaluated from National Weather Service (formerly U.S. Department of Commerce, or more recently, ESSA, Weather Bureau) sources.

2.3.1.1 General Climate

2.3.1.1.1 Temperature and Precipitation

2.3.1.1.1.1 Temperature

Based on a 30-year summary of records[1] for the period 1931-1960, Cedar Rapids, Iowa, experienced the following temperature regimes:

Highest daily maximum	109°F, July 1936
Lowest daily minimum	-25°F, January 1936
Mean number of days 0°F and below (per month)	January 6 February 4 March 1 November + (less than 1/2 day) December 3

The normal daily maximum temperature for the year is 59.7°F, and the normal daily minimum temperature is 38.7°F for the year. The average temperature is 49.2°F.

The maximum temperature of record occurred on July 5, 1911, and was 110°F. The minimum temperature of record occurred in January 1883 and was -36°F.

2.3.1.1.1.2 Precipitation

About 70% of Cedar Rapids' annual precipitation of 33.27 in. falls between April and September. The greatest daily precipitation fell on September 15, 1914 when 7.78 in. fell. In a 24-hr period, July 16 and 17, 1968, 9.31 in. of rain fell at Waterloo, Iowa, which is about 50 miles northwest of the site. Table 2.3-1 indicates the frequency of maximum rainfall by various time intervals at Cedar Rapids, Iowa. Maximum monthly rainfall is on the order of 12 to 13 in.

Snowfall averages about 31 in. per season. It has varied from 8.1 in. in 1921-1922 to as much as 62.4 in. in the 1959-1960 season. Previously, the snowfall was above 60 in. in the 1904-1905 winter period. The heaviest daily snowfall since 1930 was 16.7 in., which occurred on February 26, 1954.

In January and February of 1929, snow accumulation reached a record depth of 16 in. However, during most winters the maximum depth does not exceed 8 in. The ground is snow covered an average of 63 days per year.

The maximum rainfall rates[2] for the site vicinity are as follows:

Time	Amount of Rain (in.)
5 min	0.72*
10 min	1.15
15 min	1.46
30 min	2.05
1 hr	2.28
2 hr	3.00
3 hr	4.06
6 hr	4.85
12 hr	5.33

2.3.1.1.2 Severe Weather

2.3.1.1.2.1 Thunderstorms

Reference 1 states that there are normally about 48 thunderstorms per year, about half of which occur in the 4 months between May and August.

Thunderstorm frequency per month[3] is as follows:

Month	Thunderstorm per Month
January	+**
February	+**
March	2
April	4
May	7
June	9
July	8
August	7
September	5
October	3
November	1
December	+**

* Average of maxima for Dubuque and Davenport, Iowa. Some of these records are in excess of those cited in Table 2.3-1.

** The symbol + indicates a mean value less than 1.

The statistics above are based on approximately 70 years of record and are the higher of the two closest stations summarized, Dubuque and Davenport, Iowa.

2.3.1.1.2.2 Tornados.

There are about 20 tornados per year that are observed in Iowa on approximately 10 tornado days. The mean annual frequency of tornados in 1953-1962 for the 1-degree latitude-longitude square containing the site was 1.1.[4] On the basis of the technique shown,[4] the probability of occurrence of a tornado in the site vicinity is 8.5 x 10^{-4}, and the recurrence frequency interval is one tornado every 1171 years.

There has been a slight tendency for increasing tornado frequency in the most recent years.[5] Sufficient data have not been obtained as yet to establish any statistically significant trend from the long-term normals.

2.3.1.1.3 Winds

Preliminary estimates of wind speeds and directional persistence were presented in the PSAR from National Weather Service Observations at Cedar Rapids and Waterloo, Iowa. Since these data are of primary interest in determining atmospheric diffusion, they are now covered by Section 2.3.4, where special attention is given to the 12 months onsite data.

2.3.1.2 Regional Meteorological Conditions for Design and Operating Bases

2.3.1.2.1 Severe Weather Phenomena

The occurrence of fog, hail, and ice storms in the vicinity of the DAEC site was investigated using hourly meteorological data collected at the Des Moines Municipal Airport. The Des Moines Municipal Airport is located south of the city, about 100 miles west-southwest of the plant site. Meteorological data from Des Moines were used because these observations are the most comprehensive set of long-term data that are generally representative of the plant site meteorology. (The FAA Station at the Cedar Rapids Municipal Airport does not provide a complete tabulation of the meteorological parameters required for this climatological study.)

The Des Moines data used consist of observations made during a 5-year period (December 1959 through November 1964). The selection of the particular period to be used is normally arbitrary, since anomalous meteorological occurrences would tend to be averaged out in a climatological record of a 5-year duration. The 1959-1964 period was chosen because after January 1965, meteorological observations at Weather Bureau stations began to be tabulated for 3-hour intervals; for a study of this nature, it is preferable to use hourly meteorological observations. The data were reported in accordance with the Weather Bureau standard WBAN form and were obtained on magnetic tape. An extensive computer reduction of data was therefore possible. Approximately 43,000 hourly data records were obtained.

2.3.1.2.2 Frequency of Occurrence and Intensity of Hail, Ice Storms, and Fog

1. Hail. Hail is reported on the WBAN forms using the following categories:

   a. Light and moderate hail.
   b. Light and moderate small hail.
   c. Heavy hail.
   d. Heavy small hail.

To facilitate the data search, the hail categories were broken up into two intensity groups as follows:

Group I
Light and moderate hail
Light and moderate small hail

Group II
Heavy hail
Heavy small hail

For the Group I category, only three cases were recorded for the 5-year period as follows:

Case	Pasquill Stability Class	Average Wind Speed (knots)
1	D	10
2	D	20
3	D	24

All cases occurred in the spring season.

For the Group II category, there were no occurrences during the 5-year period.

An additional source of information on Iowa hail is contained in Reference 6, as follows:

Number of Days with Hail Based on Dubuque, Iowa Data (40 years)

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Total
0	1	12	29	25	18	6	7	5	5	4	0	112

Probable hail size distribution is the following:

Size	Percentage
Grain	3.0
Currant	22.1
Pea	44.5
Grape	19.4
Walnut	6.1
Golf ball	4.3
Tennis ball	0.6
	100.0

2. Ice Storms. Ice storms are reported on the WBAN forms using the following categories:

a. Light and moderate freezing rain.
b. Freezing drizzle.
c. Sleet or sleet showers.
d. Heavy freezing drizzle.
e. Heavy freezing rain.

To facilitate the data search, the ice storm categories were broken up into two intensity groups as follows:

Group I
- Light and moderate freezing rain
- Light and moderate freezing drizzle
- Sleet or sleet showers

Group II
- Heavy freezing drizzle
- Heavy freezing rain

For the Group I category, seasonal data for the 5-year period are as follows:

Spring

Number of Occurrences	Pasquill Stability Class	Average Wind Speed (knots)
28	D	12.8
1	E	5.0

The dominant wind direction for the 29 occurrences was east-northeast with 8 cases. The lowest wind speed observed was 5 knots.

Summer

No occurrences

Fall

Number of Occurrences	Pasquill Stability Class	Average Wind Speed (knots)
13	D	16.6

The dominant wind direction was from the northwest.

Winter

Number of Occurrences	Pasquill Stability Class	Average Wind Speed (knots)
13	C	2.7
155	D	10.0
36	E	4.9
21	F	2.8
225		

Summary (all seasons)

Number of Occurrences	Pasquill Stability Class	Average Wind Speed (knots)
13	C	2.7
196	D	10.9
37	E	4.9
21	F	2.8
267		

Of the 225 winter occurrences, only 15 occurrences during the 5-year interval were of greater duration than 2 hours. The persistence associated with these 15 occurrences is as follows:

Number of Occurrences	Persistence (hr)
6	3
7	4
1	5
1	10
15	

For the single 10-hr persistence recording, the associated average wind speed was 7.3 knots.

Pasquill stability classes in all cases were D and E.

For Group II (heavy freezing drizzle, heavy freezing rain), there was only a single occurrence during the 5-year period. This was a winter occurrence and was associated with Pasquill Stability Class D and 15-knot winds.

Additional data on ice storms are contained in Reference 7, as follows:

Number of Days with Freezing Precipitation
(1939-1948 Data)

	Dubuque	Des Moines
Nov	1	8
Dec	13	35
Jan	6	20
Feb	7	16
Mar	11	12
	38	91

It should be noted that the frequency data associated with this report are expressed in number of days with freezing precipitation whereas the Des Moines Municipal Airport data are expressed in number of hourly occurrences; hence the results cannot be directly compared.

The significantly lower frequency at Dubuque appears to be associated with a trend to a progressively lower frequency of occurrence of ice storms at locations eastward from Des Moines. Accordingly, the Des Moines data should be conservative for application to the Palo site.

3. Fog

The total hours of observed fog at the Des Moines Airport Weather Station are summarized by month in Table 2.3-2 for the 5 years of data examined. On the average, about 50% of all the hours of natural fog occurrence will occur during the winter third of the year (December through March). Table 2.3-2 also shows that fog is least likely to occur during the warmer months of the year. The average frequency of occurrence of fog is 961 hr/yr or 10.9%.

The dependence of fog occurrence on stability category is illustrated by the annual summary presented in Table 2.3-3. For the data shown in Table 2.3-3, it is seen that fog tends to occur most frequently during neutral conditions (Stability Class D), and is least likely to occur during unstable conditions (Stability Class B). For those cases where fog was recorded, 85.6% occurred under neutral conditions.

Average wind speeds associated with each stability class shown in Table 2.3-3 are as follows:

Pasquill Stability Class	Average Wind Speed (knots)
A	--
B	3.5
C	5.2
D	9.0*
E	5.9
F	4.5
G	2.8

In order to evaluate fog intensity, a separate computer search was conducted for fog hours associated with less than 1/8-mile visibility. Out of the total 43,000-hr record, only 223 hr were associated with visibility less than 1/8 mile for a probability of 5.2×10^{-3}. Stability classes associated with these low-visibility fogs were skewed to more stable conditions--23.8% Pasquill Stability Class E with 5.2-knot average wind speeds and 16.1% Pasquill Stability Class F with 2.8-knot average wind speeds. The dominant stability regime was again neutral with 60% of low-visibility fog hr associated with Pasquill Stability Classes C and D.

2.3.1.2.3 Ultimate Heat Sink

See Section 1.8.27 for a discussion of this topic.

* Average of daytime and nighttime values.

2.3.2 LOCAL METEOROLOGY

An examination of weather records from nearby weather stations indicates that the climatology of the site is continental in nature. Average annual rainfall is 33.27 in., with 70% of the rainfall occurring during the months of April through September. Maximum monthly rainfall is 12 to 13 in. Snowfall averages 31 in. per season.

Severe weather is characterized by thunderstorms, which occur predominantly in the months of May through August, and tornados. There are about 20 tornados per year observed in Iowa. Seismic Category I structures are designed to withstand a tornado with a maximum tangential wind velocity of 300 mph, a transverse velocity of 60 mph, and an external vacuum of 3.0-psi gauge developed within 3 sec.

The mean wind speed at the site, based on 12-month data, is 3.6 m/sec at 33 ft and 5.3 m/sec at 156 ft. Based on the same 12-month data, the most frequent winds are from the south at both levels with a secondary maximum from north-northwest.

Condensed meteorological summaries are shown in Tables 2.3-4 through 2.3-8 for the period January 8, 1971, to January 7, 1972. Onsite data recovery for the year was equal to or greater than 92.8% for all parameters.

From these tables, it would appear that a Pasquill F stability category and a 1 m/sec wind are conservative meteorological parameters to use as input to accident analyses.

In support of conversion to the 10 CFR 50.67 Alternate Source Term, DAEC performed a new meteorological data assessment using two years of data collected from January 1, 1997 through December 31, 1999. This assessment is described in the Reference 8 amendment request. The assessment included use of the PAVAN and ARCON96 codes to derive new X/Q values. In Reference 9, the NRC granted a partial scope amendment approving the use of the alternate source term for the fuel handling accident. In Reference 10, the NRC granted the full scope implementation for the alternate source term methods for Loss of Coolant Accident (LOCA), Main Steam Line Break (MSLB) and Control Rod Drop Accident (CRDA). The CRDA analysis was updated later and approved in Reference 11.

2.3.3 ONSITE METEOROLOGICAL MEASUREMENTS PROGRAM

An instrumented meteorological tower 1700 ft south-southeast of the reactor building (1125 ft southeast of the offgas stack) has been in operation since January 10, 1971.

The meteorological system was upgraded in 1985 with redundant instrumentation to make meteorological measurements with more reliability and provide meteorological data input to the emergency response plume model. Meteorological variables that are measured and/or calculated are displayed in the control room on a paperless recorder for use during plant operation and are input to the safety parameter display system (SPDS) computer system for input to the emergency response plume model and for historical data recording. The meteorological system is powered from two separate power sources through an automatic transfer switch. Primary power is supplied from an essential bus, and backup power is supplied from a lighting

distribution panel. System performance requirements are such as to ensure at least an annual 90% joint data recovery for the individual meteorological parameters.

Redundant (primary and backup) parameters measured are wind speed, wind direction, temperature, and wind directional variability at both 156 and 33 ft above grade, and temperature difference between 156 and 33 ft. Additional variables include dewpoint at 33 ft and precipitation at grade level. Plant grade (base of reactor building) has been raised by fill to a level of 12 ft higher than the base of the meteorological tower. This fill does not interfere aerodynamically with flow around the lower wind sensor because of the distances involved (fill is 700 ft from base of tower). It does mean the wind sensor is 21 ft higher than reactor grade but 33 ft above the base of the tower and flood plain.

In 1987, the method of mounting the wind speed and direction sensors was modified. The modification consisted of providing separate booms, 180 degrees apart, for the redundant instruments. The primary instrument boom is oriented to the west and the backup boom is to the east. Tower wake effects are thereby minimized for the most prevalent wind directions.

For the meteorological system, specific ranges and system accuracies of time-averaged values by parameter are

1. Wind speed.

   ±0.5 mph (±0.22 m/sec) for speeds less than or equal to 25 mph (11.13 m/sec) and ±1.0 mph (±0.44 m/sec) for speeds greater than 25 mph. Range 0 to 100 mph.

2. Wind direction.

   ±5 degrees of azimuth. Range 0 to 540 degrees.

3. Air temperature.

   ±0.9°F (±0.5°C). Range -40°F to 120°F (accuracy maintained from -40°F to 107.6°F).

4. Temperature difference.

   ±0.22°F for the 37.5-m height difference of the DAEC meteorological tower levels. Range -9°F to 18°F relative to temperature at the 33-ft level.

5. Dewpoint.

   ±2.7°F (±1.5°C) for relative humidity greater than 60% and temperature between -22°F and 86°F (-30°C and 30°C) and ±4.5°F (±2.5°C) for conditions outside of the above range. Range -40°F to 120°F (accuracy maintained from -40°F to 107.6°F).

6. Precipitation.

   Measured by recording rain gauge with an accuracy of recorded value ±10% of total catch. Resolution of 0.01 in. Handles rainfall rates up to 2 in./hr.

7. Other characteristics include

   1. Wind speed sensor starting threshold.

      <1 mph (0.45 m/sec).

   2. Wind direction sensor starting threshold.

      <1 mph (0.45 m/sec).

   3. Wind direction sensor damping ratio.

      0.4 to 0.6 inclusive with deflection of 15 degrees and delay distance not to exceed 2m.

      (The delay distance is defined as the distance that air flowing past a wind vane moves while the vane is responding to 50% of the step change in wind direction.)

   4. Wind direction variability, sigma theta ($\sigma\theta$), the standard deviation of the horizontal wind direction fluctuations, is provided and is calculated from 180 instantaneous values of lateral wind direction during the 15-min recording period. The standard deviation calculator is able to accept wind direction input of 0 to 540 degrees and produces an output proportional to standard deviations of 0 to 100 degrees.

Observations are averaged and recorded on a digital recorder located in the reactor control room and on the computer disk storage associated with the plume model software.

From previous average half-hourly (one per hour) observations, seasonal and annual summaries have been prepared for both the 33- and 156-foot levels as follows.

Joint frequency distribution of wind speed and direction by stability class was determined by the 33- to 156-ft temperature difference (ΔT) in accordance with the table, given below:

Pasquill Class	ΔT (°C/100 m)
A	$\Delta T \leq -1.9$
B	$-1.9 < \Delta T \leq -1.7$
C	$-1.7 < \Delta T \leq -1.5$
D	$-1.5 < \Delta T \leq -0.5$
E	$-0.5 < \Delta T \leq +1.5$
F	$+1.5 < \Delta T \leq +4.0$
G*	$+4.0 < \Delta T$

Joint frequency distribution of wind speed and direction by stability class was determined by directional variability ($\sigma\theta$) in accordance with the table below:

Pasquill Class	$\sigma\theta$
A	$\sigma\theta \geq 22.5$
B	$17.5 \leq \sigma\theta < 22.5$
C	$12.5 \leq \sigma\theta < 17.5$
D	$7.5 \leq \sigma\theta < 12.5$
E	$3.8 \leq \sigma\theta < 7.5$
F	$2.1 \leq \sigma\theta < 3.8$
G	$\sigma\theta < 2.1$

Wind directional persistence is by 22.5-degree sectors.

2.3.4 SHORT-TERM DIFFUSION ESTIMATES

The radiological effects of design-basis accidents are considered in Chapter 15. The Gaussian diffusion model used in these calculations is also described in Chapter 15, as are the meteorological diffusion evaluation methods.

In addition, the radiological effects calculations are repeated using the assumptions of AEC-DRL embodied in AEC Safety Guide 3 and Safety Guide 5. The meteorological diffusion assumptions inherent in these safety guides are listed in Section 1.8 for the loss-of-coolant accident (LOCA) and steam-line-break accident.

The field meteorological program has been designed to provide data to confirm that the diffusion parameters inherent in the assumptions used in these meteorological models are conservative.

* It is Iowa Electric's opinion that a G stability classification is made arbitrarily simply to reflect a category "worse than F" when F has a high frequency of occurrence. Iowa Electric knows of no sound technical basis or experimental data that would justify the assignment of smaller σ values to G than to F. In reality, a G classification determined by ΔT greater than 4.0°C/100 m, is more apt to produce a σ_y value similar to an A category because of plume meander. The G category is, therefore, included only to facilitate regulatory review.

Frequency distributions of annual X/Q values have been compiled by computing X/Q each hour for 1 year using a "split-sigma" approach (i.e., σ_y was determined from $\sigma\theta$ and σ_z was determined from the concurrent ΔT between 156 and 33 ft. For the steam-line break, the 33-ft wind readings were used, and for the LOCA and other elevated releases, the 156-ft wind readings were employed. Since the offgas stack is twice the height of the meteorology tower, the 156-ft wind and the 156- to 33-ft temperature difference will be conservative, showing a higher frequency of low wind speeds and stable atmospheric conditions than probably actually occurs at the stack release point.

In support of conversion to the 10 CFR 50.67 Alternate Source Term, DAEC performed a new meteorological data assessment using two years of data collected from January 1, 1997 through December 31, 1999. This assessment is described in the Reference 8 amendment request. The assessment included use of the PAVAN and ARCON96 codes to derive new X/Q values. In Reference 9, the NRC granted a partial scope amendment approving the use of the alternate source term for the fuel handling accident. In Reference 10, the NRC granted full scope amendment implementing alternate source term methodology for Loss of Coolant Accident (LOCA), Main Steam Line Break (MSLB) and Control Rod Drop Accident (CRDA). The CRDA analysis was updated later and approved in Reference 11.

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8. Letter from Gary Van Middlesworth (NMC DAEC) to Office of Nuclear Reactor Regulation “Technical Specification Change Request (TSCR-037): Alternative Source Term” dated October 19, 2000.

9. Letter from Darl S. Hood (NRC) to Gary Van Middlesworth (NMC DAEC) “Duane Arnold Energy Center – Issuance of Amendment Regarding Secondary Containment Operability During Movement Irradiated Fuel and Core Alternations (TAC No. MB1569)” dated April 16, 2001.

10. Letter from Brenda Mozafari (NRC) to Gary VanMiddlesworth (NMC DAEC) “Duane Arnold Energy Center – Issuance of Amendment Regarding Alternative Source Term (TAC No. MB0347) dated July 31, 2001.

11. Letter from Richard Ennis (USNRC) to Gary Van Middlesworth (FPL Energy), “Duane Arnold Energy Center – Issuance of Amendment Regarding Elimination of Main Steam Line Radiation Monitor Trip Function (TAC NO. MC8883),” November 15, 2006.

Table 2.3-1

FREQUENCY (Years) OF MAXIMUM RAINFALL (In.)
BY VARIOUS TIME INTERVALS
AT CEDAR RAPIDS, IOWA

Return Period (years)	30 Min	1Hr	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
1	1.0	1.3	1.6	1.7	2.0	2.3	2.7
2	1.3	1.6	1.8	2.0	2.3	2.8	3.2
5	1.6	2.0	2.3	2.5	3.0	3.5	4.0
10	1.8	2.2	2.7	2.9	3.4	4.0	4.7
25	2.0	2.6	3.0	3.3	3.9	4.5	5.2
50	2.2	2.8	3.3	3.7	4.3	5.0	5.9
100	2.5	3.1	3.8	4.0	4.9	5.8	6.6

Table 2.3-2

OBSERVED HOURS OF FOG OCCURRENCE BY MONTH
AND YEAR FOR THE DES MOINES MUNICIPAL AIRPORT

Month	1959	1960	1961	1962	1963	1964	Avg.
Jan	--	242	61	33	139	60	107
Feb	--	103	140	212	149	78	136
Mar	--	138	134	194	170	30	133
Apr	--	43	58	54	53	59	53
May	--	37	65	57	110	30	60
June	--	106	42	59	10	44	52
July	--	46	55	80	49	37	53
Aug	--	26	80	50	76	19	50
Sept	--	63	89	24	121	99	79
Oct	--	72	45	117	20	42	59
Nov	--	52	113	50	69	71	71
Dec	238	33	189	38	41	--	108
Total	---	961	1071	968	1007	---	961

Table 2.3-3

ANNUAL FREQUENCY OF OCCURRENCE OF ATMOSPHERIC STABILITY CONDITIONS FOR THE DES MOINES MUNICIPAL AIRPORT
(Fog Observations)

Stability Index	A	B	C	D	E	F	G
Day	0.02	0.73	3.16	45.34			
Night				40.25	4.35	4.20	1.95

Table 2.3-4

FREQUENCY DISTRIBUTION OF STABILITY CATEGORIES FOR ALL WIND DIRECTIONS
(January 8, 1971-January 7, 1972)

Stability	Frequency as determined by:		
Class	ΔT (156-33 ft)	$\uparrow$(33 ft)	$\uparrow$ (156 ft)
A	3.2	7.9	2.2
B	2.7	7.6	3.4
C	5.1	18.2	12.3
D	42.5	32.5	31.2
E	29.7	21.1	28.4
F	7.2	9.2	15.5
G	9.6	4.0	7.0

Table 2.3-5

ANNUAL WINDS EQUAL TO STATED VALUES
(33-ft Level)
(January 8, 1971-January 7, 1972)

Stability	m/sec			
(↙↑ category)	0.1-0.5	0.6-1.0	1.1-1.5	1.6-2.0
A	60	124	95	77
B	25	61	80	73
C	43	103	124	161
D	45	112	158	210
E	40	72	112	146
F	33	66	72	69
G	88	87	62	22

Total observations = 8347

5% = 417

Summation of occurrences within lower left corner = 376, hence "worst" condition is equal to or better than F stability and 1.0 m/sec.

Table 2.3-6

MAXIMUM PERSISTENCE (Hr) BY WIND DIRECTION
(22.5▪ Sector) ALL STABILITY CATEGORIES
(January 8, 1971-January 7, 1972)

Direction	33 Ft	156 Ft
NNE	9	13
NE	7	5
ENE	6	8
E	7	5
ESE	9	8
SE	8	8
SSE	13	15
S	16	16
SSW	6	7
SW	8	8
WSW	5	4
W	10	10
WNW	11	9
NW	10	14
NNW	12	11
N	11	14

Table 2.3-7

AVERAGE WIND SPEED (156-ft Level) BY STABILITY CATEGORY
(Determined by ΔT)
(January 8, 1971-January 7, 1972)

	m/sec				
Category	Spring	Summer	Fall	Winter	Annual
A	8.7	5.0	5.3	6.4	5.9
B	5.8	4.7	5.3	9.8[a]	5.2
C	5.4	4.4	5.1	5.4[a]	4.9
D	5.7	4.1	5.2	5.1	5.5
E	6.5	3.8	4.9	5.4	5.1
F	3.2	3.0	3.4	4.6	3.5
G	2.3	2.1	2.3	3.1	2.3

[a]Less than obs.

Table 2.3-8

ANNUAL WINDS EQUAL TO STATED VALUES (156-ft Level)
(January 8, 1971-January 7, 1972)

Stability	m/sec			
(ΔT category)	0.1-0.5	0.6-1.0	1.1-1.5	1.6-2.0
A	1	3	2	3
B	0	2	2	4
C	0	5	11	21
D	25	48	111	139
E	28	49	56	112
F	18	28	39	49
G	73	80	87	114

Total observations = 8134

5% = 407

Summation of occurrences within lower left corner of matrix = 314. Hence 5% "worst" value is equivalent or slightly better than F stability and 1.0 m/sec wind speed.

2.4 HYDROLOGIC ENGINEERING

2.4.1 HYDROLOGIC DESCRIPTION

The site is on the west bank of the Cedar River, 133.5 river miles above its confluence with the Iowa River. Figure 2.4-1 depicts the site location relative to the drainage basin above Cedar Rapids and also shows the location of stream gauging stations pertinent to hydrologic studies.

The Cedar River is the largest tributary of the Iowa River. Drainage area at the mouth is 7819 mi^2, 1024 of which are in Minnesota. Drainage area above the plant site is approximately 6250 mi^2. Basin topography is variable and typical of central Iowa farm country. The Cedar River flood plain is also variable but in general ranges from relatively narrow valley slopes to broad plains 3 to 4 miles wide. These topographic characteristics have a marked effect on flood peaks, with valley storage tending to reduce flood waves as they proceed from the upper regions of the valley.

Average flow of the Cedar River at Cedar Rapids is 3301 cfs computed from a period of continuous records dating back to 1902. Records at this gauge, which is only 20.8 river miles downstream from the site, can be considered as representative of site discharges as there is little additional inflow or outflow between the two points. The flow occurrence curve for the Cedar Rapids gauge is shown in Figure 2.4-2, which indicates that the flow exceeds 620 cfs 90% of the time and 6600 cfs 10% of the time. Flow occurrence is based on mean daily discharges. Figure 2.4-3 illustrates the seasonal variation of monthly average and extreme flows.

At the time of the initial FSAR, the maximum flood of record at Cedar Rapids had occurred on March 31, 1961, and had reached a peak of 73,000 cfs. This flood reached a stage of 746.5 ft at the plant site. Table 2.4-1 outlines the discharge expected at Cedar Rapids for various return frequencies and the corresponding stage at the site.

Agricultural withdrawals of water are made at a few locations for irrigation purposes. Such withdrawals are regulated by permit from the Iowa Conservation Commission. At the time of the initial FSAR (1972), communication with the Iowa Conservation Commission revealed that only one permit had been issued for withdrawal between the DAEC site and the City of Cedar Rapids. This permit holder was Mr. Alfred Frantz, who farmed 198 acres immediately to the south of the DAEC site. The potential for irrigation at the Frantz farm was recognized in the establishment of the DAEC environmental radiation monitoring program described in Section 11.5, and the Frantz farm is designated as sampling location 74 in original FSAR Table 2.7-1. An interview with the owner established that the irrigation system associated with this farm had not been operative for a number of years. In addition, there were no plans for use except in case of severe drought. The acreage subject to irrigation was 40 acres, and the crops grown were corn and soybeans. The irrigation pump capacity was 600 gpm.

For the stretch of river between Cedar Rapids and the junction of the Iowa and Cedar Rivers, the Iowa Conservation Commission advised that only one irrigation permit had been issued. This was for the Bulichek farm located just outside Cedar Rapids. According to Iowa Conservation Commission records, the Bulichek farm had not made an irrigation withdrawal since 1968. Subsequent communication with the owner revealed that this permit had recently been canceled as the land formerly subject to irrigation had been sold to the City of Cedar Rapids for use as a park.

For irrigation withdrawals at less than 500 gpm, no permit is required. Local, state, and federal agricultural agencies were queried to determine if any additional withdrawals occur. Other than the possibility of irrigation by a few bluegrass sod farms, irrigation without permit does not take place.

2.4.2 FLOODS

2.4.2.1 Flood History

Flood data are contained in Appendix H of the DAEC PSAR.

The estimated peak discharge associated with the standard project flood at the DAEC site is 166,000 cfs as derived from the U.S. Army Corps of Engineers Flood Plain Information Study of Linn County and referenced in Appendix H of the DAEC PSAR.

2.4.2.2 Flood Design Considerations

Flood protection measures for seismic structures are discussed in Section 3.4.1.

2.4.2.3 Effects of Local Intense Precipitation

The probable maximum precipitation storm will not cause the failure of any safety-related structures or equipment either because of local flooding or failure of roof structures and their appurtenances. This is discussed further in Section 3.4.1.1.5.

2.4.3 PROBABLE MAXIMUM FLOOD ON STREAMS AND RIVERS

2.4.3.1 Probable Maximum Precipitation

Complete data on probable maximum precipitation characteristics and distribution are presented in Appendix H, "Maximum Probable Flood," of the DAEC PSAR.

Establishing the probable maximum flood (PMF) level at the plant site was necessary to properly design and locate critical components and to provide the necessary protective measures against such a remote occurrence. The site natural grade level in the vicinity of the plant varies from above elevation 746 ft to elevation 750 ft. As noted in Section 2.4.1, the maximum flood of record at the site occurred in 1961 and rose to

elevation 746.5 ft. The standard project flood as determined by the U.S. Army Corps of Engineers would flood the plant site to elevation 754.5 ft. Consequently, the plant site finished grade is at elevation 757.0 ft. The computed maximum probable flood would have a discharge of 316,000 cfs and would reach an elevation of 764.1 ft at the site.

In addition, a possible wave height of 2.8 ft, including runup, was computed as caused by a sustained wind of 45 mph acting over a maximum fetch of 1.5 miles. Thus, the facility was designed during the construction permit period of review to resist flood waters to an elevation of 767.0 feet. Further review of the wave action and runup caused by winds resulted in additional requirements accepted by the DAEC for additional flood protection. The details are discussed in Section 3.4.1. The Cedar River reached a peak stage of 751 feet with an approximate discharge flow of 110,000 CFS on June 13, 2008.

Due to the gentle topography existing in the river valley, a landslide could not occur of a magnitude that would result in a water level at the site that would approach that of the probable maximum flood.

Figure 2.4-3a shows existing dams and lakes in the basin surrounding the site.

Two combined overbank and channel cross sections are shown in Figures 2.4-4 and 2.4-5. Figure 2.4-4 represents the valley cross section in the immediate vicinity of the safety-related facilities and is the cross section used for final determination of the maximum probable flood at the site as described in Appendix H of the DAEC PSAR. Figure 2.4-5 represents the valley cross section at the next significant point of change in upstream valley characteristics.

The water-level estimate for the probable maximum flood in the vicinity of safety-related facilities is discussed in detail in Appendix H of the PSAR. Supplemental information is shown in Figure 2.4-6, which outlines the water-surface profile expected within the plant property boundaries for the following:

1. The historical flood of record, March 1961.

2. The intermediate regional flood.

3. The standard project flood.

4. The probable maximum flood.

Information pertaining to items 1 through 4 was obtained from the U.S. Army Corps of Engineers <u>Flood Plain Information Study of Linn County</u>. The PMF profile was derived from the studies described in Appendix H of the PSAR. The coefficients used in the derivation of PMF levels are indicated in Figures 2.4-4 and 2.4-5. These coefficients were determined through combined considerations of onsite field inspections and aerial photograph analysis. Verification was made by comparisons with coefficients derived by the Corps of Engineers for similar types of ground cover as a part of their flood plain studies.

2.4.3.2 Precipitation Losses

See Appendix H of the DAEC PSAR and Section 2.4.3.3.

2.4.3.3 Runoff and Stream Course Models

The hydrologic response characteristics and model verification are discussed in detail in Appendix H of the DAEC PSAR.

To develop hydrographs of flood flows from the main tributaries under maximum probable storm conditions, unit hydrographs were first developed at main gauging stations on each of the five tributaries, using storms and recorded floods on these tributaries for this purpose.

To develop a unit hydrograph suitable for application to a maximum probable storm, theory requires that some storm and flood of record be found that satisfies the following two principal criteria:

1. The storm rainfall should be fairly evenly distributed over the drainage area and intense enough to produce surface runoff.

2. The flood hydrograph recorded at the gauging station should have a well-defined peak corresponding with the above rainfall.

Storms and recorded floods on the Cedar River basin that satisfy these two criteria have been extremely rare. This is because the travel paths of summer storms (the type producing a probable maximum flood) are generally from west to east, while the long and narrow subbasins are oriented in a north to south direction. A large number of the flood hydrographs produced by summer storms were considered inappropriate because the rainfall fell on only the lower or upper part of the subbasin. These conditions produce flood hydrographs that are either long and delayed or inordinately high and quick in developing. Neither of these conditions are satisfactory for unit hydrograph development (or verification).

Of the few storms and recorded floods that most appropriately met the criteria, the best were chosen and used to develop the unit hydrographs for each of the subbasins. It is noteworthy that no single storm was found that could be applied over the entire upbasin drainage area.

Herein lies the problem of unit hydrograph verification for the subbasins of the Cedar River. Because the best storms were used to develop the unit hydrographs, any computations using these as a basis for reproducing the flood hydrograph of another storm would be meaningless. It can be expected that the recorded and generated flood hydrographs would not be in agreement. The differences between the two would be due to the differences in storm pattern and intensity from which they were derived. There

would be no basis for changing the original unit hydrograph unless a suitable storm had occurred since the analysis was conducted. A review of the records has revealed no such storm.

The flood routing from the upstream subareas to the DAEC plant site was broken up in two parts: (1) the short river reaches from the junction of the Cedar River with Beaver Creek up to the gauging stations above this point, and (2) the 70-mile reach from this junction down to the plant site.

The flood routing in the reaches above the Beaver Creek junction was accomplished using a time-displacement approach based on recorded flood experience. More specifically, the flood peak velocities for the Cedar and Shell Rock Rivers were assumed to be 1.4 and 2.8 mph, respectively. These velocities are based on recorded flood experience for the storm of August 29-31, 1962, for Cedar River at Waterloo and the upstream gauging stations. The flood peak velocities for the other tributaries were then computed by assuming identical channel characteristics and by using the ratio of the square root of the channel slopes. On the basis of these velocities and the respective distances, travel times for the flood peaks were computed. The subbasin flood hydrographs were combined into a total flood hydrograph at the Beaver Creek junction by offsetting each one by the appropriate time differential and summing. This approach, although not based on a mathematical relationship of channel storage, yields an appropriate definition of the total flood discharge hydrograph over the short reaches involved at the upstream confluence point above Waterloo.

This total flood hydrograph was then routed, as explained in Appendix H (DAEC PSAR), through the 70-mile reach to the plant site. For this reach, an effort was made to apply the commonly used Muskingrum flood-routing method of using recorded floods to develop routing coefficients. This method was found to be inappropriate because of the nonhomogeniety of the channel characteristics and if applied to the probable maximum flood would lead to uncertain results. The Muskingrum method was therefore abandoned and the more tedious Graves method of measuring channel volumes and computing stage-discharge relationships was employed. The Graves method does not use routing coefficients as such.[1]

2.4.3.4 Probable Maximum Flood Flow

Flood flow is discussed in Section 2.4.3.3 above.

2.4.3.5 Water Level Determinations

See Section 2.4.3.1.

2.4.3.6 Coincident Wind Wave Activity

See Section 2.4.3.1.

2.4.4 POTENTIAL DAM FAILURES, SEISMICALLY INDUCED

There are 12 low-head dams on streams within the Cedar River basin that have been built primarily for power purposes, either as hydroelectric facilities or as a source of water for thermal plant cooling. These dams all have small impoundments and do not affect either peak discharge during large floods or stream flow regulation during low-flow periods. They would be submerged under PMF levels and failure would not affect the flood level at the plant site. There are also four natural and five artificial lakes located in the headwater areas of tributaries; they are used primarily for recreational purposes. Figure 2.4-3a shows dams and lakes in the Cedar River basin.

2.4.5 PROBABLE MAXIMUM SURGE AND SEICHE FLOODING

Not applicable to DAEC site.

2.4.6 PROBABLE MAXIMUM TSUNAMI FLOODING

Not applicable to DAEC site.

2.4.7 ICE EFFECTS

Consideration was given to the possibility of ice jams creating a higher flood level, but an inspection of valley topography reveals that at no point could ice create a flood wave approaching that of the probable maximum flood. As a result of the above indications, all essential structures have been designed for flood protection to elevation 767.0 ft.

2.4.8 COOLING WATER CANALS AND RESERVOIRS

See Section 9.2.2.

2.4.9 CHANNEL DIVERSIONS

See Sections 2.4.7 and 9.2.2.

2.4.10 FLOODING PROTECTION REQUIREMENTS

Flood protection is discussed in Section 3.4.1.

2.4.11 LOW-WATER CONSIDERATIONS

Figure 2.4-7 shows the result of a statistical analysis of drought flow conditions at the USGS gauge station in Cedar Rapids. It is expected that over the long term, the once in 50 year, 7-day average flow at the site may drop to 220 cfs while the corresponding

single-day flow may fall to 200 cfs. The minimum daily average flow recorded at Cedar Rapids is 212 cfs.

The rates of population and industrial growth in the Cedar River basin above the DAEC site are low, and the projection of these rates does not indicate a substantial increase in water demand within the next 50 years. Therefore, it is not considered to be possible that increased water demand in combination with the extremely conservative 1000-year minimum flow of 60 cfs would approach the minimum requirement of 13 cfs. There are no storage facilities of importance in the Cedar River basin above the DAEC site, and planning studies by state and Federal agencies do not indicate that such storage will be needed or constructed within the next 50 years. Therefore, it is not possible for extremely low flows to be caused by the operation of such installations.

2015-005 The design of the DAEC Iowa Vanes, guidewall, Spur Dikes (Wing Dams) and intake structure ensures that during periods of low flow all available river flow is diverted to the intake structure and to the river water supply pumps. A minimum 6 ft 0 in. submergence is maintained to ensure that no cavitation occurs. A minimum submergence of 2 ft 7 in. is necessary to prevent cavitation of the river water supply pumps.

2.4.12 DISPERSION, DILUTION, AND TRAVEL TIMES OF ACCIDENTAL RELEASES OF LIQUID EFFLUENTS IN SURFACE WATERS

The Cedar Rapids municipal water supply system as of 1981 consists of 31 wells with an average depth of 65 ft. All wells have 30-in. casings and intake screens of 10 to 15 ft in length; they are located in the Cedar River Valley 50 to 300 ft from the riverbank. These wells have a total estimated capacity of 42 mgd and the present average daily usage is 22.6 mgd. The single maximum day withdrawal was over 33.8 mgd. Before distribution to the city water users, the water is softened by a soda-ash treatment called "excess lime softening." This is very effective in removing particulate matter from the water. The present treatment capacity of the treatment facility is 52 mgd.

There is no correlation between Cedar River flow and emergency water withdrawal directly from the Cedar River by the municipal water system. The original low-lift pumping station is maintained intact, but would not be used unless there were a power failure or a breakage in mains coming from the pumping fields. If the water plant experienced a total loss of power, it would be able to withdraw only 5 mgd using two gasoline engines. If power was available and a well main was not available, it could withdraw directly from the low lift station. This water would also be softened before distribution. It is therefore an extremely remote possibility that any water used for drinking within the City of Cedar Rapids will come directly from the river. Cedar Rapids is the only major city using Cedar River water between the DAEC plant site and the Mississippi River. All other communities located on the river are less than 1000 population with one exception, which is less than 5000 population.

In Section 10.2.1 of the Revised Environmental Report, an analysis was made of the worst inadvertent pumpage of radioactive water. This consisted of a 20 min discharge at a 50-gpm rate of radioactive water having a concentration of 3 x 10^{-3} μCi/cc to the river. The resultant dose was 0.068 man-rem exposure to people drinking water from the municipal water system in Cedar Rapids, if all of the Cedar Rapids supply had been taken directly from the river. As noted above, this is an extremely remote possibility.

In Section 11.3.3 of the Revised Environmental Report, a dose resulting from a nonmechanistic failure of all liquid radwaste tanks within the radwaste building was calculated. The resultant calculated total fraction of maximum permissible concentration at the first municipal water intake was 0.04. Again, this radioactivity would not be taken into the Cedar Rapids water distribution system unless the river was being used as a direct source.

Precise quantitative studies have not been conducted to determine travel times for various river flow conditions between the DAEC plant site and the Cedar Rapids water intake. However, it has been estimated that for a 1500 cfs river flow, a minimum of a 9-hr travel time could be expected. If a slug of liquid was injected into the river at the plant site and had traveled to the Cedar Rapids intake point, again at 1500 cfs flow, it is estimated that this slug would pass the intake in 3 hours.

See Section 11.5.7 for a discussion of the environmental radioactivity monitoring program.

2.4.13 GROUND WATER

2.4.13.1 Description and Onsite Use

In the Cedar River basin of Iowa, ground water is obtained from two main sources: shallow wells in unconsolidated glacial and surficial deposits, and deep wells into any of three underlying bedrock aquifers. Wells in glacial deposits usually range between 70 and 200 ft deep depending on location. Wells in rock range between 300 and 1700 ft deep depending on location and on whether the upper, middle, or lower aquifer is tapped.

1. Jordan Aquifer

The lower rock aquifer is estimated to lie within the depth range of 1000 to 1700 ft below ground at the plant site. This aquifer is composed of Ordovician and Cambrian rocks, which include St. Peter sandstone, Prairie du Chien dolomite and sandstone, Jordan sandstone, and St. Lawrence dolomite. The Jordan sandstone is the most prolific source of ground water.

Water is under high artesian pressure. Well production is about 10 gpm/ft of pumping drawdown. Many wells in this region produce in excess of 1000 gpm of good

quality water. There are no plans to develop the Jordan aquifer as a primary water supply for the plant since the Jordan aquifer is a sandstone aquifer which cannot tolerate excessive pumping; alternate wet and dry conditions would lead to ultimate crumbling and collapse.

2. Shallow Aquifers

Many adequate supplies of good water are obtained from sand and gravel aquifers in the surficial deposits that overlie the bedrock. These are replenished by direct precipitation, periodic flooding, and, where adequate underground hydraulic connections with streambeds exist, by river recharge.

Borings indicate that two aquifers underlie most of the site area, an upper water table aquifer composed of fine to medium sand, and a lower artesian-type aquifer in weathered rock. The two aquifers are separated by 10 to 60 ft of relatively impervious clayey material. Boring logs and water-level measurements indicate that this clay aquiclude is probably continuous over most of the site area. This clay extends above and below river bottom elevation at most boring locations.

Ground-water measurements indicate that flows in the upper aquifer are toward the river in a general southeasterly direction across the site. Pressure surface contours indicate that flows in the lower aquifer are also in this same general direction.

Since the aquifer below the clay is under considerable pressure in the natural state, any ground-water transfer between the two aquifers would be from the lower into the upper aquifer. With the production wells operating, the lower aquifer pressure could be lower than the surface water table in the immediate vicinity of these wells. Under this circumstance, ground-water transfer could possibly be reversed over a long period of time.

In support of the site Ground Water Protection Program, 6 pair of monitoring wells were installed at the site in 2006. The wells were drilled in pairs. A pair consisting of a shallow well drilled to the base of the upper alluvial aquifer and a deeper
2016-010 "intermediate" well drilled to the base of the clay aquiclude. Water table elevation data gathered from the shallow wells has indicated lateral movement of the shallow groundwater on site to be variable. Groundwater flow directions trended from southwesterly to southeasterly.

2016-010

2016-010 From 2011 through 2017, several additional monitoring wells were installed at the site in support of the Groundwater Protection Program. The locations of these wells are arranged as follows:

- One pair of monitoring wells is located "up-gradient", northwest of the plant, to serve as a control and to help more accurately define flow characteristics across the site.

- 2016-010 Pairs of shallow and intermediate monitoring wells were installed in close proximity to "below-grade" plant systems containing radioactive liquids to provide early indication of ground water contamination in this area.

- Several shallow wells are located directly south of the Turbine Building south wall to assist in detecting and locating the source of any contamination in the alluvial ground water in that area.

- Pairs of shallow and intermediate monitoring wells, as well as several shallow wells, are located down gradient of the Protected Area near the Offgas Stack to provide early detection of ground water contamination outside the Protected Area.

- Several pairs of shallow and intermediate wells complete an arc of monitoring wells located on the south side of the site.

- 2015-008 Modeling indicated downward flow into the Limestone; therefore, a single deep monitoring well was also installed in 2015 into the Limestone formation to detect 2016-010 potential contamination in the Limestone Aquifer. This deep well, along with other monitoring wells, are located east and northeast of the power block to test for ground water flow effects due to the operation of the "D" Production Well.

During the same time frame, a number of shallow ground water extraction wells were installed southeast of the Turbine Building and in the southeast Owner Controlled Area, in the path of normal groundwater flow, for the purpose of extracting and treating any contaminated ground water identified.

Groundwater flow direction at the base of the clay rich till was also variable. Ground water flow directions trended from southeasterly to northeasterly. This variability in lateral flow direction has been attributed to the operation of the site production wells which are acting to lower the static water levels in both of the overlying aquifers. Although variability in lateral flow direction is indicated, the description in section 2.4.13.3 of a flow direction “toward the river” is accurate.

Gradients causing flow are quite steep in both aquifers. Information collected on domestic wells within a 1-mile radius of the plant indicates that all domestic wells west and north of the plant are up the ground-water slope from the plant; that is, ground water flows past these wells toward the plant or along some other path directly toward the river. Domestic wells southwest and south of the plant are approximately 1 mile away and are not in the line of ground-water flow past the plant.

Should the area be inundated by a Cedar River flood, infiltration would temporarily raise the general ground-water table. Some domestic wells south of the plant would be flooded. Those on higher ground would maintain their same relative positions on the general water table slope.

In the Village of Palo, 2.5 miles south-southwest of the plant, the water table stands approximately 12 ft below average ground-surface elevation 745, or at elevation 733. Ground-water flow is in an easterly direction toward the river.

A comprehensive subsurface exploration program was performed to establish the adequacy and quality of water available for plant use. Two production wells were drilled into the lower artesian aquifer in weathered rock, and a yield of 750 gpm for each well, pumping concurrently, was established. Test reports of water analysis indicated a good mineral quality.

2.4.13.2 Sources

There are no potable water supplies taken from the Cedar River surface water downstream of the DAEC. Irrigation uses are presented in Section 2.4.1. No permit is required nor is there any restriction on the withdrawal of water from the river for livestock watering, and no records are available.

The primary user of water that could originate from the river is the City of Cedar Rapids. Some of the recharge for the city wells comes from the river at normal withdrawal, and under periods of no or low withdrawal no recharge comes from the river.

In 1981, the average city water consumption was about 22.6 million gal per day (mgd) with a peak day consumption of approximately 33.8 million gal. It has been estimated that this will increase 2% to 5% per year. This system is expected to have an ultimate capacity of 42 mgd. Total storage capacity within the city system is approximately 16.3 million gal. All of the city water was supplied by wells located adjacent to the Cedar River. Because of this location, a large portion of the water withdrawn from these wells was recharged from the river. In addition, the city has an emergency standby system capable of withdrawing 24 mgd directly from the river.

Within a 1.5-mile radius of the plant, there were 14 property owners having 1 or more wells. The use of these wells extended beyond potable supply to such items as swimming pools, livestock watering, and irrigation.

Major industrial water use, within 50 miles downstream of the plant, is concentrated in the Cedar Rapids area. Primary uses of river water include condenser cooling and process water.

Agricultural withdrawals are made at a few locations for irrigation purposes. In addition, limited recreational use is made of the river, particularly above the power plant dams in Cedar Rapids, and in the headwater area recreational lakes. The operation of the plant does not affect these activities.

A map of surface water users is presented in Figure 2.4-8. The owner, depth of well, use rate, and type of use is presented in Table 2.4-2. The figure and table represent the situation at the time of the initial FSAR.

In the Village of Palo, 2.5 miles southwest of the plant, there are about 140 homes with individual well points. These wells are 1-1/4 in. in diameter by 3 ft long and are driven to a depth of 18 ft. Static water level is about 12 ft below the surface. In addition to residential well points, wells existed (at the time of the initial FSAR) at the school, church, two taverns, two groceries, a barber shop, a garage, the Legion Hall, and a feed store. There were also six fire plugs tied into two well points each. There was one 85-ft well belonging to Lyle Dodd.

Assuming 3.5 persons per household, the average usage for each of these wells would be 175 gpd.

Average water usage is based on the following:

User	Gallons per Day
Human	50
Cow	35
Horse	12
Hog	3
Sheep	2
Each 100 chickens	3
Each 100 turkeys	5
Each 100 ducks	5

2.4.13.3 Accident Effects

There are no wells that presently exist down gradient of the plant with respect to the upper aquifer. It is not expected that the gradient in the upper aquifer could change such that existing or future wells would be down gradient without a major geological change. Areas in the line of ground-water flow from areas of potential spills are all onsite property.[2]

Figure 2.4-9 was developed in 1968 to determine the potential for shallow wells[3] in the vicinity of the site. Figure 2.4-10 indicates the location of observation wells in the site vicinity. The "S" wells are all shallow well points 10 to 20 ft deep tapping the upper aquifer. Table 2.4-3 gives these observations for a period of low-river stage, and a period of high-river stage. These observations confirm Figure 2.4-9. A generalized water table map of Linn County is given in Figure 2.4-11. Figure 2.4-11 further indicates the ground-water flow towards the river.

Figure 2.4-12 is a generalized map of the piezometric surface of the Silurian-Devonian aquifer within Linn County. This lower aquifer also flows toward the rivers. The production wells tap this lower aquifer. Extensive pumping in the Cedar Rapids area has depressed the piezometric level about 105 ft in 70 years in the center of the area.

A piezometric level map for the site area has not been developed. Information available from borings indicates that the flow in the lower aquifer is toward the river. Test results from production well tests indicate that the recharge comes from a northerly direction in the site vicinity. Observation wells into this aquifer are highly localized making it difficult to state the direction of flow other than toward the river. Any spill at the site would seep into the ground and into the upper aquifer. This worst-case spill is considered in Section 11.2.3.

2.4.13.4 Monitoring or Safeguard Requirements

Offsite wells west, southwest, south, and southeast of the site used for domestic water supplies at the time of the initial FSAR were sampled monthly and analyzed radiometrically in the same manner as surface waters. See Section 11.5.7. The identification of these wells was as follows:

Bull residence, No. 57	West
Frantz residence, No. 58	Southwest
Frantz cottage, No. 59	Southeast
Comp residence, No. 60	South

See the Technical Specifications for the current monitoring program.

Section 11.2.3.5 presents an analysis of holdup in the ground-water system for a postulated release of radioactive liquids from the Nonseismic radwaste building. This analysis used a permeability of 10^{-2} cm/sec for the sand and clay soil mixture and an effective porosity of 15% on the basis of data obtained from well production studies done at the site.

In the immediate vicinity of the site, land use is devoted strictly to farming, and no significant change in ground-water use is expected in the foreseeable future. This is substantiated by Table 2.1-2 which shows a 1980 population of 18 people within 1 mile of the plant versus a 2010 population estimate of 27 people shown on Figure 2.1-6.

2.4.13.5 Design Bases for Subsurface Hydrostatic Loading

See Section 2.5.4.10.

2.4.14 TECHNICAL SPECIFICATION AND EMERGENCY OPERATION REQUIREMENTS

The Technical Requirements Manual requires reactor shutdown if a flood on the Cedar River reaches the DAEC plant grade.

REFERENCES FOR SECTION 2.4

1. Graves, E. A. "Improved Method of Flood Routing," Journal of the Hydraulics Division, ASCE, 1967.

2. Water-Supply Bulletin, No. 10, Iowa Geological Survey.

3. Shallow Well Potential, 10,000 gpm, DAEC, near Palo, Iowa, Commonwealth Associates, Inc.

Table 2.4-1

FLOOD FLOW-RETURN FREQUENCIES

Return Period (years)	Peak Flow at Cedar Rapids (cfs)	Stage at Plant Site (ft ms1)
1 in 5	51,000	743.4
1 in 20	63,000	745.0
1 in 50	72,000	746.2
1 in 100	79,000	747.0
1 in 500	100,000	749.0

Table 2.4-2 Sheet 1 of 20

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
1	Free Methodist Church	Drilled	5-3/16	110		Devonian-Silurian Formation	500
2	Clarence Waterbury	Drilled		120		Devonian-Silurian Formation	200
3	Ivan Oliphant	Drilled	6	120	58	Devonian-Silurian Formation	200
4	Jack Luke	Drilled	6-1/4	112	54	Devonian-Silurian Formation	200
5	Guy Cole	Drilled	6	120	53	Devonian-Silurian Formation	200
6	Merle Wilson	Drilled	6	120	69	Devonian-Silurian Formation	200
7	Stella Moore	Drilled	6	105	60	Devonian-Silurian Formation	300
8	Ralph Wildman	Drilled	6-5/8	122	65	Devonian-Silurian Formation	200
9	Ray Bowers	Drilled	5-3/16	118	64	Devonian-Silurian Formation	200
10	Meryl Bowers	Drilled	5-3/16	104	16	Devonian-Silurian Formation	200

Table 2.4-2 Sheet 2 of 20

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
11	Jay Olinger	Drilled	5-3/16	104	16	Devonian-Silurian Formation	200
12	Lon Hagerman	Drilled	5	108	31	Devonian-Silurian Formation	200
13	Toddville School	Drilled		200		Devonian-Silurian Formation	850
14	Church of Christ	Drilled	6-1/4	132	21	Devonian-Silurian Formation	500
15	Pat Roob	Drilled	6-1/4	140	38	Devonian-Silurian Formation	200
16	Melvin McBurney	Drilled	5				250
17	Russel McBurney	Drilled	5	84			200
18	Don Harter	Drilled	5				200
19	John Topinka	Drilled	6	105	29	Devonian-Silurian Formation	200
20	Alice Matheney	Drilled	6-1/4	124	32	Devonian-Silurian Formation	200
21	Bev Roman	Drilled	5	165	110	Devonian-Silurian Formation	200

Table 2.4-2

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
22	Lyle McBurney	Drilled	6	110	110	Glacial deposits immediately overlaying bedrock	250
	Lyle McBurney	Drilled	7	330	124	Devonian-Silurian Formation	500
22	Lyle McBurney	Drilled	6	195	81	Devonian-Silurian Formation	250
23	Dee Bowers	Drilled		222		Devonian-Silurian Formation	250
24	Richard Odin						
25	Doug Milburn						
26	Bob McCann						
27	Carl Holsinger	Drilled	6	150	49	Devonian-Silurian Formation	200
28	Wensull Andrews	Dug		18		Alluvial sand	200
29	Virgil Newman	Drilled	5	87	55	Devonian-Silurian Formation	200
30	Ed Phillips	Drilled	5-3/16	112	54	Devonian-Silurian Formation	200

Note: No information available where name only appears

Table 2.4-2 Sheet 4 of 20

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
31	Hugo Chandler	Drilled	5-3/16	108	50	Devonian-Silurian Formation	200
32	Eddie Anderson	Drilled	6-1/4	116	17	Devonian-Silurian Formation	200
33	Bud Wilman	Drilled	6-1/4	75	15	Devonian-Silurian Formation	200
34	Bob Engle	Drilled	6	210	82	Devonian-Silurian Formation	200
35	Bud Wilman	Drilled	6	225	85	Devonian-Silurian Formation	200
36	Leander Hoff	Drilled	6	250	88	Devonian-Silurian Formation	200
37	Lowell Sissen	Drilled	6	225	66	Devonian-Silurian Formation	200
38	Ronald Lamp	Drilled	6	180	34	Devonian-Silurian Formation	200
39	Bill Hepker	Drilled	6	190	135	Devonian-Silurian Formation	200
40	Lyle Shakespear	Drilled	5	150	150	Glacial deposits immediately overlaying bedrock	200

Table 2.4-2 Sheet 5 of 20

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
41	Ed Cosgrove	Drilled	5	130	108	Devonian	200
42	Charles Moore	Drilled	6	180	180	Glacial	200
43	John Graves	Drilled	6	180	180	Glacial	200
44	Tom Moore	Drilled	5	160			200
45	Leo Ellis	Drilled	3-3/16	110	94	Devonian	450
46	Bill Hanson	Drilled	6-1/4	120	42	Devonian	450
47	Elsie Hepker	Drilled	5-3/16	98	53	Devonian	450
48	Clarence Hepker	Drilled	6-1/4	110	74	Devonian-Silurian Formation	200
49	Ray Novy	Drilled	6-1/4	92	67	Devonian-Silurian Formation	200
50	Irene Morris	Drilled	6	305	257	Devonian-Silurian Formation	200
51	Clarence Morris	Drilled	6	250	190	Devonian-Silurian Formation	200
52	George Chrystle	Drilled	6-1/4	178	89	Devonian-Silurian Formation	200
53	Jim Washburn	Drilled	6-1/4	168	66	Devonian-Silurian Formation	200
54	Kiwanas Club	Drilled	6	270	133	Devonian-Silurian Formation	200

Table 2.4-2

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
55	Richard Schmadeke	Drilled	5-3/16	158	158	Glacial deposits immediately overlaying bedrock	200
56	Clifton Mitchell	Drilled	6	118	72	Devonian-Silurian Formation	200
57	Eldee Bowers	Drilled	6	123	105	Devonian-Silurian Formation	200
58	Green Groves Church	Drilled	6-1/4	100	28	Devonian-Silurian Formation	200
59	Wallace Oliphant	Drilled	6	150	26	Devonian-Silurian Formation	200
60	Darress Oliphant	Drilled	6-1/4	160	68	Devonian-Silurian Formation	200
61	Don Booze	Drilled	6-1/4	135	32	Devonian-Silurian Formation	200
62	Bernita Coonrod	Drilled	6-1/4	135	32	Devonian-Silurian Formation	200
63	Tom McGenis	Drilled	6	120	19	Devonian-Silurian Formation	200
64	Harlan Bruce	Drilled	6-1/4	167	22	Devonian-Silurian Formation	200

Table 2.4-2 Sheet 7 of 20

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
65	Richard Edaburn	Drilled	6	210	72	Devonian-Silurian Formation	200
66	Keith Dice	Drilled	6	210	139	Devonian-Silurian Formation	200
67`	Charlie Rozek	Drilled	6	155	112	Devonian-Silurian Formation	200
68	Larry McBurney	Drilled	6	165	113	Devonian-Silurian Formation	200
69	Ernie Paul	Drilled	6	100	100	Glacial deposits immediatel y overlaying bedrock	200
70	Melvine McBurney	Drilled	5	84		Devonian-Silurian Formation	200
71	Allen McBurney	Drilled	6-1/4	110	34	Devonian-Silurian Formation	200
72	Allen McBurney	Drilled	6	150	30	Devonian-Silurian Formation	200
73	Loland Cooper	Drilled	6-1/4	135	15	Devonian-Silurian Formation	200
74	Oliver Cox	Drilled	5-3/16	97	30	Devonian-Silurian Formation	200

Table 2.4-2 Sheet 8 of 20

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
75	Bob Shireman	Drilled	6	195	18	Devonian-Silurian Formation	200
76	John Shumaker	Drilled	6	165	25	Devonian-Silurian Formation	200
77	Russel Sleck	Drilled	6	210	40	Devonian-Silurian Formation	200
78	Larry Conover	Drilled	6	315	19	Devonian-Silurian Formation	200
79	Burnell Hines	Drilled	6	120	81	Devonian-Silurian Formation	200
80	Bud Wilman	Drilled	6	240	104	Devonian-Silurian Formation	200
81	Junior Hanover	Drilled	6-1/4	85	30	Devonian-Silurian Formation	200
82	Jim Kirchner	Drilled	6	330	98	Devonian-Silurian Formation	200
83	Laverne Kuel	Drilled	6	270	153	Devonian-Silurian Formation	200
84	Henry Michels	Drilled	6-1/4	137	56	Devonian-Silurian Formation	200

Table 2.4-2 Sheet 9 of 20

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
85	Allen McBurney	Drilled	6-1/4	110	34	Devonian-Silurian Formation	200
86	Leotia Chewning	Drilled					50
87	Richard Powers	Dug		18		Alluvial sand	100
88	J.A. Moser	Sand point			18		
89	Marion Schminke						
90	Wilma Williams	Drilled	6	100		Devonian-Silurian Formation	1150
91	Jack Wilder	Drilled	6	150		Devonian-Silurian Formation	150 One windmill for livestock
92	Laverne Fink	Drilled	6	135		Devonian-Silurian Formation	100
93	Henry Michaels						
94	Roger Wiegel	Drilled	6	105		Devonian-Silurian Formation	250
95	Leroy Boots	Drilled					
96	K.F. Schrieber	Drilled					
97	Loren Rezabek	Drilled					
98	G.T. McCormick	Drilled	6				2750
99	Curtis Schnell	Drilled					

Table 2.4-2

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
100	Clifford Bowers	Drilled					
101	Gene Anderson	Drilled	4-1/4	90	30	Devonian-Silurian Formation	1775
102	Harold Lanning	Drilled	5	90		Devonian-Silurian Formation	150
103	Don Lanning	Drilled	6	120		Devonian-Silurian Formation	2850
104	Elsie Squiors	Drilled	6	175		Devonian-Silurian Formation	3075
105	Enoch Smith	Drilled	6	286		Devonian-Silurian Formation	50
106	Enoch Smith	Drilled		186		Abandoned	
107	G.E. Moser	Drilled					
108	Fay Wisehart	Drilled	6	300		Glacial deposits immediatel y overlaying bedrock	250
109	Yarbourough	Drilled	6	320		Glacial deposits immediatel y overlaying bedrock	180
110	Carol Hines	Drilled	6	200	60	Devonian-Silurian Formation	1750

Table 2.4-2

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
111	Burnell Heinz	Drilled	6	120	8	Devonian-Silurian Formation	298
112	Jack Orman	Drilled	4	60	30	Devonian-Silurian Formation	410
113	Harold Drew	Drilled	6	100	30	Devonian-Silurian Formation	160
114	Frank Stallman	Drilled	4	100			595
115	H.W. Lutz	Drilled	5	130	80	Devonian-Silurian Formation	100
116	Melvin Wage	Drilled	4	120		Devonian-Silurian Formation	1300
117	E.W. Carson	Drilled	6	80	57	Devonian-Silurian Formation	200
118	Firman Stallman						
119	Fred Klindt	Drilled	4			Devonian-Silurian Formation	100
120	Karl Behrens						
121	John Behrens						
122	Jerry Dellrich						

Note: No information available where name only appears

Table 2.4-2 Sheet 12 of 20

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
123	Cletus Thomas	Drilled	6	196		Devonian-Silurian Formation	250
124	Elzy Morris	Drilled	6	305		Glacial deposits immediatel y overlaying bedrock	50
125	Ronald Roberts						
126	Bill Eike						
127	Randall Eike						
128	Myron Okken	Drilled	6				760
129	Orville Wright	Drilled	6	210		Devonian-Silurian Formation	190
130	Conservation Commission						
131	Ray Derby						
132	G. Fifield	Drilled	6	168		Devonian-Silurian Formation	374
133	George Loher						400
134	Ray Fifield						
135	Ed McBurney						
136	Ken Griener						
137	Quentin Collins						
138	M. Gossman						

Table 2.4-2 Sheet 13 of 20

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
139	George Williams						
140	R. M. Ulrich	Drilled	6	350		Devonian-Silurian Formation	600
141	Richard Meeny						150
142	Windmill (abandoned)						
143	Fred A. Morris						
144	Henry Just						
145	Lester Bonishek	Drilled	5	160		Devonian-Silurian Formation	2300
146	H.C. Culin						
147	Martin Hanzlik						
148	Knepper	Sand point				Alluvial sand	400
149	Roy Miller	2 S.P.					500
150	Hughes	Drilled	6	170	135	Devonian-Silurian Formation	200
151	J. Wallander						
152	G. Hazeltine	Drilled					150
153	James Sauer						

Note: No information available where name only appears

Table 2.4-2

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
154	Milfred Heck	Drilled	5	196		Devonian-Silurian Formation	250
155	Frank Votroubeck	Sand point					50
156	Lester Heck	Sand point		15			100
157							
158	Coon Hunting Club						
159	Roger Harty	Drilled					125
160	Paul Burke						
161	Earl Coleman						100
162	Don Hopkins						400
163	John Bowers	Drilled	5	150			850
164	James R. Stolba						
165	Bob Baker						
166	Max Thompson						
167	Lyle Shakespear	Sand point		25			125
168	Jess Shannon	Drilled		185			350
169	Ollie Corum	Drilled		175			50

Note: No information available where name only appears

Table 2.4-2 Sheet 15 of 20

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
170	Leona Rankin						6225
171	Charles Frantz	Drilled	5	136	43	Devonian-Silurian Formation	in-active
	Charles Frantz	Drilled	5	128	36	Devonian-Silurian Formation	300
172	Herbert Hall	Drilled					
173	G. Wayne Elliot	Drilled	5-1/2	109			
174	Joe Buhrman	Drilled	6	120	12	Devonian-Silurian Formation	250
175	W.R. Lagerquest	Drilled	6	130		Devonian-Silurian Formation	100
176	Mrs. Dewey Robins	2 S.P.		35		Alluvial sand	1750
177	Cecil Railsbeck	Drilled	4	100		Devonian-Silurian Formation	2050
178	John Stram	Drilled	6	200	190	Devonian-Silurian Formation	2250
179	Rose Myers	Drilled	4	110	45		2921
180	Sherman Hopker	Drilled	6	65		Devonian-Silurian Formation	100

Note: No information available where name only appears

Table 2.4-2

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
181	Leo Pickerell	Drilled	6	216	200	Glacial deposits immediatel y overlaying bedrock	820
182	B. Harlan Moore						
183	Don Pemrose	Drilled	4	90		Devonian-Silurian Formation	50
184	Boyd Frazier	Drilled	6	180	150	Devonian-Silurian Formation	1429
185	Tom Lewis	3 S.P.		18-25		Alluvial sand	4550
186	Kenneth Lewis	Sand points		30-40		Alluvial sand	250
187	Kenneth Lewis	Sand points		30-40		Alluvial sand	250
188	Cora Stodola	3 S.P.		30-40		Alluvial sand	2900
189	Ira Lewis	Drilled		82		Devonian-Silurian Formation	200
190	Carl Andrews	Drilled		287		Glacial deposits immediatel y overlaying bedrock	2620

Note: No information available where name only appears

Table 2.4-2 Sheet 17 of 20

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
191	Wm. Strave	Drilled	3-3/4	100		Devonian-Silurian Formation	1475
192	Melvin Young						
193	Melvin Young						
194	Melvin Young						
195	Mrs. Dewey Robins	Drilled	6	267		Devonian-Silurian Formation	450
196	Melvin Young						
197	Stan Zeiser						
198	Gary Railsbeck	Drilled	6	151		Devonian-Silurian Formation	4725
199	R.C. Hepker	Sand point		49		Alluvial sand	100
200	John Comp	Sand point		20		Alluvial sand	200
201	John Comp	2 S.P.		20		Alluvial sand	510
202	Laveren Langreth	Sand point		20		Alluvial sand	186
203	Gerald Ball	3 S.P.		20		Alluvial sand	7666
204	Maurice VanNote						

Note: No information available where name only appears

Table 2.4-2 Sheet 18 of 20

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
205	Ted Coleman	Sand points		24		Alluvial sand	200
206	Moubry	Sand points		24		Alluvial sand	250
207	Charles Stodola	Drilled	6	120	95	Devonian-Silurian Formation	350
208	Orville Faust	Drilled	5	110	57	Devonian-Silurian Formation	350
209	Marvin Johnson	Drilled	5				200
210	Jim Bemer	Sand point		18		Alluvial sand	250
211	H.L. Johnson	Drilled	6	170			580
212	Robert Shattucks	Drilled		120			150
213	Marie McCarcle	Sand point		25		Alluvial sand	200
	Marie McCarcle	Sand point		25		Alluvial sand	75
214	Dick Bull	Drilled		115		Devonian-Silurian Formation	1250

Table 2.4-2 Sheet 19 of 20

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
215	Alfred Frantz	Sand point		25		Alluvial sand	200
		Sand point		18		Alluvial sand	700
		Sand point		20		Alluvial sand	
		Sand point		25		Alluvial sand	
		Sand point		18		Alluvial sand	200
216	Rex Meyre	Drilled		140		Devonian-Silurian Formation	1050
		Sand point		15-20		Alluvial sand	500
217	Jesse Lint	Drilled	6	160		Devonian-Silurian Formation	500
218	Harold Kephart	Drilled		200		Devonian-Silurian Formation	650
219	Cliff Mather	Sand point		20-30		Alluvial sand	550
220	Loyal Meltons	Drilled	5	180	120	Devonian-Silurian Formation	250
	Loyal Meltons	Drilled	5	210	165	Devonian-Silurian Formation	700

Table 2.4-2 Sheet 20 of 20

SURFACE WATER USERS

No.	Owner	Type	Diameter (in.)	Total Depth (ft)	Total Casing (ft)	Chief Acquifer	Usage (gpd)
221	D.W. Bizek						
222	Cliff Mather	Drilled		165		Devonian-Silurian Formation	600
223	Melvin Young	Drilled		80	65	Devonian-Silurian Formation	700
224	Wencil Rehders	Drilled		80	65	Devonian-Silurian Formation	650
225	DAEC Production well 1	Gravel packed	16	120	92	Glacial deposit overlaying bedrock	750[a]
226	DAEC Production well 2	Gravel packed	16	138	110	Glacial deposit overlaying bedrock	750[a]
227	DAEC (onsite well)	Drilled	8	118	46	Devonian-Silurian Formation	400[a]

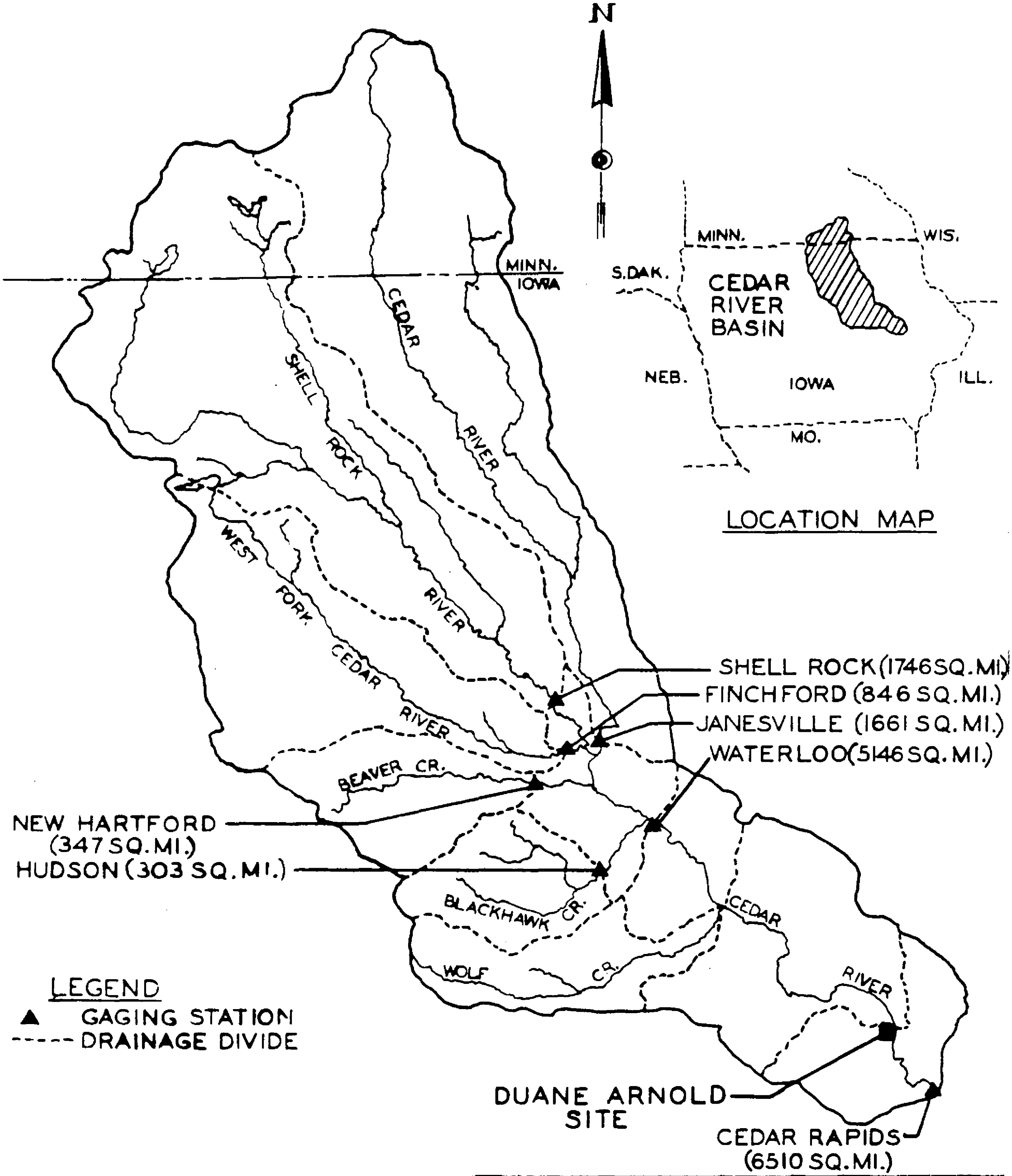
Note: No information available where name only appears

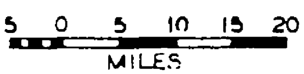
[a] Gallons per minute

Table 2.4-3
GROUND-WATER LEVELS

Date	S-1	S-2	S-3	S-4	S-5
03/26/71	738.25	737.37	737.12	737.70	739.96
04/02/71	738.16	738.08	736.83	737.41	739.63
04/09/71	738.33	736.83	736.99	737.78	739.46
04/16/71	737.66	735.49	735.98	737.68	739.33
12/03/71	732.25	730.66	731.28	733.36	736.63
12/10/71	732.26	730.66	731.20	733.36	736.64
12/17/71	732.41	731.33	731.53	733.45	736.78
12/24/71	732.83	730.83	731.45	733.61	736.88
07/14/72	734.68	732.33	732.70	735.30	738.08
07/21/72	734.88	733.11	733.60	735.22	738.28
7/28/72	734.80	732.78	733.20	735.36	737.41
08/04/72	734.98	733.78	733.53	735.42	738.96

Date	S-6	S-7	S-8	S-10	River
03/26/71	749.15	744.69	742.77	744.07	737.2
04/02/71	749.32	744.69	742.85	744.07	738.6
04/09/71	749.28	744.65	742.81	743.87	736.3
04/16/71	749.54	744.66	742.85	744.20	734.5
12/03/71	744.65	742.86	740.85	743.42	730.12
12/10/71	744.65	743.11	741.03	743.99	730.00
12/17/71	744.90	743.28	741.27	744.24	730.40
12/24/71	745.23	743.44	741.35	744.07	729.76
07/14/72	748.40	744.26	742.60	745.89	732.20
07/21/72	748.57	744.02	742.48	744.18	732.80
07/28/72	748.88	743.99	742.57	744.32	731.20
08/04/72	749.95	744.58	742.85	744.34	733.20





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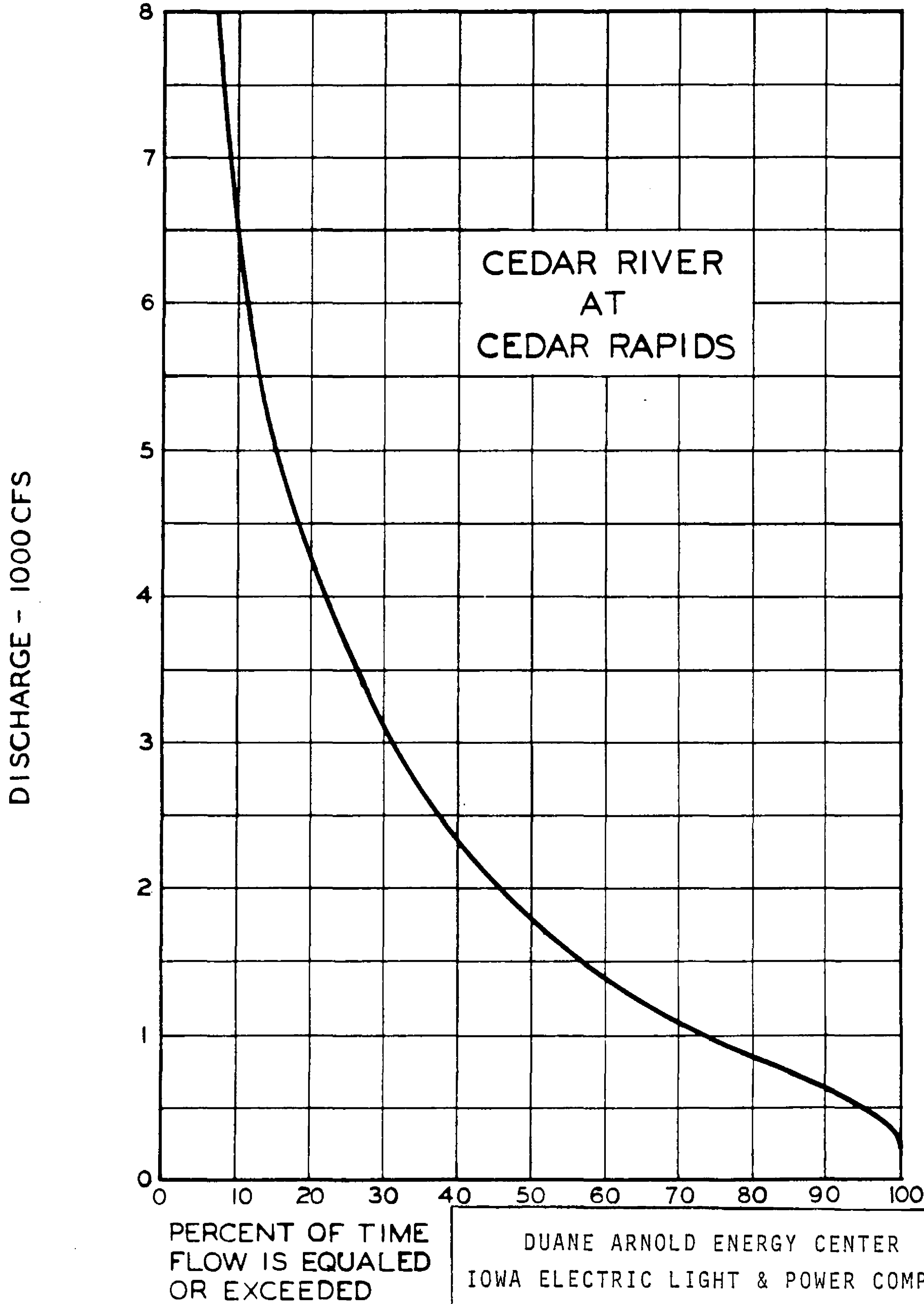
Cedar River Basin Map

Figure 2.4-1



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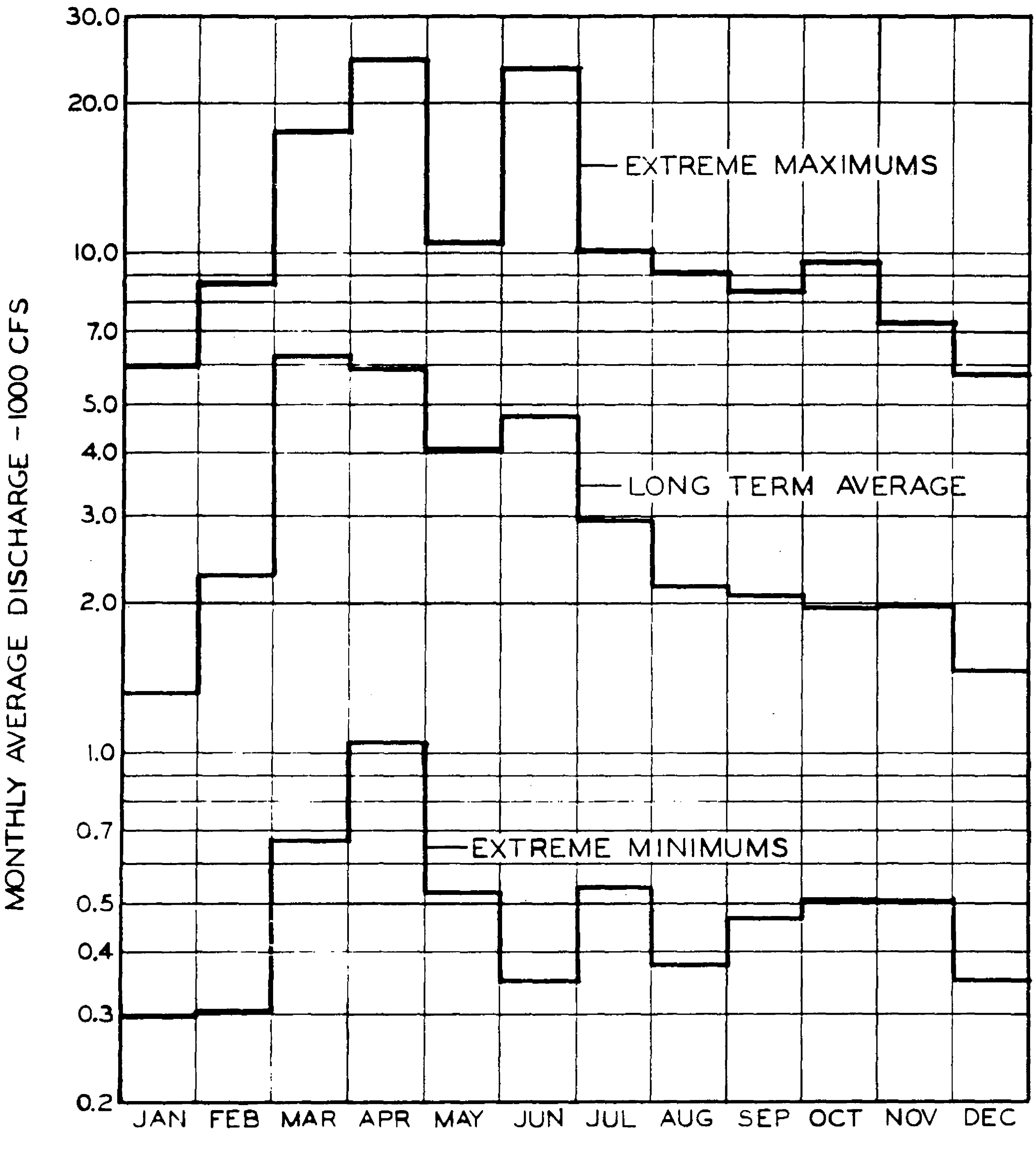
Flow Occurrence, Cedar River
at Cedar Rapids

Figure 2.4-2



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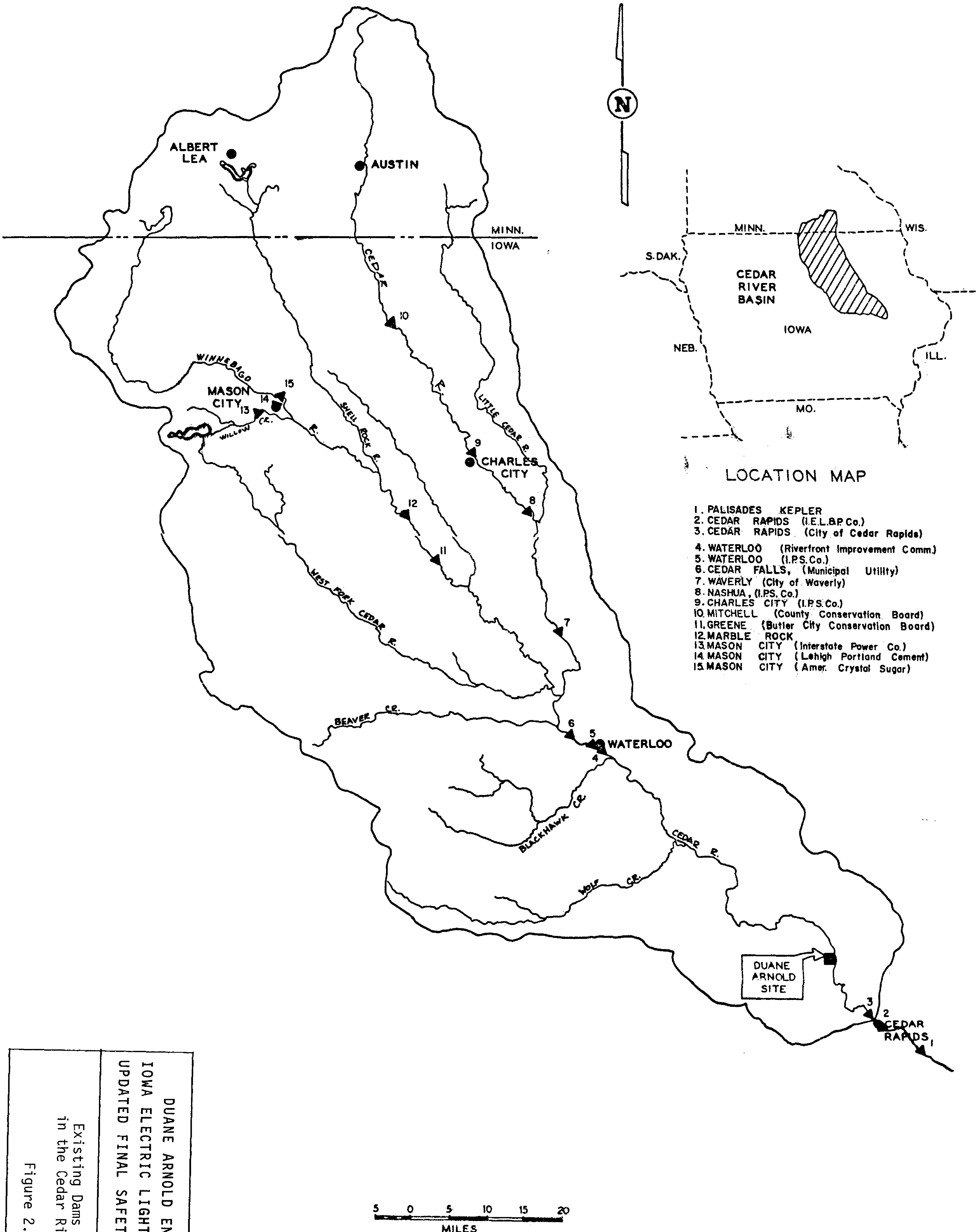
Monthly Average Discharges,
Cedar River at Cedar Rapids

Figure 2.4-3

NOTE:
FLOW RECORDS
AVAILABLE 1902 - 1967



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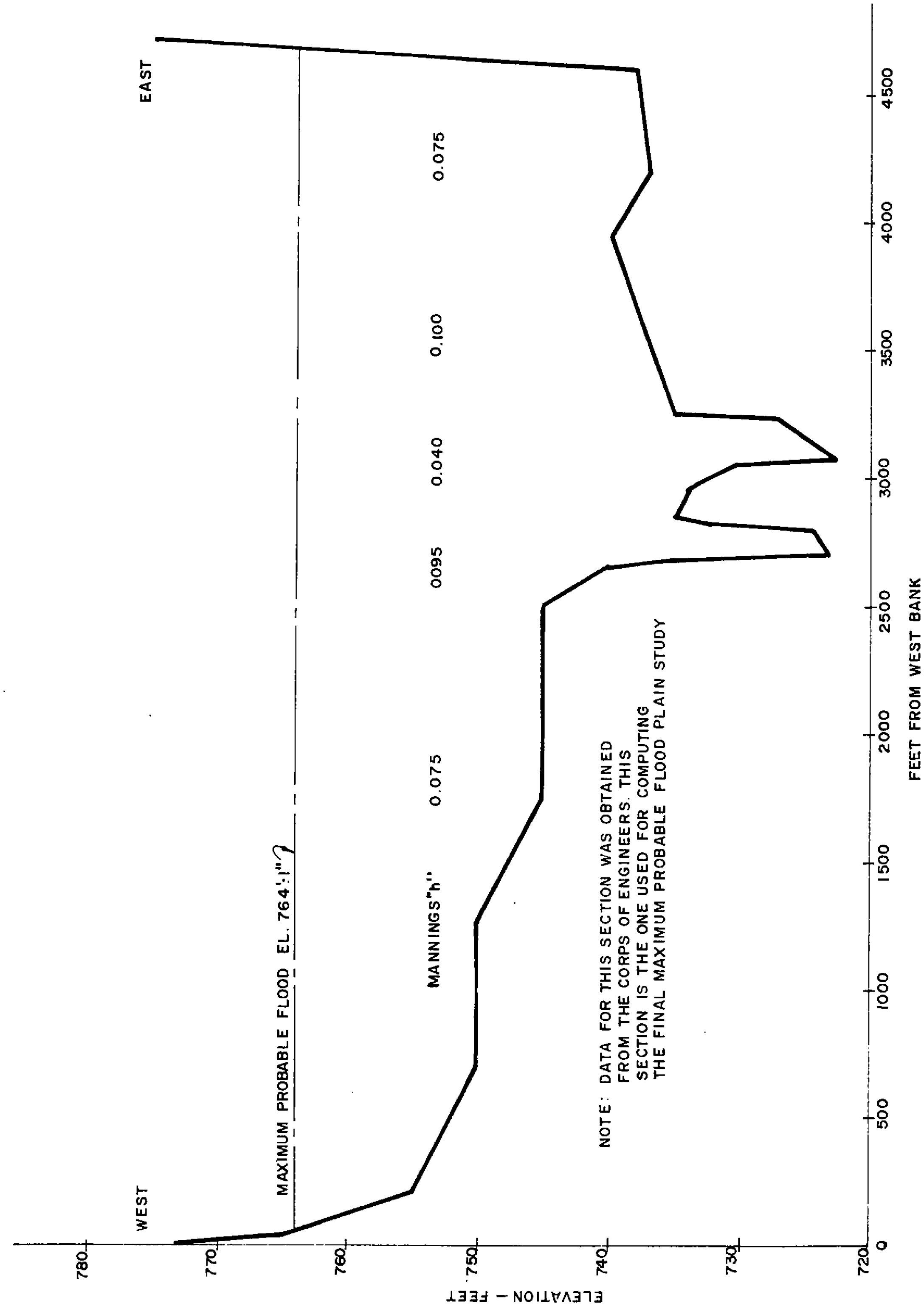
Existing Dams and Lakes in the Cedar River Basin

Figure 2.4-3a

Revision 2 - 6/84



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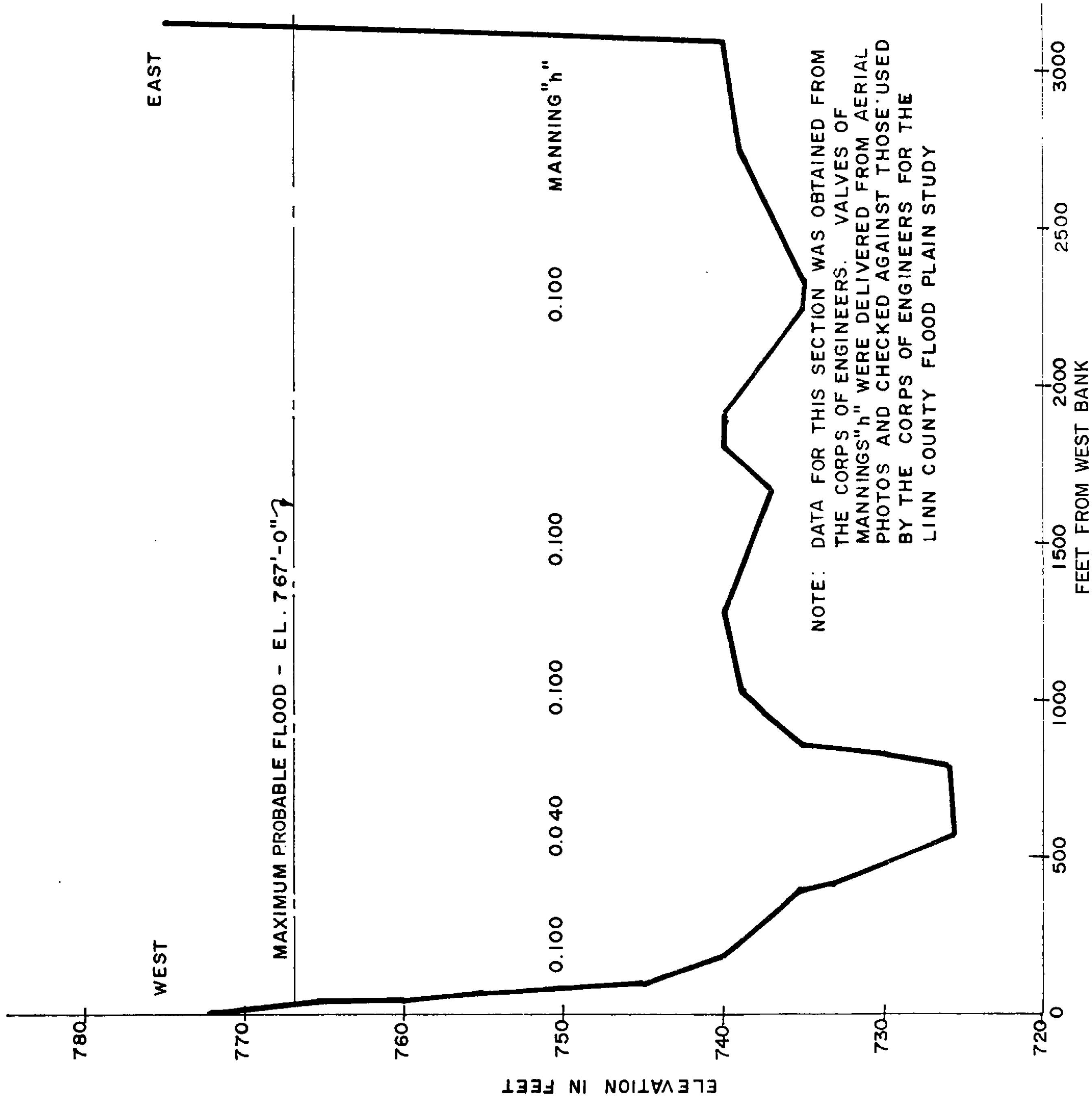
Cedar River Valley Cross Section at the DAEC-River-Mile Point 133.36

Figure 2.4-4



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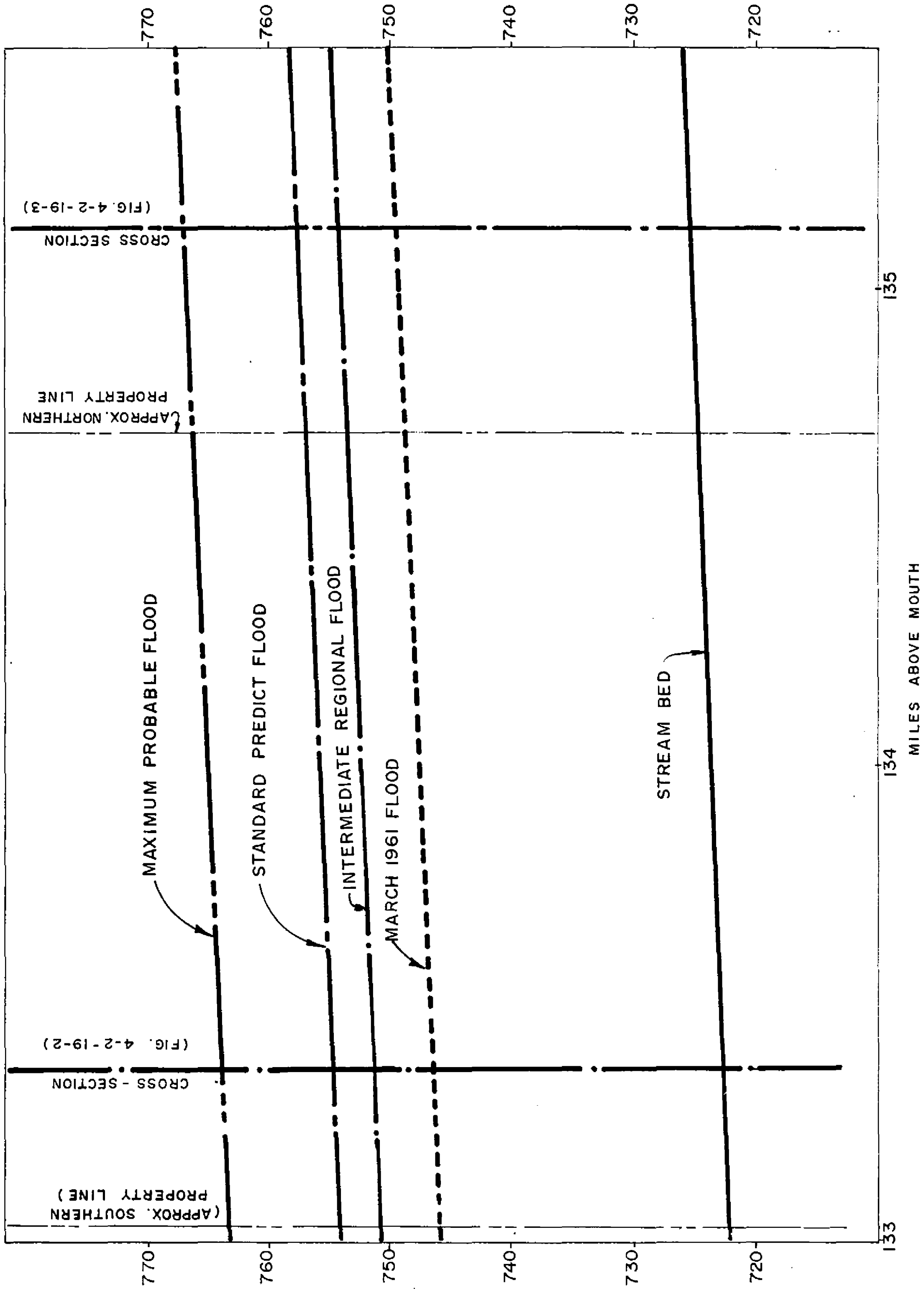
Cedar River Valley Cross Section
at Valley Constriction Upstream
of DAEC-Mile Point 135.13

Figure 2.4-5



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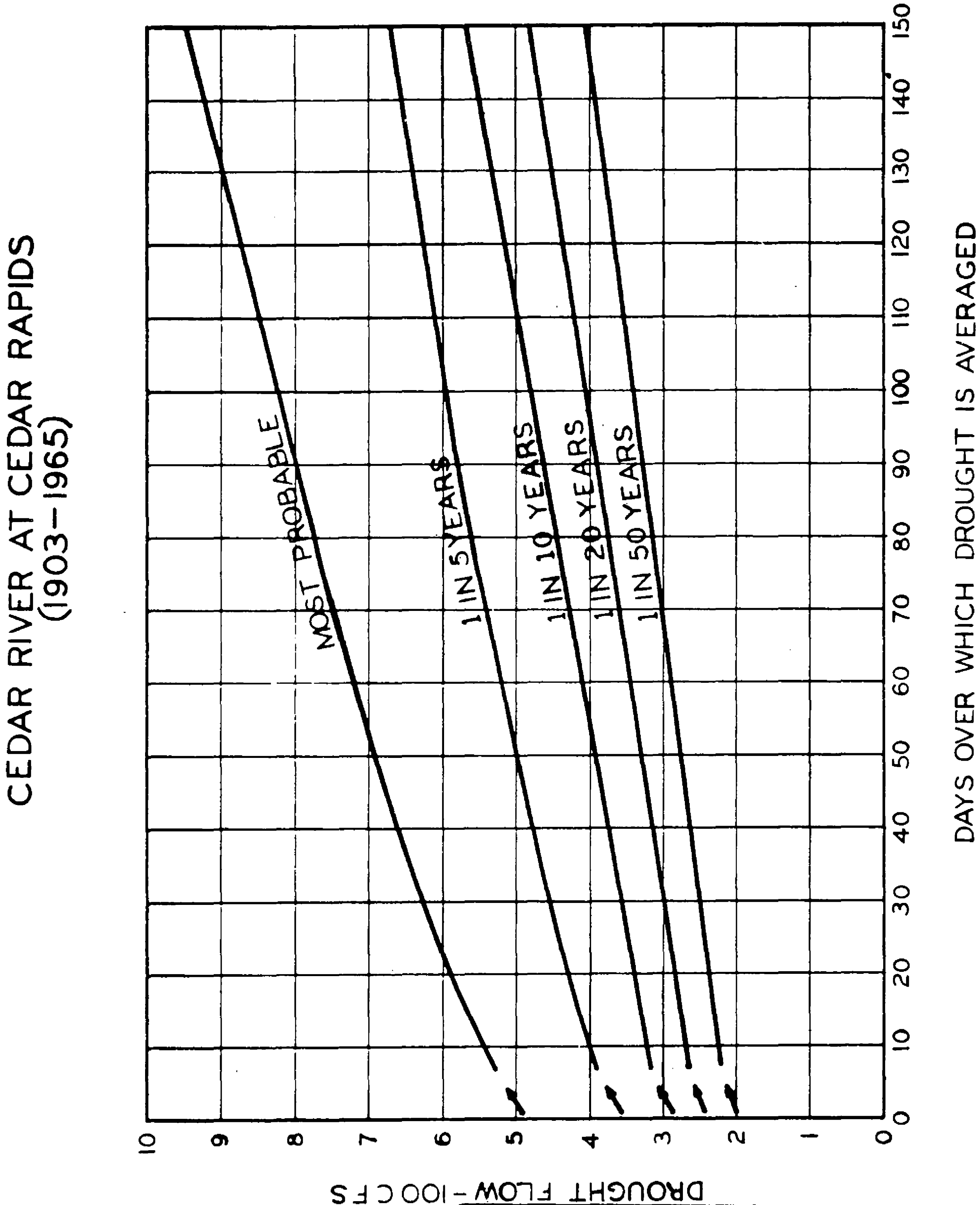
Cedar River Water Profile at DAEC Site

Figure 2.4-6



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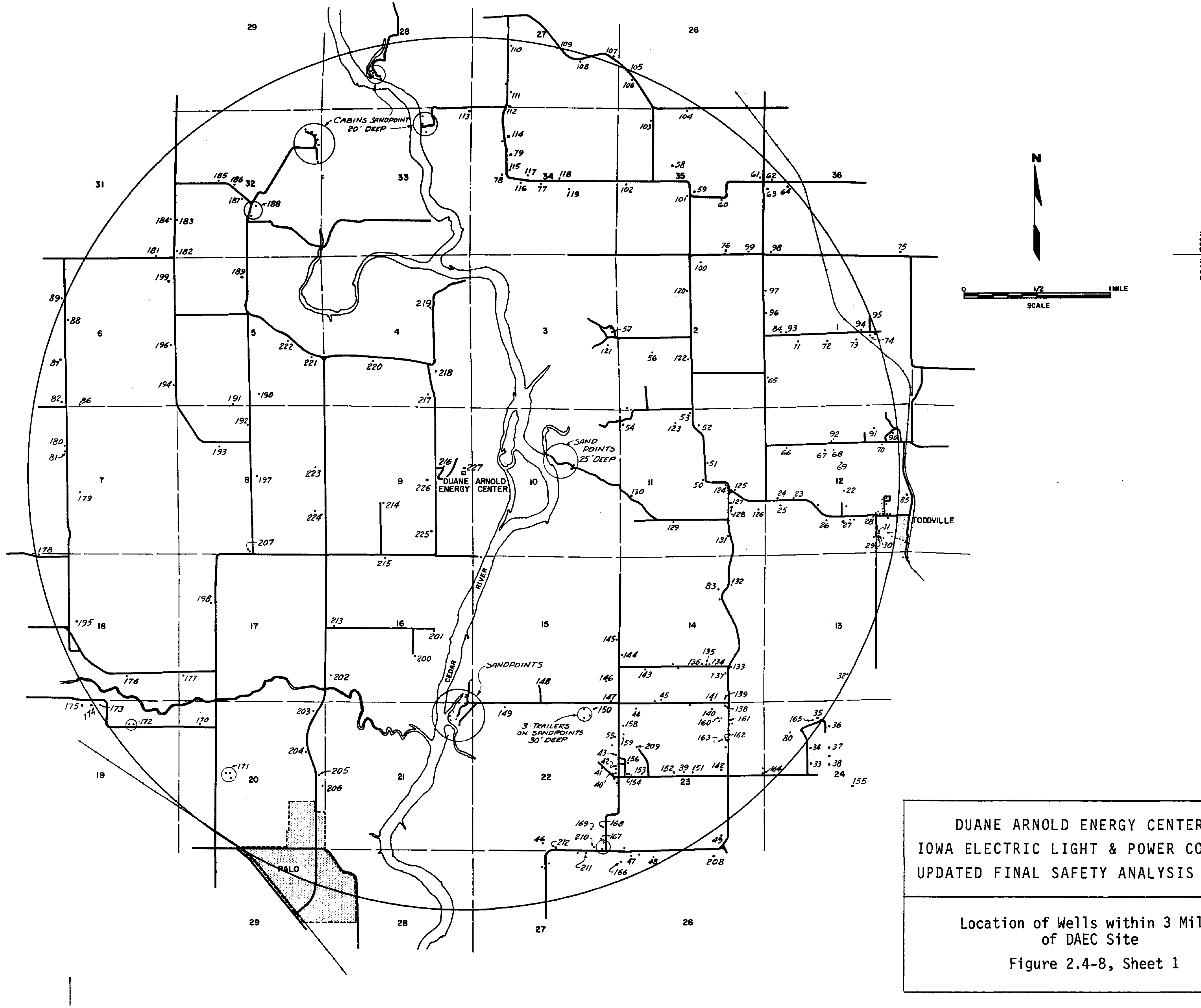
Drought Flow Cedar River at Cedar Rapids

Figure 2.4-7



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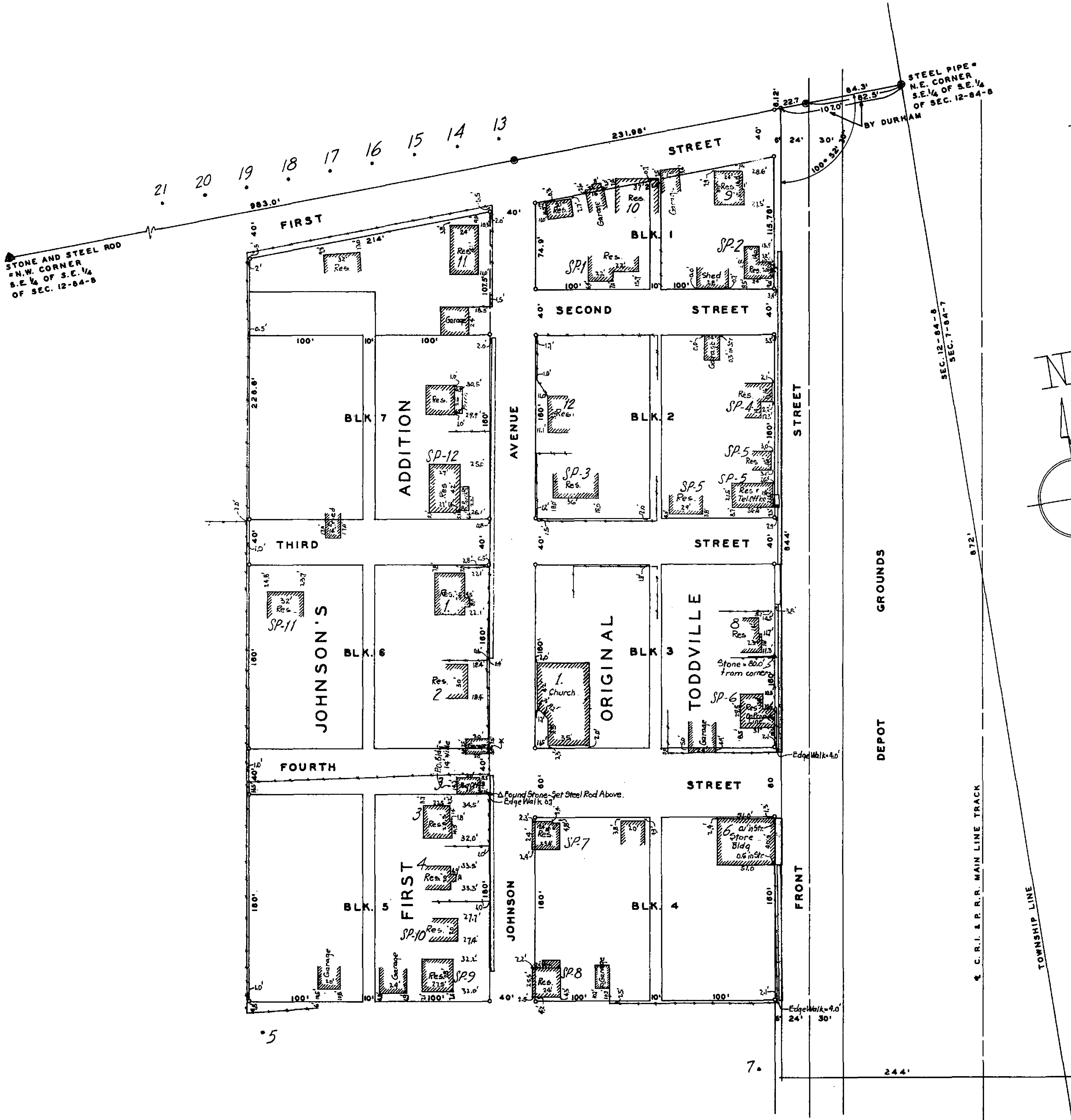
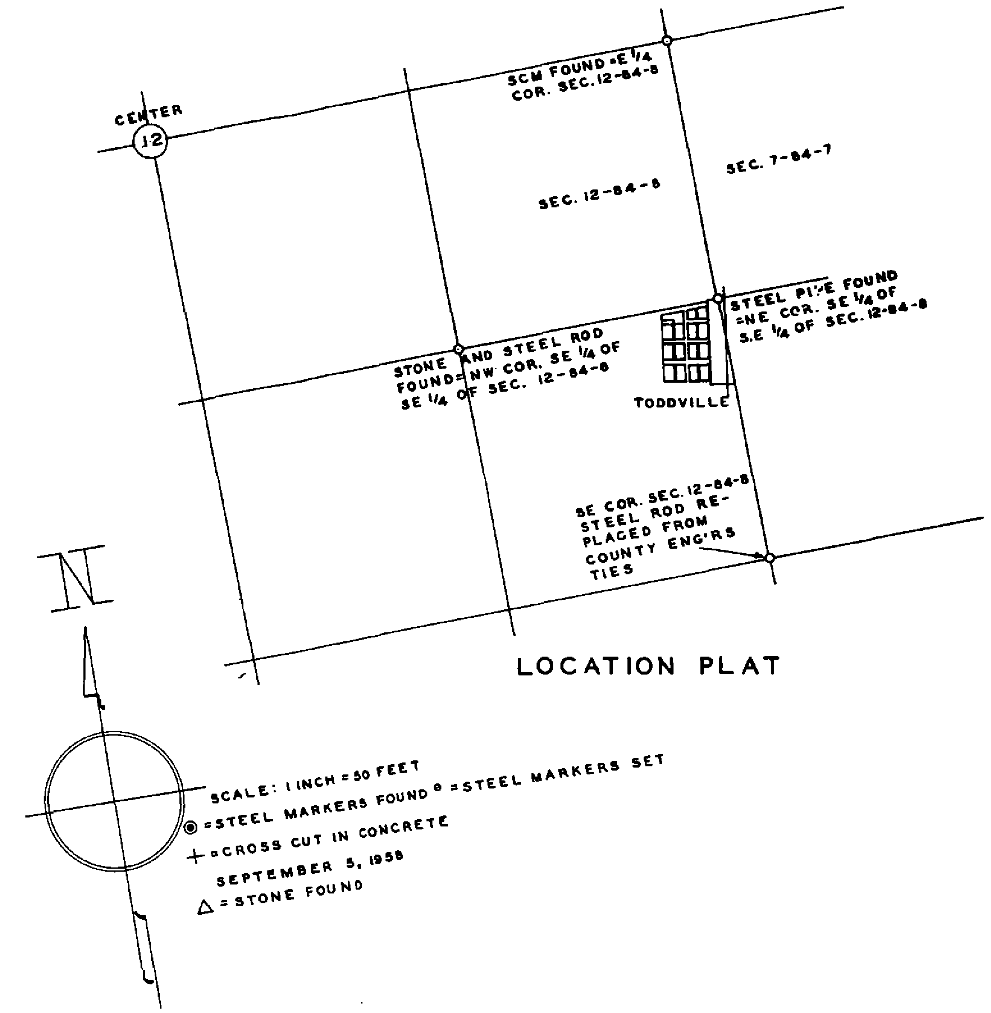
Location of Wells within 3 Miles of DAEC Site

Figure 2.4-8, Sheet 1





NOTE: S.P. = SAND POINT 27'-32' AVERAGE DEPTH

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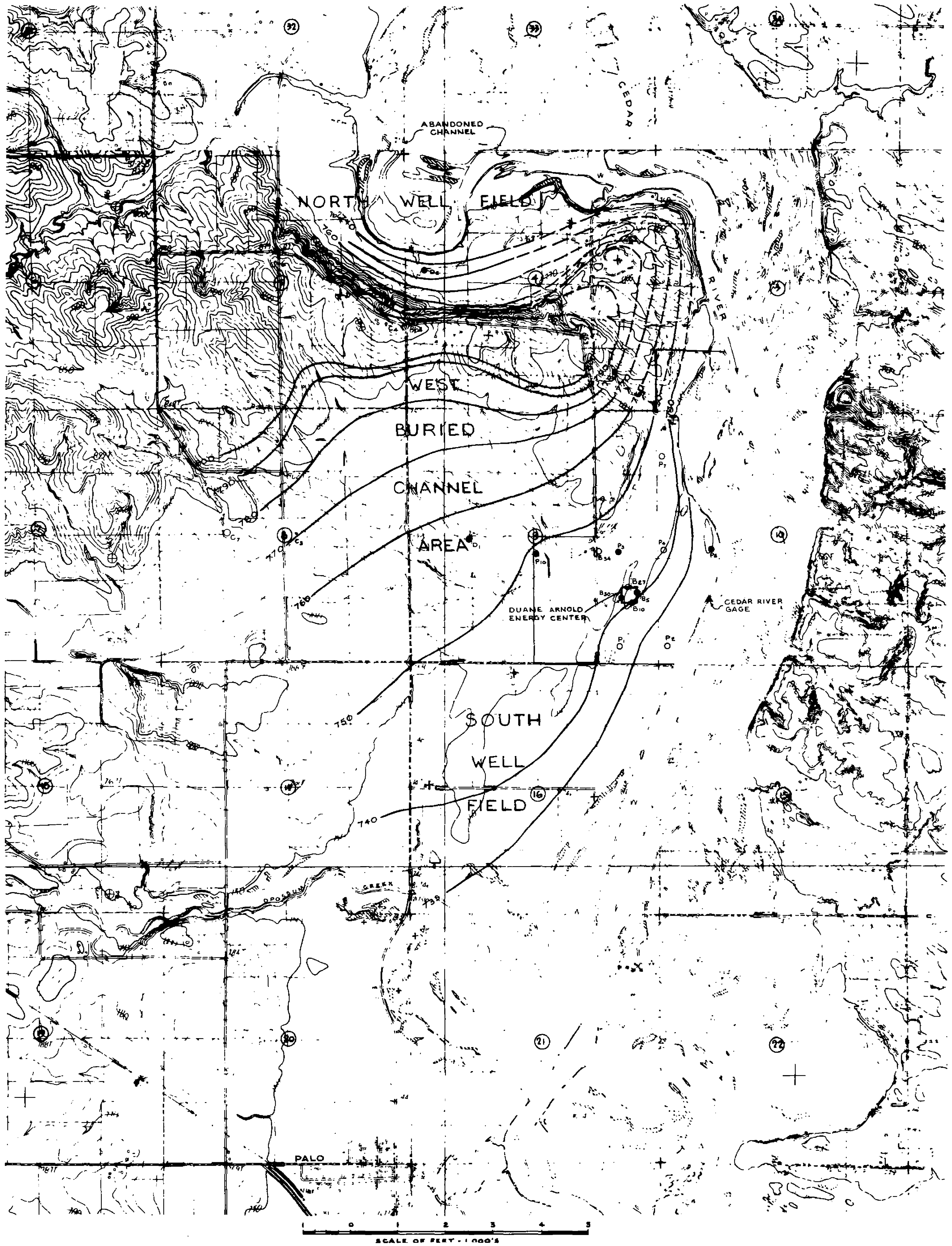
Location of Wells within 3 Miles of DAEC Site

Figure 2.4-8, Sheet 2



EXPLANATION

- ▲ STAFF GAGE - PERMANENT
- △ REFERENCE POINT
- ● BORING - OBSERVATION WELL (PLASTIC PIPE INSTALLED)
- ○ BORING - HOLE ABANDONED (NO PLASTIC PIPE)
- + DATA POINT

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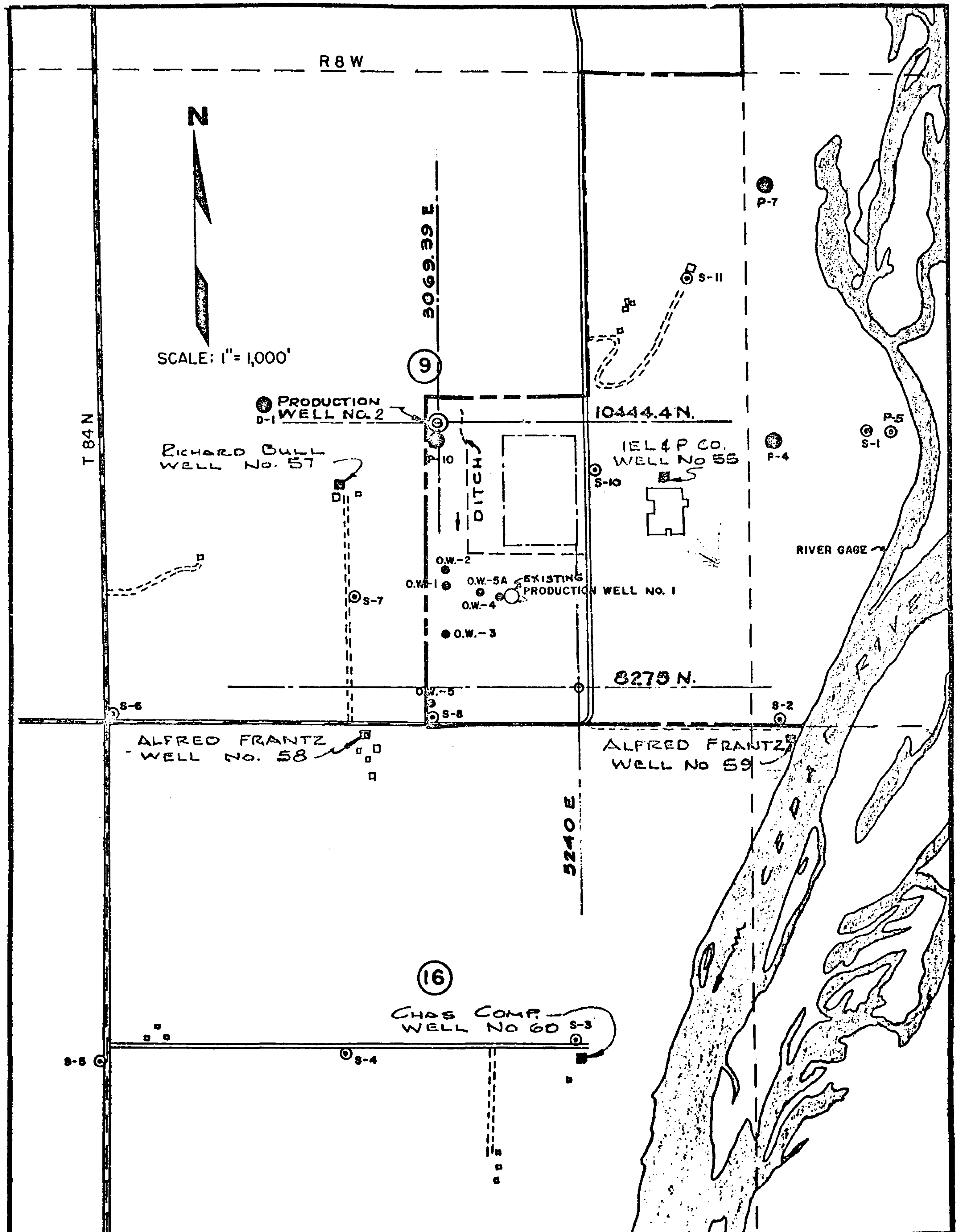
Water Table Map

Figure 2.4-9



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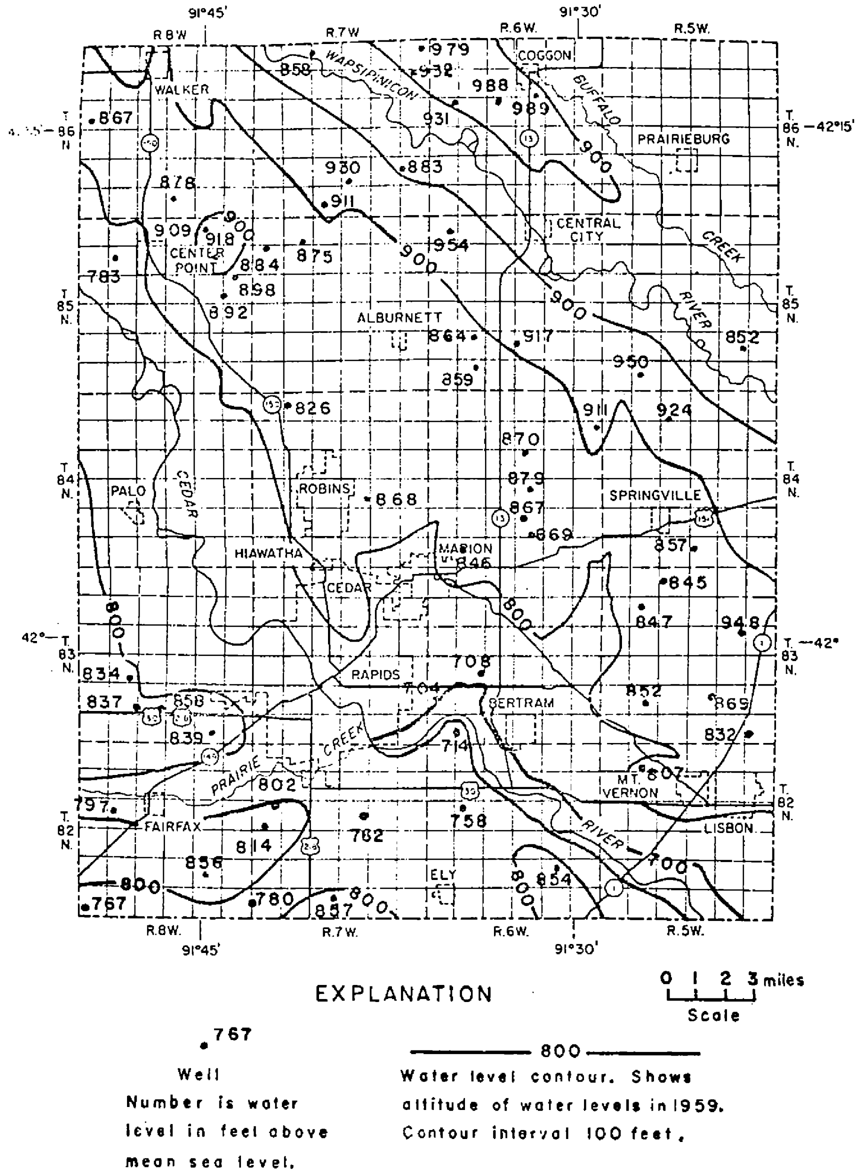
Location of DAEC Observation Wells

Figure 2.4-10



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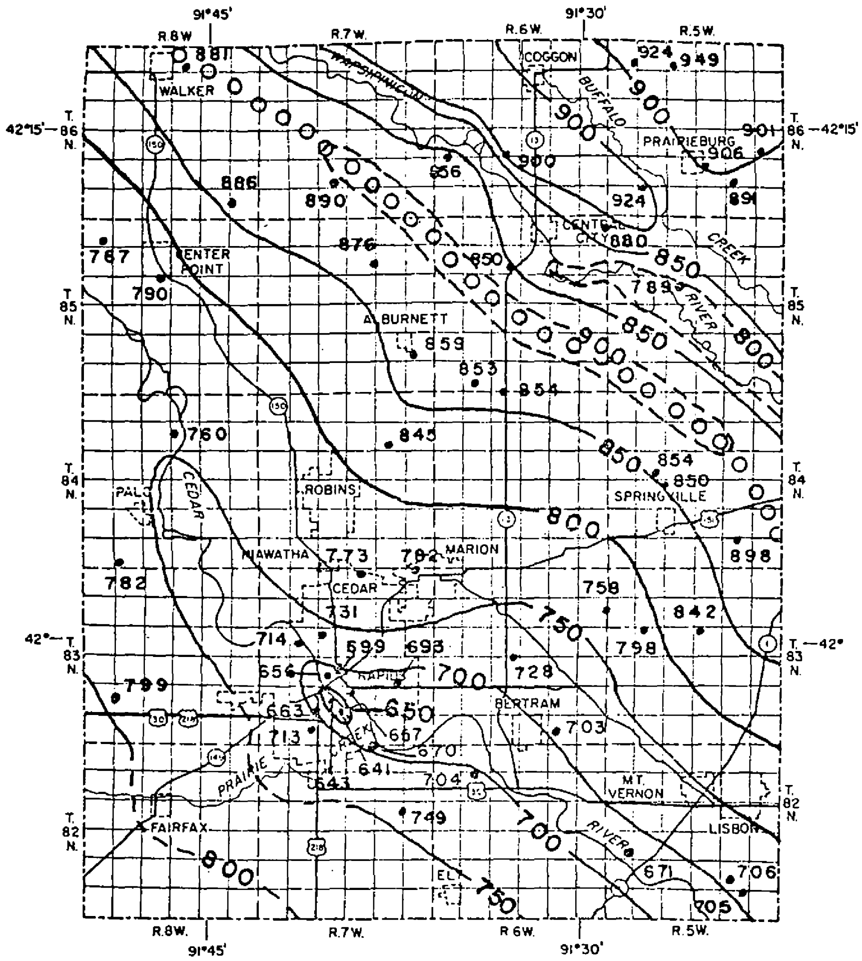
Water Table Map - Linn County

Figure 2.4-11



EXPLANATION

800 — — —

Piezometric contour shows altitude of piezometric surface. Dashed where approximately located. Contour interval 50 feet. Datum is mean sea level.

• 671

Well, number is altitude of piezometric surface in feet.

O O O O O O O O

Ground water divide

0 1 2 3 miles

Scale

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Piezometric Surface - Silurian-Devonian Aquifer-1959 Figure 2.4-12

2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING

2.5.1 BASIC GEOLOGIC AND SEISMIC INFORMATION

This section presents a summary of the geologic conditions at the site and in the region surrounding the plant. The plant is located near the town of Palo in Linn County, Iowa. The site is located adjacent to and west of the Cedar River, approximately 8 miles northwest of Cedar Rapids, Iowa.

The site lies within the Central Stable Region of North America, an area in which the geologic structure is relatively simple. The region is characterized by a system of broad, circular to oblong erosional uplifts and sedimentary basins that include the Wisconsin and Ozark Domes and the Forest City, Michigan, and Illinois Basins. Minor structures, consisting primarily of northwest-southeast trending synclines and anticlines of low relief, are superimposed on these broader features in the region. Precambrian crystalline basement rocks lie some 2600 ft below the ground surface in the vicinity of the site. The crystalline basement complex is mantled by sedimentary rocks of Paleozoic age. The bedrock surface at the site ranges in depth from approximately 25 ft to more than 100 ft and is, in turn, overlain by glacial till and surficial deposits of clayey silt, sand, and gravel.

Faults have not been identified within the basement rocks or overlying sedimentary strata in the vicinity of the site. The closest known faults are located approximately 17 miles southeast of the site and 10 miles north of the site. The vertical displacement of these faults is estimated to be about 20 ft. Other known faults are located at significantly greater distances from the site. Faults in the region are believed to have been dormant since late Paleozoic time, at least 200 million years ago. The Paleozoic strata and overlying consolidated sediments within about 100 miles of the site are essentially undeformed.

The field investigation performed for the geologic study (Section 2.5.4.3) revealed varying degrees of solution activity in the Devonian limestones and dolomites underlying the site. The solution activity ranged from the formation of very small vugs in the Spring Grove Member of the Wapsipinicon Formation, to a cavity about 12 ft deep within the Spring Grove, above its contact with the Kenwood Member.

There are no geologic features at the site or in the surrounding area that preclude the use of the site for a nuclear facility. The bedrock in the construction area is competent and will provide adequate foundation support for all major structures. Remedial measures have been taken to ensure satisfactory performance of the bedrock in cavity areas. Suitable rock exploration and treatment procedures are presented in conjunction with foundation design and construction data in Section 2.5.4.

The site geologic program included the following:

1. A thorough review of pertinent geologic literature (published and unpublished) and interviews with university, state, and Federal geologists.

2. A geologic reconnaissance of the site and surrounding area and an interpretation of maps and aerial photographs.

3. An investigation of subsurface soil, rock, and ground-water conditions by means of a test boring program, geophysical refraction surveys, and other related field studies.

The results of the geologic investigation are presented in the following sections. Detailed descriptions and results of the field explorations and laboratory tests are presented in Section 2.5.4.3. A list of publications reviewed and the organizations interviewed to obtain the information presented in this portion of the FSAR is presented at the end of this section.

A separate foundation investigation was conducted prior to construction of the low-level radwaste processing and storage facility. This investigation is discussed in Section 2.5.7.

2.5.1.1 Regional Geology

2.5.1.1.1 General

The site lies in the northern portion of the interior Lowland Physiographic Province, within the Central Stable Region of North America, south of the Canadian Shield. The region is characterized by a basement complex of Precambrian crystalline rocks overlain by a varying thickness of Paleozoic sedimentary strata. The sedimentary rocks are of Pennsylvanian age or older. During the Mesozoic and Cenozoic Eras, this region generally was above sea level and subject to erosion rather than deposition, which accounts for the absence of younger formations. Minor accumulations of Cretaceous sediments exist in western Iowa and have been reported in portions of western Illinois. These deposits have not been identified in eastern Iowa. During the Pleistocene Epoch, the stable interior of the continent was covered by continental glaciers. These glaciers scoured the bedrock surface and subsequently covered much of the region with glacial drift.

Postglacial erosion and deposition of alluvial and windblown deposits altered the landscape to its present form. Topography in the region is characterized by smoothly contoured land forms of low to moderate relief. Steep slopes are usually found only in areas where a river or stream has cut into the loess-covered hills.

2.5.1.1.2 Stratigraphy

The distribution of the major geologic units in the region is shown in Figure 2.5-1. The bedrock consists of unmetamorphosed sedimentary rocks that range in age from Cambrian to Pennsylvanian. In the vicinity of the site, the bedrock consists of the Gower Formation of Silurian age and the Wapsipinicon Formation of Devonian age. A detailed description of the bedrock stratigraphy in the vicinity of the site is shown in Figure 2.5-2. The geologic column reveals the following depositional sequence:

1. Coarse sandstones of Upper Cambrian age.

2. Alternating sandstones, limestones, and shales of Cambrian to Middle Ordovician age.

3. Alternating shales, limestones, and dolomites of Upper Ordovician to Middle Devonian age.

In general, the strata are thinner in the upper portions of the column; this condition is typical of sedimentation processes in the continental interior. Several outcrops of sedimentary rock were examined in the region. The closest outcrops inspected are about 3 miles southwest of the site.

Pleistocene glaciation, which occurred several hundred million years after deposition of the uppermost bedrock strata, mantled the entire region with unconsolidated sediments of variable thickness and composition. The distribution of surficial unconsolidated materials in the region is shown in Figure 2.5-3.

2.5.1.1.3 Tectonics and Structural Pattern Conclusions

Since Precambrian time, the stable interior platform of the United States has developed into a system of broad, circular to oblong erosional uplifts and sedimentary basins. The most prominent uplifts in the vicinity of the site are the Wisconsin Dome, Sioux Uplift (Trans-Continental Arch), and the Ozark Dome, to the northeast, northwest and south, respectively. Prominent basins in the region include the Salina and Forest City Basins, the Michigan Basin, and the Illinois Basin. Stable regions, those that have experienced relatively minor vertical movements, separate the deeper regional basins and generally connect the broad domes. These areas, within the region, include the Mississippi River Arch, the Kankakee Arch, and the Trans-Continental Arch. The locations of the various significant structures in the region surrounding the site are shown in Figure 2.5-4. The site is located on a stable shelf area between the Mississippi River Arch and the Forest City Basin, within an area that has experienced relatively minor vertical movements.

In marked contrast to the broad, relatively simple, regional geologic structures described above are the complex gravity and magnetic fields observed in surveys of the continental interior. A Bouguer gravity map of the region is included as Figure 2.5-5.

It has been pointed out by many authors that the high-gradient anomalies associated with the gravity and magnetic fields are due almost solely to density and magnetic contrasts in the Precambrian crystalline basement rocks. It has been further suggested that the distribution of mass anomalies provides the basic mechanism for the subsequent development of basins and uplifts. Regions of high density have subsided and formed basins, whereas low-density regions have formed uplifts. The interconnecting arches (stable areas) have intermediate densities.

Although the significant structural features in the region have been identified and located, innumerable minor structures have been superimposed on these broader features. These minor structures, in the vicinity of the site, generally consist of a series of northwest-to-southeast trending synclines and anticlines having general relief of less than 150 ft.

Most of the faults in the region formed during the development of the uplift-basin-arch configuration of the Central Stable Region. It is believed that these faults are essentially vertical and developed along crustal heterogenetics as a result of imposed stresses of Paleozoic age. Early geologic studies have ascribed tectonics in the continental interior to lateral compressive stresses during the Appalachian and Ouachita orogenies. Subsequently, later work has indicated that most of the tectonics were the result of Paleozoic isostatic movements. Therefore, most of the observed faulting would have been caused by tensional stresses. In general, faults developed along the flanks of stable arches adjacent to a basin or dome.

Faults within the region that appear to have formed in Paleozoic times include the Sandwich Fault, associated with the La Salle Anticlinal Belt along the southwest flank of the Kankakee Arch, and the La Platte and Humboldt Faults, associated with the Nemaha Uplift separating the Forest City and Salina Basins. These faults are approximately 120 miles east and southwest of the site, respectively. The Thurman-Redfield Structural Zone is a minor feature that extends northeasterly from the Nemaha Uplift. Minor faulting of about 30 ft is known to exist on the Missouri River associated with this structural zone. This structural zone is apparently developed along the southeastern edge of the Mid-Continent Anomaly and is presently known to extend from the Kansas-Iowa border northeastward to just north of Des Moines, Iowa. It is not presently known whether this zone continues further north along the Mid-Continent Anomaly to connect with the small faulting near Hastings, Minnesota.

The only significant example of faulting that did not occur during the development of the uplift-basin-arch configuration of the Central Stable Region is the Ste. Genevieve-Cottage Grove-Shawneetown-Rough Creek fault complex that extends from central Missouri, easterly, through southern Illinois and into Kentucky. This system of faults lies some 300 miles southeast of the site. These fault zones intersect major structures including the Ozark Dome and the Illinois Basin. Since the faults are imposed on the regional structure, they must postdate the development of the Paleozoic uplift-basin-arch

system. It has been suggested that this fault system is a hinge line separating the stable continental interior to the north from the subsiding Mississippi Embayment. There is no geologic evidence to relate this fault system with any structure or faulting within the continental interior.

No known faults exist within the basement rock or overlying sedimentary strata in the vicinity of the site. The closest known faults are located approximately 10 miles north of the site, and about 17 miles southeast of the site. These faults have only minor vertical displacements.

A few other faults are known in Iowa. These are located near the cities of Washington, Ottumwa, Decorah, and Fort Dodge. The maximum displacement associated with these faults is at Washington where at least 200 ft of displacement is evident.

Vertical crustal movements in the stable interior occurred during the ice loading and rebounded with glacial retreat. Since the site area has undergone multiple Pleistocene glaciation, it may be inferred that this region has been subjected to repeated slight bending in the last few hundred thousand years. Present-day adjustment in the site area is nonexistent, as far as is known. However, slight movements, indicated by earthquakes along dormant Paleozoic fault zones, could represent a deep crustal readjustment to glacial advance and retreat.

Based on the geologic research and interpretation of data outlined above, it was concluded that there was no structural feature in the region to preclude the use of the site for a nuclear power plant.

2.5.1.2 Site Geology

2.5.1.2.1 Bedrock

The bedrock strata immediately underlying the site are the Wapsipinicon and Gower Formations, of Middle Devonian and Upper Silurian age, respectively.

The Gower Formation is a grayish-brown, finely crystalline to aphanitic limestone with occasional vugs. A second facies in the Gower consists of a massive, light gray, highly porous, vuggy, and fossiliferous limestone that is a biohermal (reef) type deposit. The vugs have formed as a result of the solution of organic shells during dolomitization. The top of the Gower is an erosional surface on which the Wapsipinicon was deposited. Sinks that developed on this erosional surface were subsequently filled with Wapsipinicon sediments. It is estimated that the Gower Formation is approximately 60 to 100 ft thick at the site.

Although the Wapsipinicon Formation, regionally, contains six members, only three were identified at the site during the boring program. The three lower members of the Wapsipinicon Formation, the Otis, the Coggon, and the Bertram, are not present.

The lowest member present at the site, the Kenwood, is a gray, crystalline, dolomitic limestone that grades to argillaceous limestone and is interbedded with bluish-green calcareous shales. Locally this member is known as a "garbage" zone because of its varying composition. The lower beds of the Kenwood are composed of greenish shales that mark the contact between rocks of Devonian and Silurian age. The Kenwood ranges up to approximately 30 ft in thickness at the site.

The Spring Grove Member lies above the Kenwood and is a light grayish-brown, finely crystalline to aphanitic dolomitic limestone. It is highly vulgar in places and is generally very massive. The member contains no identifiable fossils. This member ranges up to 27 ft thick at the site.

The upper member of the Wapsipinicon, which forms the bedrock surface in the plant area, has been identified as the Davenport. It is a light grayish-brown, aphanitic limestone that includes several solution-breccia zones and a 0.5-ft thick bed of dolomitic limestone. It is thinly bedded and is usually argillaceous. This member is about 10 ft thick in the central portion of the site, but has been eroded and is absent in the bedrock valleys.

The bedrock surface was encountered at depths ranging from approximately 25 ft to more than 100 ft below the existing ground surface. The configuration of the bedrock surface is shown in Figure 2.5-6. A bedrock high is centered directly beneath the plant. This bedrock high is probably a result of preglacial erosion modified by interglacial erosion. Weathered bedrock, found in the valleys, and hard unweathered rock, encountered on the highs, is attributed to differential glacial sour. In general, the rock encountered at the site is relatively fresh and hard.

Two zones of cavity development were observed in the Wapsipinicon Formation during this investigation. One is within the Davenport limestone, usually just above the Spring Grove-Davenport contact. The second is in the Spring Grove, just above its contact with the underlying Kenwood. Other relatively minor solution activity was observed in some enlarged joints of the Kenwood and in enlarged vugs of the Spring Grove. No cavities were encountered in the Gower Formation during this investigation.

The major cavity observed in the Spring Grove Member is in the vicinity of Boring 21. Nine probe holes were drilled around Boring 21, and five of these penetrated the cavity. On the basis of these explorations, it is estimated that this cavity is approximately 12 ft deep at the maximum. The top of the cavity ranges from about 12 to 20 ft below the rock surface. The trend of this cavity is generally northeast to southwest. The cavity was more than half-filled with Pleistocene soils consisting of clay and silt with some organic debris.

A second area of solution activity was encountered in the lower part of the Davenport Member, in the northern portion of the site near Borings 18, 25, 26, and 32. These cavities were small, usually less than 2 ft in size.

Cavity development may be accounted for by the fact that the site is located on a bedrock high that was formerly subjected to the action of downward percolating preglacial waters. The bedrock high beneath the site probably was exposed before till cover, and the cavities apparently developed before glaciation. The subsurface water in the bedrock is presently artesian, probably precluding present-day solution activity.

2.5.1.2.2 Soil Conditions

A clay till containing some sand and gravel interspersed in the clay matrix directly overlies the bedrock surface. The till has, at various times, been described as both Kansan and Iowan, the latter being early Wisconsinan in age. The till thickness varies from 12 to 80 ft in the site area. The till thickness averages 20 ft in the plant area.

Flood plain deposits averaging about 20 ft in thickness and consisting of fine to coarse sand, with some silt and gravel, form the surficial material at the site. Hills adjacent to the site are composed of till mantled with loesses of Wisconsinan age.

A general description of the soils and rocks encountered at the site is shown in Figure 2.5-7. Logs of the geologic and foundation borings are included in Section 2.5.4.3. Logs of the borings made for the low-level radwaste processing and storage facility are provided in Section 2.5.7.

2.5.2 VIBRATORY GROUND MOTION

This section presents a summary of the seismologic studies performed at the site and in the region surrounding the plant. The plant is near the town of Palo, in Linn County, Iowa.

Detailed descriptions and the results of the field explorations and laboratory tests performed in connection with the seismologic studies are presented in Section 2.5.4.3. This site is located in one of the most seismically stable regions in the United States. There has been no earthquake epicenter reported closer than about 75 miles and only 15 earthquakes were reported within about 200 miles of the site since the beginning of the nineteenth century. The intensities of these earthquakes were not greater than Intensity VII. (All intensity values in this section refer to the Modified Mercalli Scale revised in 1956. The intensity scale, which is described in Table 2.5-1, is a means of indicating the relative size of an earthquake in terms of its perceptible effect). The closest reported earthquake was a 1934 Intensity VI shock near Rock Island, Illinois. This earthquake caused some slight damage near its epicenter, but was not felt in the vicinity of the site. One or both of two 1909 Intensity VII shocks in northern and central Illinois may have been felt in the vicinity of the site. However, the effects of these shocks, in Iowa, were not great and no damage resulted.

Most of the few reported earthquakes in the region are associated with well-defined geologic structural zones. To the east of the site, earthquakes are related to faulting in southern Wisconsin and northern and central Illinois. To the west of the site, earthquake activity is related to uplifted areas in eastern South Dakota, eastern Nebraska, and northeastern Kansas. None of these structural zones associated with earthquake activity extends into Iowa. There are no known faults within 10 miles of the site.

Significant earthquake ground motion is not expected at the site during the life of the plant. However, Seismic Category I structures are conservatively designed to respond elastically, with no loss of function (operating basis), to horizontal foundation level accelerations of

1. Six percent of gravity, if supported on rock or on about 10 ft of compacted fill and/or natural glacial soils, soil cement fill, or lean concrete fill.

2. Nine percent of gravity, if supported on about 30 to 50 ft of compacted fill and/or natural glacial soils.

Foundation level response spectra (horizontal) for the operating and design-basis earthquakes are presented in Figure 2.5-8. For structures on rock or about 10ft of soil, the vertical design accelerations shall be 80% of the appropriate horizontal foundation level accelerations. For structures on about 30 to 50 ft of soil, the vertical design accelerations shall be two-thirds of the appropriate horizontal foundation level accelerations.

The purposes of this study were as follows:

1. Evaluate the seismicity of the area.

2. Evaluate the effect of earthquake motion on the foundation materials at the site.

3. Develop seismic design parameters.

The seismic program included the following:

1. Literature research to compile a record of the seismicity of the area.

2. A comprehensive geologic study to evaluate the geologic structure and tectonic history of the region.

3. Field explorations to measure in-situ dynamic properties.

4. A program of dynamic laboratory testing and analyses to evaluate the response of the foundation materials to earthquake-type loading.

Detailed description and results of the field explorations and laboratory tests, which provide background information, are presented in Section 2.5.4.3.

Geophysical studies were performed at the site to aid in evaluating the dynamic properties of the natural subsurface materials. The dynamic soil properties were used in evaluating the response of these materials to earthquake loading. The results of the field geophysical studies are presented in Section 2.5.4.3.

A seismic refraction survey and an uphole velocity survey were performed to measure the velocity of compressional wave propagation at the site. Micromotions were measured to indicate the pattern of vibration at the site on the basis of ambient background vibration analyses. These measurements are of assistance in estimating any predominant natural period of vibration at the site. Poisson's ratios for the various materials in the stratigraphic section at the site were estimated from empirical data for similar materials. An attempt was also made to develop shear wave velocity from the field measurements of surface waves; however, this was not successful.

Shear wave velocities for the shallower materials at the site were computed using the measured compressional wave velocities and estimated Poisson's ratios. Compressional wave velocities for the deeper rock strata were not measured. Therefore, estimates were made of both the compressional wave velocity and Poisson's ratio (based on measurements in similar material), and the corresponding shear wave velocities were then computed. Geophysical data for the entire stratigraphic section are presented in Figure 2.5-9.

2.5.2.1 Seismicity

The site is situated in an area that has experienced very little earthquake activity. No earthquake epicenter has been reported closer than about 75 miles to the site. Since the region has had a permanent population for over 100 years, it is probable that all earthquakes of about Intensity V or greater would have been reported during this period. Any major earthquake (Intensity IX or greater) that occurred during the time the region has been sparsely populated (300 or so years) probably would have been reported in the local newspapers or in private journals or diaries. The absence of such documentation is indicative of the absence of significant earthquake activity in the region during this period.

The zone of major earthquake activity closest to the site is in the vicinity of New Madrid, Missouri, approximately 400 miles to the southeast. Earthquakes near New Madrid in 1811 and 1812 are considered among the largest ever to have occurred in the United States. Intensities in the region of the site were probably on the order of I to III. It is reported that these shocks were felt in an area of 2 million mi^2 and changed the surficial topography in an area of about 30,000 to 50,000 mi^2. The structural damage resulting from these earthquakes was small because of the lack of construction and habitation in the region. However, it is undeniable that these were major earthquakes. These earthquakes are probably related to the extensively faulted Ste. Genevieve-Cottage Grove-Shawneetown-Rough Creek fault complex which extends from eastern Missouri to western Kentucky.

The geologic structure in eastern Missouri, southern Illinois, and western Kentucky is not related to the geologic structure in the vicinity of the site. The Ste. Genevieve fault complex crosses major regional structures and probably forms a hinge line separating the stable continental interior to the north from the subsiding Mississippi Embayment. There is no geologic evidence to relate this fault system with structure or faulting within the continental interior. Thus, the seismically active region at the hinge line and to the south should be considered dissimilar and distinct from the seismically quiet region to the north.

Most of the earthquakes reported in the region are associated with zones of well-documented structure. To the east of the site, earthquakes are related to faulting in southern Wisconsin and northern and central Illinois. To the west of the site, earthquake activity is related to uplifted areas in eastern South Dakota, eastern Nebraska, and northeastern Kansas. None of the structural zones associated with earthquake activity extends into Iowa.

Several earthquakes, which are probably related to faulting associated with the Abilene Arch and Nemaha Uplift, have occurred in northeastern Kansas. The largest of these was an 1867 Intensity VII shock with its epicenter near the towns of Manhattan and Lawrence, Kansas, about 300 miles from the site. This shock was felt in an area of about 300,000 mi^2 in the states of Illinois, Indiana, Missouri, Nebraska, Arkansas, Kentucky, and possibly Ohio. The shock also was probably felt in Iowa, but there are no reports of this. The maximum damage near the epicenter of this shock was minor, consisting primarily of cracked plaster and walls.

Only four earthquakes have been reported within 100 miles of the site and only 15 earthquakes reported within about 200 miles of the site since the beginning of the nineteenth century. None of these shocks was greater than Intensity VII. Few were of high enough intensity to cause structural damage, and only two of these shocks can be considered more than minor disturbances. These were two 1909 Intensity VII earthquakes in northern and central Illinois. The epicenter of each shock was about 150 miles from the site. The closest reported earthquake to the site was a 1934 Intensity VI shock near Rock Island, Illinois. Although one or more of these shocks may have been felt in the locality of the site, no damaging effects were experienced (intensities were on the order of III or less).

A list of earthquakes with epicenters located within a distance of about 200 miles from the site is presented in Table 2.5-1a. These epicenters as well as the epicenters of more distant regional earthquakes are shown in Figure 2.5-10.

For purposes of this investigation, it is considered that the most significant earthquakes in the region are the 1934 Intensity VI earthquake near Rock Island, Illinois, and the 1909 Intensity VII earthquakes in northern and central Illinois.

This evaluation has been made considering such factors as epicentral intensity (with regard to both damage to structures and perceptible area of effect), distance from the site, and geologic structure (with regard to possible relationship of geologic structure near the earthquake epicenter to structure near the site).

The earthquake of November 12, 1934, occurred at approximately 8:45 a.m. near the town of Rock Island, Illinois. The duration of the shock was very short with the major ground motion consisting of a single strong jar. Damage near the epicenter was minor. The only damage of consequence was a dislodged stucco cornice that fell from a school in Rock Island. There were also reports of loose plaster shaken from a few buildings at Augustana College in Rock Island. On the basis of damage reports, the epicentral intensity of this earthquake was probably a low VI. The earthquake was felt in a northeast-southwest trending elliptical area about 100 miles long and 65 miles wide. The shock was not felt in the vicinity of the site. A magnificent feature of this shock was a very loud explosive sound accompanying the ground motion. This shock has not been related to a known major tectonic feature.

The earthquake of May 26, 1909, occurred at about 8:38 a.m. It was felt in a relatively large and irregular area extending from the Wisconsin-Illinois border as far south as Bloomington, Illinois. The total perceptible area of this shock was on the order of 500,000 mi^2. The duration of shaking of this earthquake was probably about 15 to 30 sec. The maximum damage from this earthquake consisted of many fallen chimneys in the Aurora, Illinois, area where the earthquake was reported to be "just under the point of damage to buildings." In Chicago, buildings swayed; the effect being most noticeable in the upper stories. There was no structural damage, although there was fear that walls would collapse. The maximum intensity of this shock was VII. The shock was felt in the vicinity of the site and was reported in Cedar Rapids newspapers (about Intensity III). The effect in Iowa was not great and no damage resulted. It is probable that this earthquake was related to the Sandwich Fault in northern Illinois. This fault is limited and does not project west into Iowa. Possible future earthquakes associated with the Sandwich Fault would occur no closer than 125 miles to the site, the westerly limit of the fault.

The earthquake of July 18, 1909, occurred at about 10:34 p.m. in the vicinity of Havana and Petersburg, Illinois, north of Springfield. The shock was felt in an area of about 40,000 mi^2. The maximum damage from this earthquake consisted of fallen chimneys at Petersburg, Illinois; Hannibal, Missouri; and Davenport, Iowa. Near Petersburg, more than 20 windows were broken, brick was pushed out over doors, and plaster was cracked. On the basis of damage reports and perceptible area, the epicentral intensity was probably a low VII. The epicenter of this shock is on the edge of the Illinois Basin. The shock cannot be related to any specific fault or fault system.

2.5.2.2 Geologic Structures and Tectonic Activity

The site is located in the Central Stable Region of North America, between the Wisconsin Dome and the Forest City Basin. On the basis of the available geophysical data, this area is a single crustal block. Precambrian crystalline rocks form the basement complex. These rocks rise to the bedrock surface in an area to the north defined as the Canadian Shield and extend to depths of nearly 14,000 ft in the sedimentary basins of the continental interior. In the immediate vicinity of the site, approximately 2600 ft of Paleozoic sedimentary rocks overlie the Precambrian crystalline basement complex.

Since Precambrian time, the stable interior platform of the United States has developed into a system of broad, circular to oblong erosional uplifts and sedimentary basins. Stable arches (platforms), which have experienced relatively minor vertical movements, separate the deeper regional basins and connect the broad domes. Most of the faults in the region were formed during the development of the basin-dome-arch configuration of the Central Stable Region during Paleozoic time. A significant exception is the Ste. Genevieve-Cottage Grove-Shawneetown-Rough Creek fault complex that extends from central Missouri to western Kentucky. Since these faults are imposed on the regional structure, they must postdate the development of the Paleozoic basin-dome-arch structure in the region. Subsequently, crustal movements in the region occurred during Pleistocene time. Since the area has undergone multiple Pleistocene glaciation, it may be inferred that this region has been subjected to repeated slight bending in the last few hundred thousand years. Faults have not been identified in the immediate vicinity of the site. The closest known faults are located approximately 10 miles north of the site and about 17 miles southeast of the site. Other faults or structural zones in the region that may influence the seismicity of the site are discussed in the geology section and are shown in Figure 2.5-10. The most significant geologic structures are described below:

1. The Sandwich Fault, trends northwesterly in north-central Illinois, approximately 130 miles east of the site. Maximum displacement along the fault is about 600 ft. The Savannah-Sabula Anticline, probably an unfaulted extension of the Sandwich Fault, extends westward from the western extremity of the Sandwich Fault. Gravity anomalies indicate that the crustal block along which the fault and anticline formed cannot be extended into the site area. The La Salle Anticlinal Belt forms the southerly extension of the Savannah Anticline-Sandwich Fault structural zone and is seismically inactive.

2. The Nemaha Ridge, with the associated Abilene Arch and La Platte and Humboldt Faults, forms a structural zone some 200 miles west of the site. This structural zone parallels a segment of the Mid-Continent gravity high, which can be traced from Oklahoma northward to the western tip of Lake Superior. The location of the gravity high and its apparent relationship to this structural zone indicate that the Nemaha Ridge is separate from and unrelated to the tectonics of the site area.

3. The Thurman-Redfield Structural Zone extends northeasterly from the Nemaha Ridge approximately 100 miles and has a maximum throw of about 30 ft. Based on the available data, the structure may terminate about 100 miles west of the site, coinciding with the southeast bounds of the Mid-Continent gravity high in southeast Iowa. There is no geologic or geophysical evidence to indicate that this structural zone extends further east than Des Moines, Iowa; however, even if it did extend into Minnesota the conclusions drawn in this section would not be changed. The zone may be the result of shear that occurred during the development of the Nemaha Ridge.

4. The Cap au Gres Fault, about 200 miles south of the site, trends west and northwest. Small, northwest trending folds south and east of the site indicate that this structure cannot be extended into the region of gentle folds and slight crustal tilt of the site area.

5. The Ste. Genevieve-Cottage Grove-Shawneetown-Rough Creek fault complex in eastern Missouri, southern Illinois, and western Kentucky approaches to within about 300 miles south of the site. This structural belt, which is seismically active, separates the stable Continental Platform, on which the site is located, from the gradually subsiding Mississippi Embayment to the south. This post-Paleozoic structural belt is unrelated to the Paleozoic tectonic features of the site area.

2.5.2.3 Correlation of Earthquake Activity with Geologic Structures or Tectonic Provinces

Refer to Section 2.5.2.2.

2.5.2.4 Maximum Earthquake Potential

See Section 2.5.2.6.

2.5.2.5 Seismic Wave Transmission Characteristics of the Site

Deformation moduli and damping characteristics of the foundation materials at the site are presented here. These moduli and damping factors were used for the design of the major structures to resist earthquake loading.

Since soil is not a truly elastic medium, the commonly accepted terminology of modulus of elasticity and modulus of rigidity are not completely applicable. However, for ease of subsequent discussion, these terms will be used to describe soil properties that follow the general definitions used for elastic media. Although soils are not a fully elastic medium, the assumption of stress-strain linearity can usually be made for a particular stress level range. Thus, the assumption of elasticity theory is fairly suitable for use in measuring moduli of elasticity and rigidity. For competent rock, the assumption of a linear stress-strain relationship is generally quite good.

The moduli and damping values selected for the foundation materials at the site are presented in Table 2.5-2 and are applicable in the range of loading that might be experienced by the foundation materials during earthquake loading. The moduli of elasticity and rigidity and the damping values presented in Table 2.5-2 were evaluated from various dynamic tests.

2.5.2.6 Design-Basis Earthquake

Seismic Category I structures at the power plant have been designed so that they can be safely shut down in the event ground accelerations at the site exceed those established for the OBE. Consequently, an evaluation has been made of the degree of ground motion which is remotely possible, considering both seismic history and geologic structure. The critical structures are designed for safe shutdown due to the appropriate ground accelerations at foundation level, presented in Table 2.5-3. In developing the DBE factors, it has been considered that there is a history of minor to moderate earthquake activity in the region, which has not been related to known tectonic features.

The 1934 Intensity VI Rock Island shock and the 1909 Intensity VII central Illinois shock are the largest earthquakes in the region which cannot be related to specific tectonic features. However, it is believed that these shocks (and several smaller recorded shocks) may be related to minor faults formed during development of the Paleozoic uplift-basin-arch regional configuration. Although these fault zones are believed to have been dormant since Paleozoic time, earthquake activity in the area may represent deep crustal readjustment to Pleistocene glacial advance and retreat. Glacial rebound at the site area is nonexistent as far as it is known. However, since the tectonics of the entire region are essentially similar, minor earthquakes, similar to the 1909 (July) and 1934 shocks, could occur in the vicinity of the site. On the basis of this very conservative hypothesis, the effect of a shock similar to the July 1909 central Illinois earthquake with its epicenter near the site has been considered. It is estimated that the maximum horizontal ground acceleration at the rock surface due to such a shock would be about twelve percent of gravity.

The effect at the site of a possible future earthquake similar to a large historical shock has also been investigated. The results of this study are presented below:

1. The May 26, 1909, Intensity VII northern Illinois earthquake. Should a shock similar to this earthquake occur, even the closest approach to the site of the Sandwich fault (about 125 miles), the effect at the site would be barely perceptible.

2. The 1867 Intensity VII northeastern Kansas earthquake. This shock was probably related to the Nemaha Uplift-Abilene Arch structural system. Making the conservative assumption that a similar shock could occur as close to the site as the closest approach of the Thurman-Redfield Structural Zone (about 100 miles), the attenuated ground acceleration at the site would be less than 5% of gravity.

3. The 1811-1812 New Madrid earthquakes. Should a shock as large as one of these earthquakes occur as close to the site as the closest approach of the Ste. Genevieve-Cottage Grove-Shawneetown-Rough Creek fault complex (about 300 miles), the attenuated ground acceleration at the site probably would be less than 5% of gravity.

It is concluded that these occurrences would result in ground motion at the site significantly less than that selected for the DBE.

Foundation level response spectra (horizontal) are presented in Figure 2.5-8 for the operating-basis and design-basis earthquakes. The response spectrum for the operating-basis earthquake (OBE) is equal to one-half of the corresponding DBE.

For structures supported on bedrock or on lean concrete, Housner’s average response spectra, normalized to 6% and 12% of gravity (with appropriate damping), were used as the criteria response spectra. For structures supported on about 10 ft of compacted fill and/or natural glacial soils or soil cement fill overlying bedrock, the smoothed response spectra for the 1952 Taft, California, earthquake, normalized to 6% and 12% of gravity (with appropriate damping), were used as the criteria response spectra. For structures supported on 30 to 50 ft of overburden soils and/or compacted fill soils, the smoothed response spectra for the 1952 Taft, California, earthquake, normalized to 9% and 18% of gravity (with appropriate damping), were used as the criteria response spectra.

The response spectra for structures supported on soil were selected because it is believed that the subsurface conditions at the site are comparable to those at the strong-motion recording stations for the 1952 Taft, California, earthquake. In addition, the epicentral distance of the shock and the expected maximum ground accelerations are near to those recorded at Taft.

The response spectra indicate the estimated responses of a structure subjected to earthquake ground motion. The spectra are presented over a range of frequencies corresponding to the natural frequencies of the various structural elements. The spectra

represent the maximum amplitude of motion in the various elements of the structure for typical degrees of damping.

Time-history analyses were performed for the seismically analyzed structures. One earthquake time history was developed for use as the input motion in performing the seismic analysis of structures as discussed in Section 3.7.1.2. Modal response spectrum analyses were not performed because the time history produced spectra that were conservative relative to the criteria response spectra.

On the basis of the seismic history of the area, it does not appear likely that the site will be subjected to significant earthquake ground motion during the life of the plant. However, Seismic Category I structures are conservatively designed to respond elastically, at foundation level, with no loss of function, to the appropriate foundation level accelerations presented in Table 2.5-3. See Table 2.5-4 for design information on these structures.

2.5.2.7 Operating-Basis Earthquake

The operating-basis earthquake is discussed in Section 2.5.2.6.

2.5.3 SURFACE FAULTING

Surface faulting is discussed within the context of Section 2.5.2.

2.5.4 GEOLOGIC FEATURES

2.5.4.1 Geologic Features

The site is located on the western flood plain of the Cedar River approximately 2.5 miles northeast of Palo, Iowa. The ground surface at the site slopes gently downward from northwest to southeast. The ground surface within the plant area is relatively level with elevations ranging from approximately 746 to 750 ft.

The average depth of frost penetration in the vicinity of the site is about 3.5 ft.

The principal soil and rock strata at the site are shown in Table 2.5-5. To assist in visualizing the subsurface conditions at the site, three subsurface sections are presented in Figures 2.5-11, 2.5-12, and 2.5-13.

The principal soil and rock strata in the immediate plant area are as follows.

The upper topsoil generally consists of loose silt and fine sand with a variable clay content.

The alluvial deposits consist of loose to medium dense predominantly granular soils that range in gradation from silty and clayey fine sand to coarse sand with some gravel and occasional cobbles and boulders. The silt and clay content of this stratum generally decreases with depth. A medium stiff alluvial deposit of cohesive soils containing organic material and fragments of decayed wood is present in certain borings. This layer of cohesive soils, consisting of clayey silts and silty clays with a variable sand content, ranges in thickness from 4 to 9 ft and was encountered at approximately elevation 722 to 737 ft and extended to approximately elevation 717 to 732 ft.

The glacial till soils are predominantly stiff to very stiff silty clay with some sand and gravel and occasional boulders.

The Wapsipinicon Formation at the site contains three members. The upper member is the Davenport, which consists of light grayish-brown aphanitic limestone, which includes several breccia zones and a 6-in.-thick layer of dolomitic limestone. The Davenport Member is thinly bedded and is usually argillaceous.

The middle member is the Spring Grove, which consists of light grayish-brown, finely crystalline to aphanitic dolomitic limestone. Numerous zones of solution-enlarged vugs up to 6 in. in diameter were encountered in this zone. A trace of grayish-brown residual clay was detected in some of the vugs.

The Kenwood is the third member of the Wapsipinicon Formation. The Kenwood consists of gray crystalline dolomitic limestone, which grades to argillaceous limestone and is interbedded with calcareous shales. The Wapsipinicon Formation is underlain by the Gower Formation, which extends to the depth penetrated by the borings. The Gower Formation is composed of gray aphanitic dolomite. Occasional highly argillaceous, vugged, and fossiliferous zones were encountered within the formation.

A cavity was encountered in Boring 21 that extended from approximately elevation 676 to 684 ft. The cavity, which is partially soil filled, is located in the Spring Grove Member at the interface between the Spring Grove and Kenwood Members of the Wapsipinicon Formation. The soils encountered in the cavity consisted of clayey silts and silty clays containing occasional pockets of sand and gravel and occasional seams of black decayed organic materials.

Additional borings revealed that the cavity may be a channel or a series of interconnected cavities. The vertical dimension of the cavities encountered ranged from approximately 12 to 3.5 ft.

No other major cavities were encountered in the 65 borings drilled in the plant area. Numerous additional open and soil filled cavities, up to a few inches in thickness, and small solution-enlarged vugs and joints were encountered in many of the test borings.

Although only one major cavity was encountered in the test-drilling program, geologic studies and the interpretation of test-boring results indicated that other such cavities could be present in the immediate plant area. A rock inspection and exploration-probing program, cavity stability analyses, and a rock-grouting program were performed to provide assurance that the rock would provide suitable foundation support. These programs are discussed in Section 2.5.4.3.

2.5.4.2 Properties of Subsurface Materials

The properties of subsurface materials are discussed within the context of Section 2.5.4.3. Section 2.5.7 provides additional information for the area in the vicinity of the low-level radwaste processing and storage facility.

2.5.4.3 Explorations

The original exploration program is discussed below. Explorations made for the low-level radwaste processing and storage facility are discussed in Section 2.5.7.

2.5.4.3.1 General

Field explorations and laboratory tests were performed to evaluate the geologic, seismologic, and foundation characteristics of the site. The field exploration program consisted of the following:

1. A geologic reconnaissance of the region and site.

2. A test-boring program.

3. Geophysical explorations that included geophysical refraction surveys, an uphole velocity survey, and micromotion measurements.

The field exploration program was conducted under the technical direction and supervision of Dames & Moore geologists, engineering seismologists, geophysicists, and soils engineers. All surveying necessary to determine the locations and surface elevations related to the field explorations was provided by Soil Testing Services of Iowa, Inc.

2.5.4.3.2 Geologic Reconnaissance

A geologic reconnaissance of the general area surrounding the site was performed to examine surface features to aid in the evaluation of the geologic characteristics of the area. The site was inspected with respect to topography, river features, surface soils, drainage, and other related surface features.

Geologic literature and aerial photographs of the site area were studied. Representatives of local, state, and Federal agencies; private organizations and universities were interviewed to obtain all available geologic data.

2.5.4.3.3 Test-Boring Program

The subsurface conditions at the site were investigated by drilling 15 test borings for the geologic study, 57 test borings for the foundation investigation in the plant area, and 9 test borings to define the lateral extent and shape of a solution cavity encountered in Boring 21. In addition, nine auger holes were augered to define the bedrock surface. The borings were drilled to depths ranging from 44 to 198.5 ft below the existing ground surface. The locations of the field explorations are shown in Figures 2.5-6, 2.5-14, 2.5-15, and 2.5-16.

Borings Pl through P10 (P8 omitted) were drilled at the site by Soil Testing Services of Iowa, and the results were provided to Dames & Moore for review and use during the geologic studies. The logs of these borings are presented in Figures 2.5-17 through 2.5-20. In addition, Borings Al through A9 were augered by Soil Testing Services of Iowa. The results of the auger borings are presented in Table 2.5-6.

The drilling operations for Borings Pll through P13, 1 through 54, and 21A through 21I were supervised by Dames & Moore engineers and geologists, who maintained a log of the borings, obtained relatively undisturbed samples of the soil using a Dames & Moore soil sampler and shelby tubes, performed standard penetration tests, and supervised the diamond core drilling operations performed to extract cores of the underlying rock. Soil samples were not obtained from Borings 21A through 21I. Graphical representations of the soils and rock encountered in these borings are shown in Figures 2.5-21 through 2.5-82. The method used in classifying the soils encountered in the borings is defined in Figure 2.5-83.

Relatively undisturbed samples of the soils penetrated were obtained in a Dames & Moore soil sampler as illustrated in Figure 2.5-84 and 2-in. shelby tubes. Standard penetration tests were also performed in the test borings. The method of obtaining

samples and the sample type is explained in the log of borings. Rock cores were obtained from the borings by using BX- and NX-size coring equipment.

Perforated pipe was installed in several borings at the completion of the drilling operations. The casing prevented the walls of the boring from caving and allowed periodic ground-water-level measurements to be taken. The results of the ground-water observations are recorded in the log of borings.

The ground-surface elevation is shown above the log of each boring and refers to U.S. Geological Survey datum.

2.5.4.3.4 Geophysical Explorations

The following geophysical explorations were conducted at the site:

1. Seismic refraction lines to define the bedrock topography.

2. A seismic refraction line for the determination of dynamic soil and rock properties.

3. An uphole velocity survey to provide additional dynamic and rock properties.

4. Micromotion observations to determine predominant periods of ground motion at the site.

The locations of the above explorations are shown in Figure 2.5-6. A description of each phase of the geophysical explorations is provided in the following sections.

2.5.4.3.4.1 <u>Seismic Refraction Surveys</u>

Five seismic refraction lines were run to define the bedrock topography at the site. The north-south and east-west seismic lines were performed by Soil Testing Services of Iowa. Seismic lines X-X, Y-Y, and Z-Z were also performed by Soil Testing Services of Iowa under the direct technical supervision of a Dames & Moore geophysicist. These seismic lines were used in conjunction with the geologic and foundation test borings to arrive at the bedrock contours presented in Figure 2.5-6.

In addition, a short seismic refraction line was run by a Dames & Moore geophysicist to determine dynamic properties for the soil and rock. Six groups of geophones, spaced at 100-ft intervals, were used to detect the various seismic waves generated by small explosive charges. Each geophone group consisted of two geophones, oriented vertically and horizontally in a radial direction. The charges were generally buried at least 3 ft below the ground surface and were placed at distances of 20, 400, and 800 ft from both ends of the seismic line. Permanent records of the seismic waves generated were obtained by using an Electro-Technical Labs M-4-E amplifier and a SDW-100 oscillograph.

The apparent compressional velocities measured during these studies are presented in Figure 2.5-85. Because of the relatively large number and the thinness of soil and rock layers at the site, measurements could not be obtained of either the shear wave velocity or a surface wave velocity as is generated by Raleigh waves.

2.5.4.3.4.2 Seismic Measurements and Resistivity Survey

Additional geophysical surveys were made by Western Geophysical Engineers, Inc., at foundation level under the reactor building to further evaluate foundation conditions.

These geophysical studies, which consisted of seismic measurements and resistivity surveys, were to serve as an aid in outlining solution cavities filled with detrital glacial till and clay in the bedrock limestone underlying the reactor building. The field measurements were made during the period of March 14 through March 18, 1970.

Data were recorded on six seismic lines in the reactor area, presented in Figure 2.5-86. Three of these lines had a common point at borehole B-288 on the southern side of the excavation. The three remaining lines had a common point at B-237 in the southwestern corner of the excavation site. The three lines from B-288 were laid toward B-67, B-55A, and B-101, with the four boreholes being used as shot points off the ends of the seismic lines. Similarly, the three lines shot from B-237 extended to shot points in B-101, B-67, and B-248.

The line between B-288 and B-67 was also occupied by eight short, overlapping refraction lines. Energy for running these lines was obtained by either hammer blows from the ends of the lines or light explosive charges in B-67.

The seismic geophone locations are shown in Figure 2.5-86, except for the short refraction lines with detector spacing of 3 and 6 ft between B-288 and B-67.

Once the velocity of the sound rock was established from the recorded data, the arrival times at the individual geophones were interpretable in terms of whether the seismic energy arrived at this velocity or at a slower velocity. A slower velocity indicated that a portion of the travel path was through clay zones between the shot point and the detector.

All of the long seismic lines recorded compressional (P) wave velocities of 15,000 fps, indicating in general that sound rock extended throughout the reactor area. The short surface refraction lines with penetration limited to the top few feet of rock had P-wave velocities between 9000 and 10,000 fps, and the transverse (S) wave velocities between 3300 and 3700 fps.

Individual detector positions having clay zone indications are shown in Figure 2.5-86, qualified, as shown in the legend, on whether based on one-way or two-way arrival data.

A Wenner electrode configuration with 10-ft electrode spacing was used to obtain resistivity data from the north-south pattern of lines labeled A through M in Figure 2.5-86.

Lines B to F were run on March 15; lines G, H, K, and M on March 16; and lines K and M were repeated on March 17, in addition to new lines A, J, and L. The level of surface water and the dryness of the surface rocks and muds changed markedly from day to day. The high water level on the 16th influenced the resistivity results on the eastern half of the excavated area and reduced the depth penetration seriously. The affected lines were rerun successfully the following day after additional pumping had lowered the water level.

The contoured resistivity values are shown in Figures 2.5-87 and 2.5-88. These resistivity values are plotted in ohm-feet. Units of ohm-feet were used for convenience, but may be converted to ohm-centimeters by multiplying by 30.5. In general, lower resistivity values correlate with known clay-filled solution cavities. However, the low resistivity values on the outer north-south lines (A and M) are a result, in part, to the lateral influence of the glacial till slopes to the side and above the rock floor of the excavation. The surface drainage pattern running north-south through the center of the excavation area appeared to be responsible for the broad area of lower resistivity values outlined essentially by the 2000 ohm-ft contour in this part of the excavation.

A resistivity high was located at B-234 (not shown) north of the line joining B-238 and B-248, and in line between B-288 and B-67. The sharp saddle in this type of feature illustrated the case of a sharply defined low-resistivity section in a high-resistivity area.

The resistivity results confirmed the evaluation of the seismic data that the overall rock quality is good with sharply localized zones of solution cavities. Combined seismic and resistivity data indicated areas of possible cavity zones. These were further investigated with drill holes.

2.5.4.3.4.3 Uphole Velocity Survey

An uphole velocity survey was performing in Boring 12 to provide a check on the compressional wave velocities measured during the seismic refraction surveys. The boring was cased to the rock surface, 47 ft below the ground surface, with steel casing. Small explosive charges were buried at a depth of approximately 3 ft and at a radial distance of 15 ft from the boring. The seismic response to the explosive charges was detected in the boring with a 12-trace geophone cable and was recorded with an Electro-Technical Labs M-4-E amplifier and a SDW-100 oscillograph.

The results of the uphole velocity survey are presented in Figure 2.5-89. It should be noted that the compressional velocity of the bedrock measured from this survey is less than the corresponding compressional velocity measured by seismic refraction. Since the seismic refraction surveys record the average dynamic properties of the bedrock over a distance and the uphole velocity survey provides dynamic properties at an isolated point,

the compressional velocities measured during the seismic refraction survey are more representative of the actual dynamic properties of the bedrock.

2.5.4.3.4.4 Micromotion Observations

Micromotion observations were made at four locations using the Dames & Moore micromotion equipment (Kosaka recording system). This equipment measures ground displacement and is capable of magnifying ground motions up to 150,000 times. The equipment is capable of recording ground displacements ranging in frequency from 1 to 300 Hz. The micromotion records indicate predominant periods of ground motion at the site.

Micromotion observation 1 was obtained on a rock outcrop near a quarry on the west section line of Section 18, Township 84 north, Range 8 west, approximately 3 miles southwest of the site. The intensity of ground motion was very low with no predominant period, which is indicative of hard rock. Observations were also made at three locations shown in Figure 2.5-6. The depth to bedrock at each location and the predominant ground periods observed are indicated in Table 2.5-7.

2.5.4.3.5 Laboratory Tests

Samples extracted from the test borings were subjected to a laboratory testing program to evaluate the physical properties of the soils encountered at the site. The laboratory test program included the following tests:

1. Static tests

   a. Direct shear.
   b. Unconfined compression.
   c. Triaxial compression.
   d. Consolidation.
   e. Rock compression.

2. Dynamic tests

   a. Triaxial compression.
   b. Shockscope.
   c. Resonant column.

3. Other physical tests

   a. Moisture and density tests.
   b. Particle-size analyses.
   c. Atterberg limits.

2.5.4.3.5.1 Static Tests

Strength Tests

Selected representative soil samples recovered from the borings were tested to evaluate their strength characteristics. These tests were performed to evaluate the bearing capacity of the soils underlying the site. The direct shear tests were performed in the manner described in Figure 2.5-90. Unconfined compression and triaxial compression tests were performed in the manner described in Figure 2.5-91. A load-deflection curve was plotted for each strength test, and the strength of the soil was determined from this curve. For the direct shear tests, the shearing strength is a yield strength, and for the unconfined compression tests, the shearing strength is either the peak strength or the strength at an axial deflection of one-tenth of the sample height, whichever occurs first. Determinations of the field moisture content and dry density of the soils were made in conjunction with each strength test. The results of the strength tests and the corresponding moisture content and dry density determinations for Borings 1 through 36 are presented in the log of borings. The method of presenting the test data is described by the key to test data shown in Figure 2.5-83. The results of strength tests and the corresponding moisture content and dry density determinations for Borings Pll through P13 are presented in Table 2.5-8.

Consolidation tests

Representative samples of the soils that were obtained from the borings were subjected to consolidation tests. These tests were performed to evaluate the compressibility characteristics of the soils. The method of performing consolidation tests is described in Figure 2.5-92. The results of these tests and the associated moisture content and dry density determinations are presented in Figures 2.5-93 through 2.5-95.

Rock Compression Tests

Rock compression tests were performed on selected samples of the bedrock that underlie the site of the proposed plant. The rock compression tests were performed to evaluate the strength and elasticity characteristics of the bedrock. The tests on the rock cores were performed by the Robert W. Hunt Company and Soil Testing Services of Iowa. The results of the rock compression tests are presented in Table 2.5-9.

2.5.4.3.5.2 Dynamic Tests

Triaxial Compression Tests

To evaluate the effect of vibratory motion on the strength of the in-situ soils and to determine the dynamic properties of those soils, selected samples were subjected to dynamic triaxial compression tests. The test procedure used is similar to that for static triaxial compression tests as defined in Figure 2.5-91. Each sample was subjected to a predetermined chamber pressure. The soil sample was then subjected to a series of

oscillating loads applied axially to the sample at a specified deviator stress. The additional deformation or strain of the soil sample under each oscillating load was recorded. The results of the dynamic triaxial tests are presented in Section 2.5.2.5.

Shockscope Tests

Several samples of the soil and rock underlying the site were tested in the shockscope. The shockscope is an instrument developed by Dames & Moore to measure the velocity of propagation of compressional waves in the material tested. The velocity of compressional wave propagation observed in the laboratory is used for correlation purposes with the field velocity measurements obtained in the geophysical refraction surveys.

In the shockscope tests performed, samples were subjected to a physical shock under a range of confining pressures, and the time necessary for the shock wave to travel the length of the sample was measured using an oscilloscope. The velocity of compressional wave propagation was then computed. Since this velocity is proportional to the dynamic modulus of elasticity of the sample, the data are also used in evaluating the dynamic elastic properties. The results of the tests are presented in Table 2.5-10.

Resonant Column Tests

Resonant column tests were performed on samples of soil and rock to determine the modulus of rigidity of the materials. The samples are set up in an apparatus that is similar to a triaxial compression cell. The sample is then subjected to steady-state, sinusoidal, torsional forces applied to the top of the sample. The frequency of the force application is varied until the resonant frequency is attained, that is, the frequency associated with the maximum steady-state amplitude. The modulus of rigidity can be computed from the resonant frequency since it is primarily a function of the stiffness (modulus of rigidity) of the samples.

The results of the resonant column tests are presented in Table 2.5-11.

2.5.4.3.5.3 Other Physical Tests

Moisture-Density Determinations

In addition to the moisture content and dry density determinations made in conjunction with the strength and consolidation tests, independent moisture and density tests were performed on other undisturbed soil samples for correlation purposes. The results of all moisture and density determinations are presented in the log of borings.

Particle-Size Analyses

A number of selected soil samples were analyzed to determine their grain-size distribution. The results of the analyses were used principally for classification purposes and to provide information for dewatering purposes. Grain-size curves illustrating the results of the particle-size analyses are presented in Figures 2.5-96 through 2.5-100.

Atterberg Limits

Representative samples were tested to evaluate the plasticity characteristics. The results of these tests were used primarily for classification purposes. The Atterberg limit determinations are presented in Table 2.5-12.

2.5.4.3.5.4 Sandfill

Laboratory tests were performed on a representative sample of sandfill to evaluate the dynamic soil properties of the sandfill that was placed for support of certain structures. The laboratory tests consisted of a sieve analysis, compaction test, maximum and minimum densities, and resonant column tests. The results of these tests are presented in Table 2.5-13.

2.5.4.4 Geophysical Surveys

Geophysical surveys have been discussed in the context of Section 2.5.4.3. Additional information for conditions in the vicinity of the low-level radwaste processing and storage facility is discussed in Section 2.5.7.

2.5.4.5 Excavations and Backfill

The information below is not inclusive of the low-level radwaste processing and storage facility, which is discussed in Section 2.5.7.

2.5.4.5.1 General

This section presents general criteria that were followed during excavating, dewatering, and filling. The methods of foundation support for the various structures are discussed and foundation design criteria are provided in Section 2.5.4.11.

2.5.4.5.2 Dewatering

Excavations for the turbine building and pump house extended below the ground-water level in the upper alluvial deposits. The excavation for the reactor building and intake structure extended to the rock surface, thus removing the impermeable cap existing over the artesian bedrock aquifer. Dewatering operations were therefore required in connection with all subsurface construction below approximately elevation 736 ft. The elevation of the ground water varied during the construction period, in

response to rain-fall, snow melt, and surface runoff conditions. Dewatering at the site was conducted by a qualified dewatering contractor in the following manner:

1. A well-point system was installed to lower the water table within the upper alluvial deposits. No dewatering of the lower glacial till soils was necessary because of the impermeable nature of these materials. However, minor water seepage through the glacial till was collected and pumped from the excavation.

2. The rock dewatering installation consisted of well points and deep wells into the rock and a system of sumps in the rock in which water was collected and pumped from the excavation.

The dewatering system maintained the water level in the upper soils and bedrock below all excavations or fill surfaces. Piezometers were installed to ensure that the water level in the over-burden soils and the artesian head in the bedrock were continuously maintained at a satisfactory level.

2.5.4.5.3 Excavation

This section presents the excavating operations that were required to attain planned grades and to prepare soils for the support of foundations or fill materials. The treatment of rock required for the support of foundations is discussed in Section 2.5.4.12.

The maximum depth of excavation was about 40 to 45 ft in the vicinity of the reactor building and intake structure. A portion of the soils excavated were granular materials that provided an excellent source of fill materials. These soils were stockpiled during excavation and used later as select fill and backfill material under foundations, floor slabs, and adjacent substructure walls.

In connection with attaining the required foundation levels, for all structures other than those supported on rock, all natural granular soils were either excavated from below foundation level in Seismic Category I building areas, or the granular soils were investigated to verify that liquefaction will not occur during the postulated DBE.

All loose granular soils and soft cohesive and organic soils were excavated in the turbine building area. The excavation of these soils and subsequent backfilling with controlled compacted fill where required was necessary to provide uniform support and to minimize differential settlement of the turbine building foundation. All loose or soft material and water were removed from the bottom of the excavations and the exposed soils were thoroughly proof rolled to detect any localized zones of soft or loose soils and to compact the soils disturbed by construction operations. Zones of soft or loose soils that could not be compacted were removed and replaced with controlled compacted fill.

The glacial till soils are susceptible to a loss of strength resulting from frost action, disturbance, and/or the presence of water. Insulating materials were installed in foundation excavations left open during the winter to prevent the softening and disturbance of the upgrade soils because of frost action.

On the attainment of final foundation grade in each area, a working mat of lean concrete was poured to prevent the loss of strength in the subgrade soils from water seepage and disturbance by construction operations.

Banks of the excavations were constructed on stable slopes that underwent only minor localized sloughing. Where localized sloughing occurred, the disturbed materials were removed before placing any backfill.

2.5.4.5.4 Sources of Fill Materials

Available sources of fill materials were essentially the following:

1. Materials removed from the plant excavations. These materials consisted of the following:

    a. Alluvial deposits (predominantly granular soils underlying the topsoil). Some of these granular soils were used in controlled compacted fill for the support of foundations and floor slabs and as back-fill adjacent to substructure walls. The silty soils encountered in the upper portion of this stratum were not used for structural backfill.

    b. Alluvial deposits (predominantly clayey silts and silty clays with some sand and some organic material). These soils were not considered suitable for use in controlled compacted fills for the support of structural loads but were used for site grading for construction activity.

    c. Glacial till soils. These soils would be difficult to place and compact in a controlled compacted fill because of their sensitivity to moisture. These materials were, therefore, not considered desirable fill material for the support of foundations, floor slabs or adjacent to substructure walls, and consequently, they were not used for these purposes. They were used for general site grading in the construction area.

2. Materials obtained from other onsite sources. Two potential sources of fill were investigated: (a) a borrow area in the river flood plain at the site outside the immediate plant area, and (b) a borrow area in the offshore islands of the Cedar River adjacent to the site. Both of these areas yielded acceptable granular materials that were satisfactory for use in the construction of fills for the support of foundations and floor slabs and as backfill adjacent to substructure walls. Item b, a borrow area in the offshore islands of the Cedar River adjacent to the site, was used to obtain the granular material for fill at the site.

3. Materials imported from offsite sources. Several possible offsite sources in the Cedar Rapids area were investigated. Available materials include clean granular materials such as processed sand and crushed limestone. Such materials were considered suitable for use in the construction of controlled compacted fills for the support of foundations and floor slabs and as backfill adjacent to substructure walls. Granular material from offsite sources was tested before placement to establish compaction criteria.

2.5.4.5.5 Filling and Backfilling

Fills up to approximately 7 to 13 ft in thickness were required in the attainment of the proposed final grade of elevation 757 ft. In addition, fills and backfills were required below and adjacent to structures.

All areas that are to receive structural loading, in which the final grade was raised by the placement of fill, were stripped of all topsoil and thoroughly proof rolled. The proof-rolling operations were required to detect any localized zones of soft or loose soils and to compact the exposed soils. Zones of soft or loose soils that could not be compacted were removed and replaced with controlled compacted fill.

All fill and backfill materials were placed at or near the optimum moisture content in lifts not exceeding 8 to 10 in. in loose thickness. Each lift was compacted in accordance with the criteria of Table 2.5-14.

It was considered that granular fills, for the support of Seismic Category I structures, placed in accordance with the above criteria, would have a significant margin of safety against possible liquefaction under the postulated design-basis earthquake. Following the selection of a source or sources of granular fill materials, laboratory tests were performed on representative compacted samples to verify that the margin of safety against liquefaction was satisfactory.

Filling operations were performed under the continuous technical supervision of a qualified soils engineer who performed in-place density tests in the compacted fill to verify that all materials were placed and compacted in accordance with the recommended criteria.

2.5.4.6 <u>Ground-Water Conditions</u>

During the original exploration at DAEC, ground water was encountered in the alluvial deposits overlying the glacial till at approximately elevation 738 to 742 ft. The ground-water level will vary during the year in response to rainfall and snow melt and will generally slope toward the Cedar River. The glacial till deposits are practically

impermeable but may contain trapped water in pockets of granular soils. Most of the borings that penetrated the bedrock surface demonstrated a sustained artesian head up to 4 ft above the ground surface, approximately to elevation 749 to 751 ft. Many of the original borings displayed artesian heads of up to 14 ft above grade when the bedrock was initially penetrated. The glacial till probably forms a complete cap over the artesian aquifer in the bedrock.

The level of the Cedar River at the site varied between elevation 732 to 734 ft during the time of the field investigation, June and July 1968. The maximum flood of record occurred in March 1961 when the Cedar River reached elevation 746.5 ft at the site. The Cedar River reached a peak stage of 751 ft with an approximate discharge flow of 110,000 cfs on June 13, 2008 at the Duane Arnold Energy Center.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

See Sections 3.7.2.4, 2.5.4.3, and 2.5.7.

2.5.4.8 Liquefaction Potential

No problem of liquefaction or partial loss of soil strength under seismic loading is expected for any of the major structures discussed in this report. All loose granular soils extending below foundation level of Seismic Category I structures were investigated to verify their stability under seismic loading. If an insufficient margin of safety existed, these soils were removed and replaced by compacted fill that was sufficiently dense to provide a significant margin of safety against a loss of strength or liquefaction during the postulated design-basis earthquake.

2.5.4.9 Earthquake Design Basis

The seismic design basis is discussed in Chapter 3.

2.5.4.10 Static Stability

The walls of the power plant structures that are constructed below grade are subjected to horizontal components of adjacent foundation loads, plus lateral pressures due to backfill soils and water. In the design of these relatively rigid walls to resist backfill and water pressures only, the compacted granular fill material above the water table was assumed to act as an equivalent fluid having a unit weight of 50 lb/ft^3. Below the water table, the compacted fill and the water are assumed to act as an equivalent fluid having a unit weight of 90 lb/ft^3. These values are based on the original DAEC soil boring report. All Seismic Category I structures were designed for hydrostatic and uplift pressures from the probable maximum flood, elevation 767.0 feet msl.

Substructure walls that are established below adjacent foundations are designed to resist certain horizontal components of foundation loads imposed by the adjacent foundations. Uplift loads are resisted by the deadweight of the structure and the frictional

resistance between the foundation walls and the granular backfill material. The frictional resistance was computed by assuming the granular soils have a unit weight of 120 lb/ft^3 above the assumed ground-water level, and 60 lb/ft^3 below the assumed ground-water level, an angle of internal friction of 35 degrees, a coefficient of lateral earth pressure equal to 0.3, and a coefficient of friction between soil and concrete equal to 0.4. A minimum factor of safety against uplift on the order of 1.2 was maintained.

Floor slabs established below the design floor level were designed for full hydrostatic pressure and were provided with adequate drainage facilities.

All backfill adjacent to walls is composed of clean granular materials.

All earth-supported floor slabs are underlain by a lean concrete mud mat or by a layer of thoroughly compacted clean granular fill at least 6 in. in compacted thickness.

2.5.4.11 Design Criteria

The reactor building is supported on a mat foundation established on rock, [redacted] The program of rock inspection, exploration, and grouting for the reactor foundation is discussed in Section 2.5.4.12.

The intake structure was founded at the same level as the reactor building. It is supported similarly and required rock exploration.

Exploratory drillings to detect cavities were also conducted for the stack, radwaste building, pump house, and turbine building as discussed in Section 2.5.4.12, although a wider spacing of exploration probings was used.

The low-level radwaste processing and storage facility, a non-seismic structure, is separated into processing and storage portions. The processing section has a reinforced concrete footing foundation supported on piles, while the storage section has a mat foundation also supported on piles.

Mat foundations for the radwaste building and control building were founded at higher elevations. These structures were founded predominantly on compacted granular materials placed as backfill adjacent to the reactor and turbine building substructures. Little or no natural granular materials remain in place under the foundations of Seismic Category I structures. Any such materials that remained in place were evaluated to verify stability under seismic loading or were removed and replaced with compacted fill soils.

The stack is supported on a mat foundation established at elevation 745 ft. Natural granular materials below the stack foundation were evaluated to verify stability under seismic loading.

The turbine building, a Nonseismic structure, is supported on a mat foundation [redacted] The turbine mat foundation was established entirely on stiff cohesive glacial till soils or on suitably controlled compacted fill materials overlying the till.

2.5.4.12 Techniques to Improve Subsurface Conditions

2.5.4.12.1 Rock Exploration

The reactor building and intake structure are supported directly on rock or on a lean concrete fill overlying the rock surface. Since solution cavities were encountered in the bedrock beneath the plant during the initial site explorations, a program for rock inspection was conducted under the supervision of experienced engineering geologists. The rock inspection was as follows:

1. The exposed bedrock was carefully inspected to detect surface joint patterns, fracturing, weathering, and cavities. All unsuitable rock or foreign materials were removed and replaced with lean concrete fill.

2. Exploration probe holes in the area under the reactor building and control building were drilled on a grid pattern 14 ft on centers in each direction. One probing in the reactor area was drilled to a depth of 175 ft. All other probe holes extended a minimum of 40 ft below the rock surface below foundation and grade, and probe into bedrock. In areas of heavy foundation loading below the reactor drywell and reactor peripheral foundations, a supplementary grid pattern of probings was intermeshed with the probes at 14-ft centers. These supplementary probes consisted of another grid pattern 14 ft on centers in each direction extending 40 ft into bedrock. Thus, for heavily loaded areas, the resulting overall drilling grid was 10 ft on centers. Additional probe holes were drilled if required to further define height and lateral extent of cavities of significant size that were detected by the initial drilling pattern.

The turbine building was founded at elevation 729 ft, and an exploration probe hole grid on 40-ft centers extending 40 ft into the rock surface was established.

The radwaste building and stack are founded at elevations 745 to 750 ft. For these structures, exploration probe holes for cavity detection were spaced at approximately 28-ft centers and 40-ft centers, respectively, and extended a minimum of 40 ft below the rock surface.

The pump house is founded at elevation 723 ft, and an exploration probe hole grid of approximately 16-ft centers extending 20 ft into the rock surface was established.

The intake structure is founded at elevation 706 ft, and an exploration probe hole grid of 15-ft centers both 20 and 40 ft into the rock was established.

The low-level radwaste processing and storage facility is founded at elevation 752 to 756 feet. Four exploratory borings were established to a depth of five feet into the rock.

Figures 2.5-6, 2.5-14, 2.5-15, and 2.5-16 show the location, spacing, and depth of exploratory probe holes under all Seismic Category I structures or Nonseismic structures housing Seismic Category I equipment. Figure 2.5-101 depicts a typical probing log.

2.5.4.12.2 Remedial Treatment of Rock

All exploration probe holes were pressure grouted. Before grouting the probe holes, each probe hole was cleaned with rotary wash drilling equipment to remove any debris that had fallen in the hole subsequent to drilling and to ensure that grout would penetrate to the full depth of the probe hole.

The rock-grouting program consisted of pressure grouting probe holes with a water-cement mix having a 1:1 ratio or a water-cement-sand mix having a ratio ranging from 1:1:1/2 to 1:1:2. In most of the probe holes, a packer was set at a depth of approximately 5 ft below the top of the rock, and the probe hole below the packer was then pressure grouted by using, a pressure of 5 psi. In some probe holes, a packer was set lower than 5 ft and correspondingly higher grouting pressures were used. Above a depth of 5 ft, the probe holes were filled by simple backfilling procedures. In probe holes where an adequate seal of the packer could not be obtained, the hole was filled by backfilling, and a new probe hole was drilled and pressure grouted to provide assurance that grout had filled all significant voids in the bedrock.

Detailed daily records were kept of the probe holes grouted, the depth grouted, the grout mix used, the grouting pressure used, and the sacks of cement and sand placed in each hole.

Rock surface treatment consisted of cleaning and filling with concrete all cavities and solution channels encountered at the bedrock surface. Surface fissures were generally filled with a stiff clay, and this clay was removed to a depth of at least twice the width of the fissure.

Several fissures or clay-filled openings increased in size with depth below the rock surface, and these areas were inspected by removing essentially all the clay fillings. At some locations, this required removing rock cover from over an opening in order to gain access for cleaning, inspecting, and filling.

The largest soil-filled cavity encountered was in the vicinity of probe holes 224 and 225. From a depth of 4 to 8 ft below the rock surface, this cavity was approximately 20 ft long and 12 ft wide. From a depth of 8 to 12 ft below the rock surface, the cavity was approximately 8 ft long and 6 ft wide. The cavity did not extend beyond a depth of approximately 12 ft below the rock surface.

The rock at the site is considered suitable for structural support on the following bases:

1. All cavities and solution channels encountered at the bedrock surface were cleaned to a depth at least twice the lateral extent of the fissure and filled with concrete.

2. No large cavities that could detrimentally affect structural performance were encountered by the probing program.

3. Pressure grouting of all probe holes was performed, thus minimizing the effects of minor cavities and vugs not otherwise treated.

4. Hypothetical cavities that might not have been detected by the exploration-probing program can be spanned by the structures and will not adversely affect foundation performance.

2.5.4.12.3 Mat Foundations

The ultimate bearing capacities of materials supporting mat foundations of the major units of the proposed construction have been evaluated on a conservative basis. The ultimate bearing capacities presented in Table 2.5-15 can be developed for mat foundations established at the appropriate tabulated elevations.

The ultimate bearing pressure values in Table 2.5-15 are gross values and consider ground water and finished plant grade to be at elevation 757 ft.

The ultimate bearing capacities of mat foundations established near the upper surface of the cohesive glacial soils were computed by using Skempton's[1] method. The ultimate bearing capacities of mat or spread foundations established in natural or compacted granular soils were computed by using Terzaghi's[2] method and were modified to prevent overstressing of the underlying glacial till soils. The significant parameters in both of these methods are the strength characteristics of the supporting soils, the width of the foundation, and the depth to foundation grade.

The bearing capacities shown in Table 2.5-15 are ultimate values, and suitable factors of safety have been applied. It is considered that a minimum factor of safety of 3 is appropriate for dead loads and frequently applied live loads. It is considered that a minimum factor of safety of 2 is satisfactory for dead, live, and seismic loads (design-basis earthquake). Table 2.5-16 presents a summary of the factors of safety for the various units that will be supported on mat foundations.

The results of settlement analyses for mat foundations imposing the bearing pressures given in Table 2.5-16 are presented in Table 2.5-17.

2.5.4.12.4 Spread Foundations

Conventional spread foundations are used for the support of the administration building and other appurtenant structures. These foundations are established in controlled compacted granular fill that has been placed to attain finished grade. The spread foundations are established at 3.5 ft below the lowest adjacent grade, and all controlled compacted fill was placed and compacted in accordance with the previously indicated criteria. The spread foundations were designed using the bearing pressures presented in Table 2.5-18. A higher bearing value may be used for foundations having an embedment larger than 3.5 ft. The bearing pressures were computed on the assumptions that the water table level is at elevation 757 ft and that foundations are at a relatively shallow depth. These bearing pressures refer to the total of all dead and live loads and are net values. Since these are net pressures, the weight of the backfill over the foundations and the weight of the concrete in the foundations was ignored in proportioning the foundations. The bearing pressures contain a factor of safety on the order of 3 and pertain to all design loads, excluding seismic loads. For seismic loads, the net bearing pressures have been increased by one-half.

2.5.4.12.5 Settlement

Settlement analyses were performed from results of consolidation tests that indicate that the glacial soils at the site have been preconsolidated under overburden and glacial pressures on the order of 10,000 lb/ft^2. The results of the consolidation tests are presented in Section 2.5.4.3. The settlement analysis has been based on previously stated assumptions regarding foundation elevations and foundation loading conditions. The results of the analysis will therefore be reviewed when structural loads and foundation elevations are finalized.

The results of the settlement analyses for mat foundations are presented in Table 2.5-17.

It is estimated that shallow spread foundations supporting a total design load of up to 30,000 lb and proportioned using the bearing pressures presented in the previous section will undergo settlement on the order of 0.5 in. or less.

Differential settlements between adjacent structures may be estimated by computing the differences between the settlements given in Table 2.5-17.

Earthquake loading of short duration should not cause additional settlement of appreciable magnitude. The effects of earthquakes were evaluated by dynamic laboratory tests. Using an appropriate dynamic modulus of elasticity of 1,000,000 lb/ft^2 for soil, it is estimated that additional settlements of earth-supported mat foundations under earthquake loading will be less than 1/8 in.

The time rate of settlement for earth-supported mat foundations underlain by natural glacial till soils has been estimated from results of engineering analyses of the consolidation test data. The approximate portion of the total settlement of a mat foundation that will have occurred at various times after the full bearing pressure has

been applied to the supporting soils has been estimated and is summarized in Table 2.5-19.

The settlement of conventional spread foundation, established on an appreciable thickness of controlled compacted granular fill or within the natural granular soils overlying the glacial till, will occur essentially as the load is applied to the foundation.

2.5.4.13 Subsurface Instrumentation

See the response to Safety Guide 12 in Section 1.8.12.

2.5.5 STABILITY OF SLOPES

This section presents a summary of foundation design criteria prepared on the basis of a comprehensive foundation investigation performed at the site. The plant is located near the Village of Palo in Linn County, Iowa. The site is located adjacent to and west of the Cedar River, approximately 8 miles northwest of Cedar Rapids, Iowa.

The subsurface conditions at the site consist of an upper stratum of loose to medium dense granular soils underlain by stiff glacial till soils. The glacial till is underlain by limestone bedrock of the Wapsipinicon formation. The bedrock, which is at a depth of 40 to 50 ft in the immediate plant area, contains some solution cavities that are discussed in detail in Sections 2.5.4.1 and 2.5.4.8. The solutioning process apparently developed before glaciation and is considered inactive at this time.

Competent foundation support has been ensured by a program of explorations to detect major cavities that may underlie foundations by a comprehensive rock-grouting program and by a reactor mat foundation designed to span any undetected cavities. The exploration and rock-grouting programs were conducted as outlined in Section 2.5.4.12 under the supervision of experienced engineering geologists.

The reactor building foundation, including the high-pressure coolant injection appendage, is supported directly on the limestone bedrock. Other Seismic Category I structures are founded as follows.

The intake structure is supported on bedrock at approximately the same level as and in a manner similar to the reactor building. The offgas stack and pump house are supported primarily on compacted granular backfill materials placed in contact with the natural glacial till soils. [redacted]

The turbine building is supported on a prepared subgrade of natural glacial till and controlled compacted fill soils. The low-level radwaste processing and storage facility is supported on piles and the radwaste building is supported on compacted granular fill over glacial till material. Other Nonseismic structures such as the office building and cooling tower are supported as required.

Criteria pertaining to foundation design and installation are presented in subsequent paragraphs.

The following comprehensive foundation evaluation program was completed:

1. Drilling of test borings in the immediate plant area.

2. Performance of laboratory tests required to evaluate the pertinent physical properties of the soil and rock underlying the site.

3. Evaluation of the types of foundations that will be suitable for the support of various plant structures.

4. Formulation of foundation to design data including the following:

    a. Allowable foundation-bearing capacities.

    b. Magnitude and time rates of settlement of foundations subjected to static and dynamic loading, including the consideration of differential settlement between various buildings.

    c. Recommended hydrostatic uplift pressures and lateral soil and water pressures.

5. Formulation of recommended criteria for site preparation and earthwork including the following:

    a. Excavating, dewatering, and bracing and sloping excavations.

    b. Explorations for the detection of cavities, evaluation of cavity stability, and remedial measures for the correction of cavities.

    c. Selection, placement, and compaction of fill materials for the support of foundations.

6. Discussion of foundation design and installation considerations associated with the possible future construction of Unit 2.

The DAEC consists of one unit. Its location was chosen considering the possibility of a future second unit. The locations and arrangements of the major structures are shown in Figure 2.5-14. The final plant grade has been established at approximately elevation 757 ft, which is about 7 to 13 ft above the existing grade. The foundation elevations and foundation loading conditions that have been used in developing the conclusions of this report are given in Table 2.5-4.

2.5.6 EMBANKMENTS AND DAMS

See Section 9.2.2.

2.5.7 LLRPSF Foundation Investigation

A foundation investigation was performed in the vicinity of the LLRPSF to investigate subsurface conditions at the proposed site by exploratory borings and to evaluate the engineering properties of the subsurface materials with appropriate field and laboratory tests.

Four exploratory borings were drilled using 3/4 inch I.D. hollow-stem augers and rotary wash down equipment. The four-borings extended to auger refusal and were then cored for 5 feet into the underlying rock. During drilling, samples of the subsurface soils were obtained using a split-spoon sampler and shelby tubes. Unconfined compressive strength tests were performed on the cohesive and semi-cohesive soil samples. These borings were supplemented by four additional test holes. The locations of the field explorations are shown in Figure 2.5-6. Boring Logs are provided as Figures 2.5-106 through 2.5-109.

Samples obtained from the borings were classified in accordance with the Unified Soil Classification Systems. Laboratory testing on representative samples from the borings included natural moisture contents, unit densities and unconfined compressive strengths. The results of the testing are shown in Figures 2.5-102 through 2.5-105. Groundwater levels are indicated on the boring logs (Figures 2.5-106 through 2.5-109). Borings RW-3 and RW-4 in the storage portion of the building encountered groundwater at depths of 12 to 13 feet below the ground surface. This indicates that the groundwater table is at elevation 739 to 744. For borings B-1 and B-2, the water table elevation ranged from approximately 742.7 to 743.7. Borings RW-1 and RW-2 in the processing portion of the building encountered groundwater about 13.5 feet below the ground surface for an elevation of approximately 741.7 to 742.7. Borings B-3 and B-4 indicated groundwater levels at an approximate elevation of 743.3.

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AGENCIES INTERVIEWED FOR SECTION 2.5

Iowa Geological Survey, Iowa City, Iowa

Northern Illinois University

St. Louis University, St. Louis, Missouri

U.S. Geological Survey, Iowa City, Iowa

U.S. Geological Survey, Washington, D.C.

Table 2.5-1

MODIFIED MERCALLI INTENSITY (DAMAGE) SCALE OF 1931
(Abridged)

(The intensity scale is a means of indicating the relative size of an earthquake in terms of its perceptible effect. The intensities indicated in this report are maximum intensities and indicate the damage caused by an earthquake at its epicenter.)

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 motor cars 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 motor cars rocked noticeably. (IV to V Rossi-Forel Scale)

V. Felt by nearly everyone; many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbances of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop. (V to VI Rossi-Forel Scale)

VI. Felt by all; many frightened and 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 gooddesign 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 motor cars. (VII Rossi-Forel Scale)

Table 2.5-1

MODIFIED MERCALLI INTENSITY (DAMAGE) SCALE OF 1931
(Abridged)

VIII. Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motor cars disturbed. (VIII + to IX Rossi-Forel Scale)

IX. Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broke. (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-la

EARTHQUAKE EPICENTERS WITHIN 200 MILES OF THE SITE

Year	Date	Time	Intensity	Location	N. Lat.	W. Long	Approximate Perceptible Area mi^2	Approximate Distance from Site (miles)
1804	Aug. 24	14:10	VI	Fort Dearborn Chicago, Illinois	42	88	30,000	200
1905	Apr. 13	10:30	V	Keokuk, Iowa	40.4	91.4	5,000	120
1909	May 26	08:42	VII	Northern Illinois	42	89	500,000	145
1909	July 18	22:34	VII	Central Illinois	40.2	90.0	40,000	160
1912	Jan. 2	10:21	VI	Northern Illinois	41.5	88.5	40,000	170
1928	Jan. 23	03:19	III-IV	Mt. Carroll, Illinois	42.0	90.0	400	90
1930	May 28	11:31	I-III	Hannibal, Missouri	39.7	91.3	Local	175
1933	Dec. 6	23:55	III-IV	Stoughton, Wisconsin	42.9	89.2	--	145
1934	Nov. 12	08:45	VI	Rock Island, Illinois	41.5	90.5	5,000	75
1935	Jan. 5	00:40	I-III	Davenport, Iowa	41.5	90.5	Local	75
1935	Feb. 26	08:15	I-III	Burlington, Iowa	40.8	91.1	Local	95
1939	Nov. 24	13:45	I-III	Davenport, Iowa	41.5	90.5	Local	75
1942	Mar. 1	09:43	IV	Kewanee, Illinois	41.2	89.7	--	120
1947	May 6	15:25	V	Milwaukee, Wisconsin	42-3/4	88	3,000	205
1956	Mar. 13	09:05	IV	Fulton Co., Illinois	40-1/2	90-1/4	--	130

Table 2.5-2

MODULI AND DAMPING VALUES

Material	Modulus of Elasticity ($lb/ft^2 x 10^6$)	Modulus of Rigidity ($lb/ft^2 x 10^6$)	Damping Factor[a] Operating Basis (%)	Damping Factor[a] Design Basis (%)
Alluvial sand	1.5	0.5	5-10	10-20
Glacial till	2	0.7	5-10	10-20
Wapsipinicon Formation (limestone/ dolomite)	500	200	1	1

[a] Expressed as a percentage of critical damping.

Table 2.5-3

FOUNDATION LEVEL ACCELERATIONS (ZPA)
for the design of
Seismic Category I Structures

OPERATING-BASIS EARTHQUAKE (OBE)

Supporting Material	Percentage of Gravity Horizontal (Acceleration)	Percentage of Gravity Vertical (Acceleration)
Rock, or about 10 ft of compacted fill and/or natural glacial soils, soil cement fill or lean concrete fill	6	4.8
About 30 to 50 ft of compacted fill and/or natural glacial soils	9	6

DESIGN-BASIS EARTHQUAKE (DBE)

Supporting Material	Percentage of Gravity Horizontal (Acceleration)	Percentage of Gravity Vertical (Acceleration)
Rock, or about 10 ft of compacted fill and/or natural glacial soils, soil cement fill or lean concrete fill	12	9.6
About 30 to 50 ft of compacted fill and/or natural glacial soils	18	12

Table 2.5-4
SUMMARY OF DESIGN DATA

Structures	Approximate Plan Dimensions (ft)	Foundation Type	Bottom Foundation Elevation (ft)	Actual Foundation Load (psf) Dead and live Loads	Actual Foundation Load (psf) Dead, Live and Seismic (DBE) Loads
		Seismic Category I			
Reactor building	143 x 146	Mat on bedrock	[redacted]		
Dry well	86 diam.			12,200	23,000
Peripheral walls	16 width			8,600	35,000
Remainder of mat				1,400	3,400
Control building	70 x 84	Mat on fill	[redacted]	4,500	7,000
Pump house	84 x 56	Mat on fill	[redacted]	2,200	4,000
Intake structure	40 x 75	Mat	[redacted]	2,600	5,400
Stack	60 diam.	Mat on fill	[redacted]	Later	Later
		Nonseismic			
Turbine building	142 x 262	Mat on fill	[redacted]	3,100	5,400
Administration building	103 x 95	Spread footing on fill	[redacted]	4,500	5,200 (UBC)
Radwaste building	69 x 98	Mat on fill	[redacted]	3,300	4,100 (UBC)
Low-level Radwaste Processing and Storage Facility					
Storage Portion	155 x 66 72 x 28	Mat on piles	[redacted]	Reference 3	
Processing Portion	161 x 146 46 x 74	Footing on piles	[redacted]	Reference 4	

* Floor Elev.

Table 2.5-5

PRINCIPAL SOIL AND ROCK STRATA

Stratum	Top Elevation	Bottom Elevation	Thickness (ft) Minimum	Thickness (ft) Maximum
Topsoil	746 to 750	744 to 747	0.8	2.5
Alluvial deposits	744 to 747	717 to 736	8	30
Glacial till	717 to 736	705 to 707	12	33
Wapsipinicon Formation				
Davenport Member	705 to 707	701 to 692	8	15
Spring Grove	692 to 696	671 to 678	15	21
Kenwood Member	672 to 680	645 to 653	25	40
Gower Formation	640 to 645			

Table 2.5-6

SUMMARY OF AUGER BORINGS

Boring Number	Ground-Surface Elevation (ft)	Depth to Rock[a] (ft)
A1	[redacted]	49
A2	[redacted]	52
A3	[redacted]	57
A4	[redacted]	52.5
A5	[redacted]	44
A6	[redacted]	68
A7	[redacted]	([b])
A8		44
A9	[redacted]	44

[redacted]

Table 2.5-7

MICROMOTION OBSERVATIONS

Micromotion Observation Number	Depth to Bedrock (ft)	Predominant Period of Ground Motion (sec)
2	45	0.13 to 0.14
3	100	0.35
4	25	Quiet, no signal

Table 2.5-8

STATIC STRENGTH TESTS FOR
BORINGS P11, P12, AND P13

Boring Number	Depth (ft)	Moisture Content (%)	Dry Density (lb/ft^3)	Shearing Strength (lb/ft^2)
P11	12	15.7	117	3200
	14	12.6	123	1900
	18	18.8	108	
	28	13.8	120	
	46	16.8	116	
	56	16.5	114	
	61	17.3	122	
	66	19.6	111	2200
	71	12.4	123	
P12[a]	14	15.3	114	2300
	19	15.5	117	3000
	24	15.0	112	1850
	29	15.8	113	1250
	31	15.5	112	2200
	34	17.3	114	1400
	38	15.1	115	1700
	44	18.3	112	1500
	49	18.0	109	1500
	54	32.4	97	1700
P13	18	17.3	115	
	28	14.4	120	2600
	33	15.7	118	2100
	39	19.2	112	
	44	16.1	115	

[a] All samples from Boring 12 were tested by Soil Testing Services of Iowa, Inc.

Table 2.5-9 Sheet 1 of 2

ROCK COMPRESSION TEST RESULTS

Boring Number	Depth (ft)	Density ($1b/ft^3$)	Ultimate Compressive Strength ($1b/ft^2$)	Modulus of Elasticity ($1b/ft^2$)
3	64	--	3,190	--
4	47	465	10,890	--
6	62	154	1,920	--
8	57	--	5,730	--
11	46	--	16,820	--
17	52	--	16,050	--
18	68	154	4,330	--
20	43	170	16,880	--
20	45	--	9,680	10.5×10^6
21	46	--	17,580	--
21	65	--	2,360	4.5×10^6
21A	47	--	7,120	--
21B	60	--	4,410	--
21C	66	--	3,110	--
21D	54.5	--	8,470	--
21F	49	--	10,430	--
21G	58	--	7,060	--
21H	45	--	9,600	--
21H	57	--	9,450	12.7×10^6
21I	55.5	--	8,850	8.3×10^6
22	58	--	7,260	--
24	64	--	3,190	--
27	54	--	8,600	--
28	71	--	11,150	--
30	56	158	10,510	--
30	60	--	12,230	--
40	56	157	7,420	1.3×10^6
40	64	142	2,850	0.7×10^6
40	65	130	3,600	0.8×10^6
40	68.5	130	2,800	0.6×10^6
41	46	162	10,810	0.9×10^6
41	67	127	2,800	0.5×10^6
41	85.5	170	11,850	1.7×10^6
42	58	146	2,080	0.3×10^6
42	71	153	3,500	0.4×10^6
43	65	120	500	0.2×10^6
43	70	144	4,150	0.5×10^6
43	79	171	9,900	1.1×10^6
44	66	129	1,760	0.5×10^6
45	59	158	6,910	0.9×10^6
45	68	159	11,840	0.9×10^6
46	64	141	3,320	0.5×10^6

Table 2.5-9 Sheet 2 of 2

ROCK COMPRESSION TEST RESULTS

Boring Number	Depth (ft)	Density (lb/ft^3)	Ultimate Compressive Strength ($1b/ft^2$)	Modulus of Elasticity ($1b/ft^2$)
48	46	164	9,350	1.3×10^6
48	62	146	1,860	0.2×10^6
51	105.5	144	3,630	0.3×10^6
41	94	155	1,580	0.04×10^6
51	100	130	18,400	1.0×10^6
51	100	137	15,400	2.5×10^6

Table 2.5-10

SHOCKSCOPE TEST RESULTS

Boring Number	Depth Type	Soil Type	Confining Pressure (lb/ft^2)		Velocity of Compressional Wave Propagation (ft/sec)
7	23	CL	0,	2,000	5,500
			4,000,	6,000	5,500
7	38	CL	0,	2,000	5,900
			4,000,	6,000	5,900
9	5	SP	0		1,000
			2,000		1,200
			4,000		1,400
			6,000		2,100
10	43	CL	0,	2,000	5,900
			4,000	6,000	5,900
17	48	Rock	0,	6,000	14,000
17	66	Rock	0,	6,000	15,000
17	70	Rock	0,	6,000	16,000
17	91	Rock	0,	6,000	11,000
17	133	Rock	0,	6,000	16,000
20	35	CL	0,	2,000	5,200
			4,000		5,200
			6,000		5,900
23	5	SP	0		1,400
			2,000		1,700
			4,000		2,100
			6,000		2,800
23	40	CL	0,	2,000	4,600
			4,000	6,000	5,200
28	5	SP	0		700
			2,000		800
			4,000		1,000
			6,000		1,400
30	35	CL	0,	2,000	5,500
			4,000	6,000	5,500

Table 2.5-11

RESONANT COLUMN TEST RESULTS

Boring Number	Depth (ft)	Soil Type	Modulus of Rigidity (lb/ft^2)
7	33.5	CL	0.983×10^6
12	63	Rock	143×10^6
17	49	Rock	181×10^6
17	85	Rock	171×10^6
17	103	Rock	170×10^6
20	40.5	CL	0.762×10^6
23	25.5	CL	0.823×10^6
30	30.5	CL	0.851×10^6

Table 2.5-12

ATTERBERG LIMIT DETERMINATIONS

Boring Number	Depth (ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index
12	41.5	26	16	10
25	35.5	26	15	11
28	10.5	33	15	18
33	18.5	29	26	3

Table 2.5-13

SANDFILL LABORATORY TESTS

Sieve Analysis

U.S. Standard Sieve Number	Percent Passing
10	94.5
20	80.0
40	40.7
60	15.1
100	4.4
200	0.9

Compaction Test, ASTM-1557

Maximum dry density	114 lb/ft^3
Optimum moisture content	12.0%

Minimum and Maximum Densities, ASTM-D2049

Minimum dry density	101.9 lb/ft^3
Maximum dry density (dry method)	120.3 lb/ft^3
Maximum dry density (wet method)	124.2 lb/ft^3

Resonant Column Test Results

Sample dry density	107.8 lb/ft^3
Sample moisture content	3.4%

Confining Pressure (lb/ft^2)	Modulus of Rigidity (lb/ft^2)	Shear Strain
993.6	1.303×10^6	0.143×10^{-3}
1497.6	1.707×10^6	0.123×10^{-3}
2995.2	2.640×10^6	0.082×10^{-3}
5011.2	3.379×10^6	0.067×10^{-3}

Table 2.5-14

SOIL COMPACTION REQUIREMENTS

Purpose of Fill	Recommended Minimum Compaction Criteria (Percentage of Maximum Density) [a]: Cohesive Soil	Granular Soils
Support of Seismic Category I structures	95	100
Support of Nonseismic structures	90	95
Adjacent to structures	90	95
Areal Fill (not supporting or adjacent to structures)	85	90

2013-004

[a] Maximum density and optimum moisture content were determined by the American Association of State Highway Officials Test Designation: T180-57.

Table 2.5-15

ULTIMATE BEARING CAPACITIES FOR SEISMIC CATEGORY I
AND NONSEISMIC STRUCTURES

Structures	Supporting Materials	Foundation Elevation (ft)	Ultimate Bearing Capacity (lb/ft^2)
	Seismic Category I		
Reactor building	Limestone rock or lean concrete immediately overlying rock	[redacted]	100,000
Pump house	Stiff to very stiff natural glacial till soils[a]	[redacted]	13,500
Intake structure	Limestone rock or lean concrete immediately overlying rock	[redacted]	100,000
Stack	Controlled compacted granular fill overlying natural glacial till soils	[redacted]	15,000
Control building	Controlled compacted granular fill overlying bedrock	[redacted]	15,000
Turbine building	Stiff to very stiff natural glacial till soils [a]	[redacted]	13,500
Radwaste building	Controlled compacted granular fill overlying natural glacial till soils	[redacted]	15,000
Administration building	Controlled compacted granular fill overlying very stiff natural glacial till	[redacted]	15,000
Low-level Radwaste Processing & Storage Facility	Steel 12 x 74 H-piles driven to limestone rock	[redacted]	190,000 lb/pile

2013-004 [a] Some over-excavation and backfilling with controlled compacted fill was necessary in certain areas to ensure that all unsuitable soils were removed.

Table 2.5-16

BEARING PRESSURE VERSUS FACTORS OF SAFETY

Structures	Dead and Live Loads Bearing Pressure (lb/ft^2)	Dead and Live Loads Factor of Safety	Dead, Live, and Seismic (DBE) Loads Bearing Pressure (lb/ft^2)	Dead, Live, and Seismic (DBE) Loads Factor of Safety
Seismic Category I				
Reactor building				
Dry well	12,200	8.2	23,000	4.3
Peripheral walls	8,600	11.6	35,000	2.9
Remainder of mat	1,400	71.0	3,400	29.0
Control building	4,500	3.3	7,000	2.2
Intake structure	2,600	38.0	5,400	19.0
Stack	Later	Later	Later	Later
Pump house	2,200	6.1	4,000	3.4
Nonseismic				
Turbine building	3,100	4.3	5,400	2.5
Radwaste building	3,300	4.5	4,100	3.7 (UBC)
Administration building	4,500	3.3	5,200	2.9 (UBC)
* Low-Level Radwaste Processing and Storage Facility				
Storage Portion	163,800 lbs/pile	1.16	179,300 lb/pile	1.06 (UBC)
Processing Portion	157,700 lbs/pile	1.24	153,700 lb/pile	1.24 (UBC)

2013-004 * Based on maximum pile loadings

Table 2.5-17

ESTIMATED SETTLEMENT

Structure	Estimated Settlement (in.)
Reactor building and intake structure	0 to 1/4
Radwaste building	1/2 to 1
Control building	1/2 to 1
Stack	1/2 to 3/4
Turbine building	1/2 to 1
Low-level Radwaste Processing and Storage Facility	0 to 1/8

Table 2.5-18

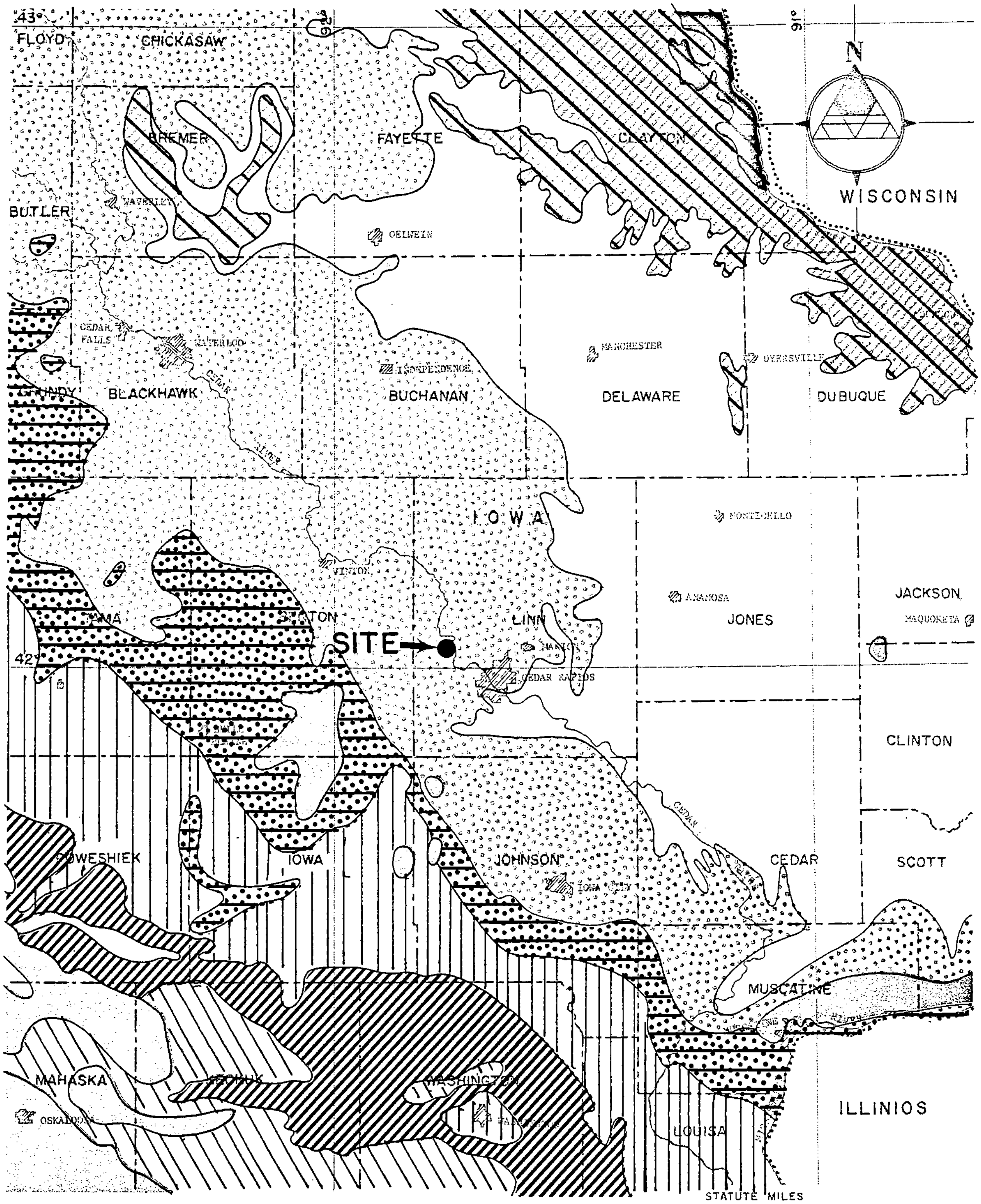
ALLOWABLE NET BEARING PRESSURES FOR SPREAD FOUNDATIONS

Supporting Soils	Allowable Net Bearing Pressure (lb/ft^2)
Controlled compacted granular fill	
Foundation width = 2 ft	3000
Foundation width = 4 ft	3500
Foundation width = 8 ft	4500
Foundation width = 12 ft	5000

Table 2.5-19

TIME-SETTLEMENT RELATIONSHIP

Total Settlement (%)	Time (Months)
20	1
50	7
90	30



10 5 0 10 20

LEGEND:

COLOR	PERIOD	SERIES
	PENNSYLVANIAN	DES MOINES
	MISSISSIPPIAN	MERAMAC
	MISSISSIPPIAN	OSAGE
	MISSISSIPPIAN	KINDERHOOK
	DEVONIAN	UPPER

COLOR	PERIOD	SERIES
	DEVONIAN	MIDDLE
	SILURIAN	UNDIFFERENTIATED
	ORDOVICIAN	MAQUOKETA, GALENA, DECORAH, PLATTEVILLE
	ORDOVICIAN CAMBRIAN	UNDIFFERENTIATED

REFERENCE:

THIS MAP WAS PREPARED FROM A PORTION OF USGS BASE MAP OF THE STATE OF IOWA, 1950.

THIS MAP WAS PREPARED FROM AN ENLARGED PORTION OF "THE PRELIMINARY GEOLOGIC MAP OF IOWA" BY IOWA GEOLOGICAL SURVEY, 1962.

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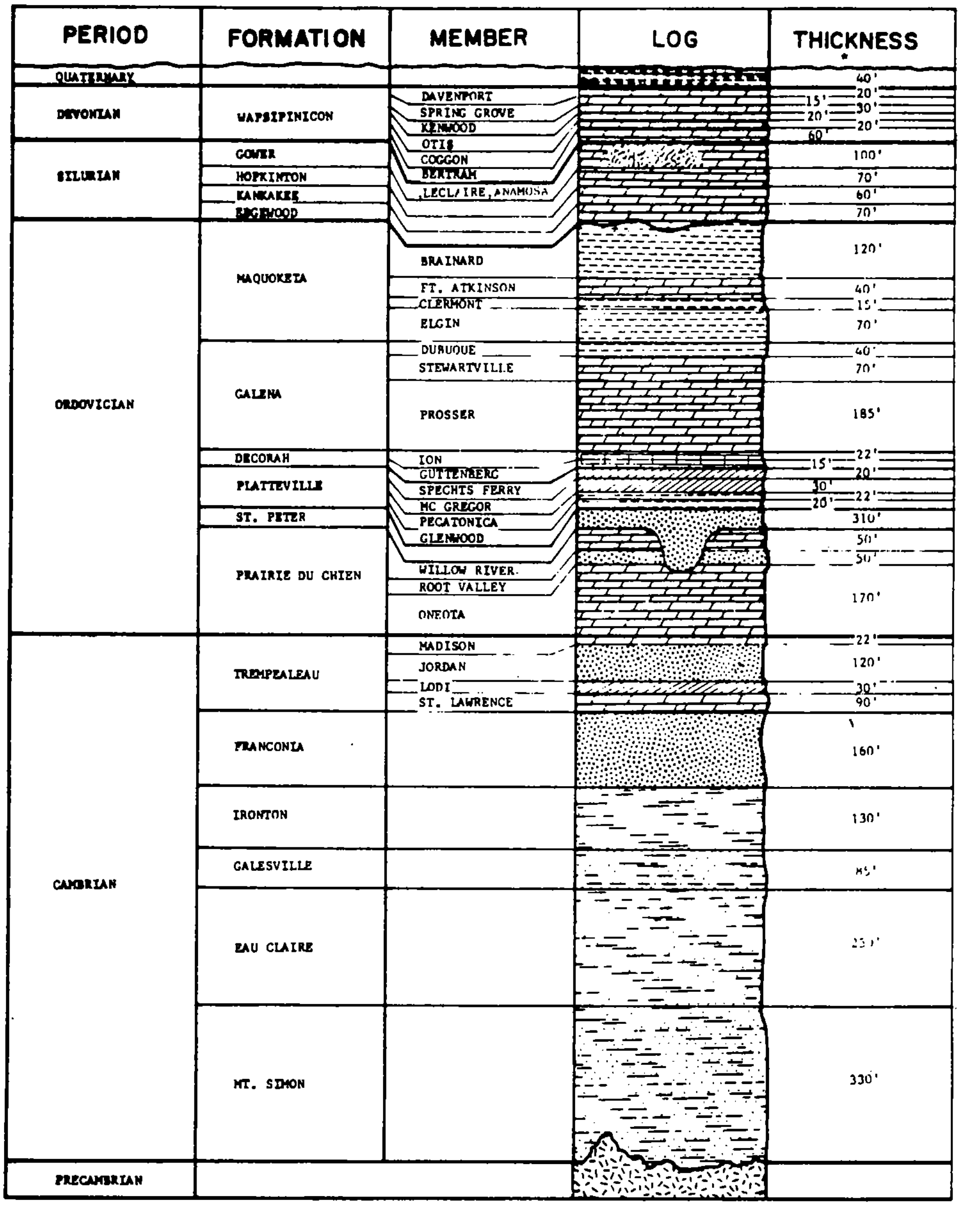
Regional Geologic Map

Figure 2.5-1



*APPROXIMATE THICKNESS IN FEET OF ROCK UNITS IN THE SUBSURFACE

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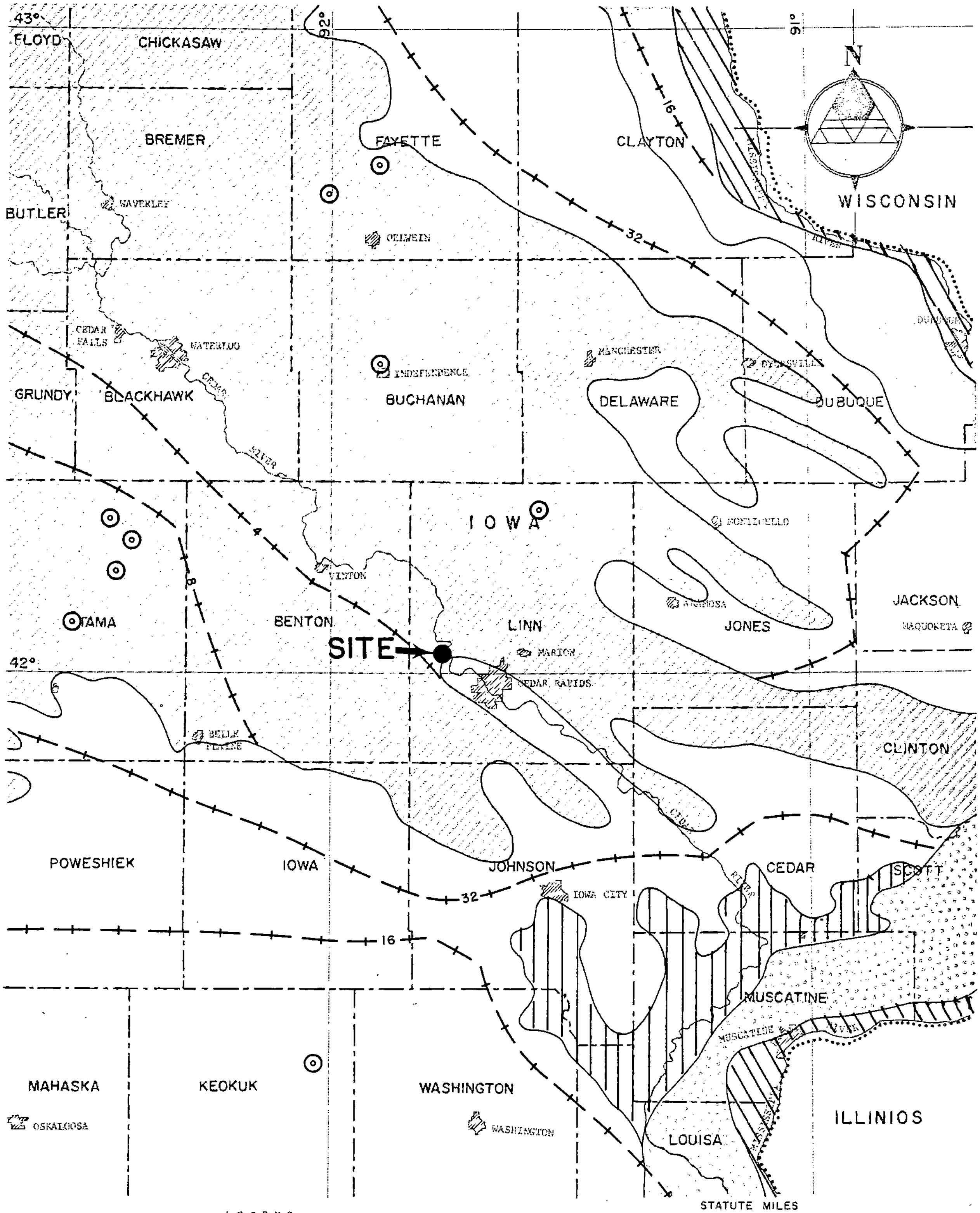
Generalized Stratigraphic Column
of Eastern Iowa

Figure 2.5-2

NOTE:

BASED ON A COLUMN BY R. B. CAMPBELL DRAWN BY J. N. ROSE.



LEGEND:

COLOR	DESCRIPTION	COLOR	DESCRIPTION
	WISCONSIN DRIFTS-IOWAN-LOESS-MANTLED		PRE-WISCONSIN DEPOSITS-KANSAN DRIFT
	PRE-WISCONSIN DEPOSITS-ILLINOIAN DRIFT		DRIFTLESS AREA
	PRE-WISCONSIN DEPOSITS-GLACIAL LAKE CALVIN (ILLINOIAN) BEDS		ALLUVIUM

WISCONSIN LOESS THICKNESS IN FEET   RADIOCARBON SITES

REFERENCE:

THIS MAP WAS PREPARED FROM A PORTION OF USGS BASE MAP OF THE STATE OF IOWA, 1950.

THIS MAP WAS PREPARED FROM AN ENLARGED PORTION OF A FIGURE ENTITLED "PLEISTOCENE DEPOSITS IN IOWA", GLACIATION OF MINNESOTA AND IOWA BY H. E. WRIGHT JR. R. V. RUHE, 1965.

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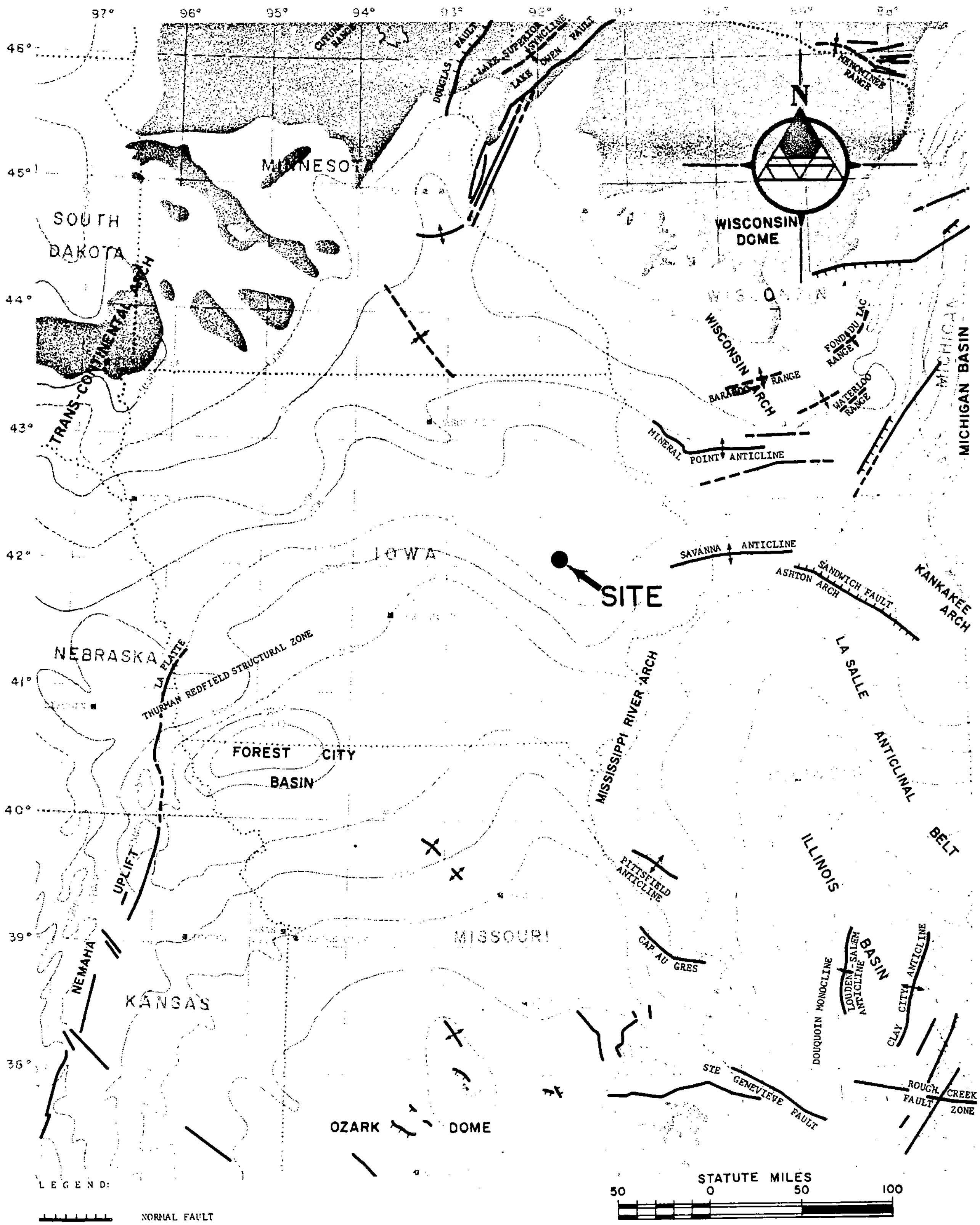
Pleistocene Deposits in Eastern Iowa

Figure 2.5-3



LEGEND:

NORMAL FAULT

ANTICLINAL AXIS

SYNCLINAL AXIS

REFERENCE:
THIS MAP WAS PREPARED FROM A PORTION OF THE FOLLOWING USGS "IFR WALL PLANNING CHART EAST AND WEST 1968". TECTONIC FEATURES TAKEN FROM A PORTION OF THE "TECTONIC MAP OF THE UNITED STATES" BY USGS AND AAPG, 1962. BASEMENT FEATURES TAKEN FROM "BASEMENT MAP OF NORTH AMERICA" BY USGS, 1967.

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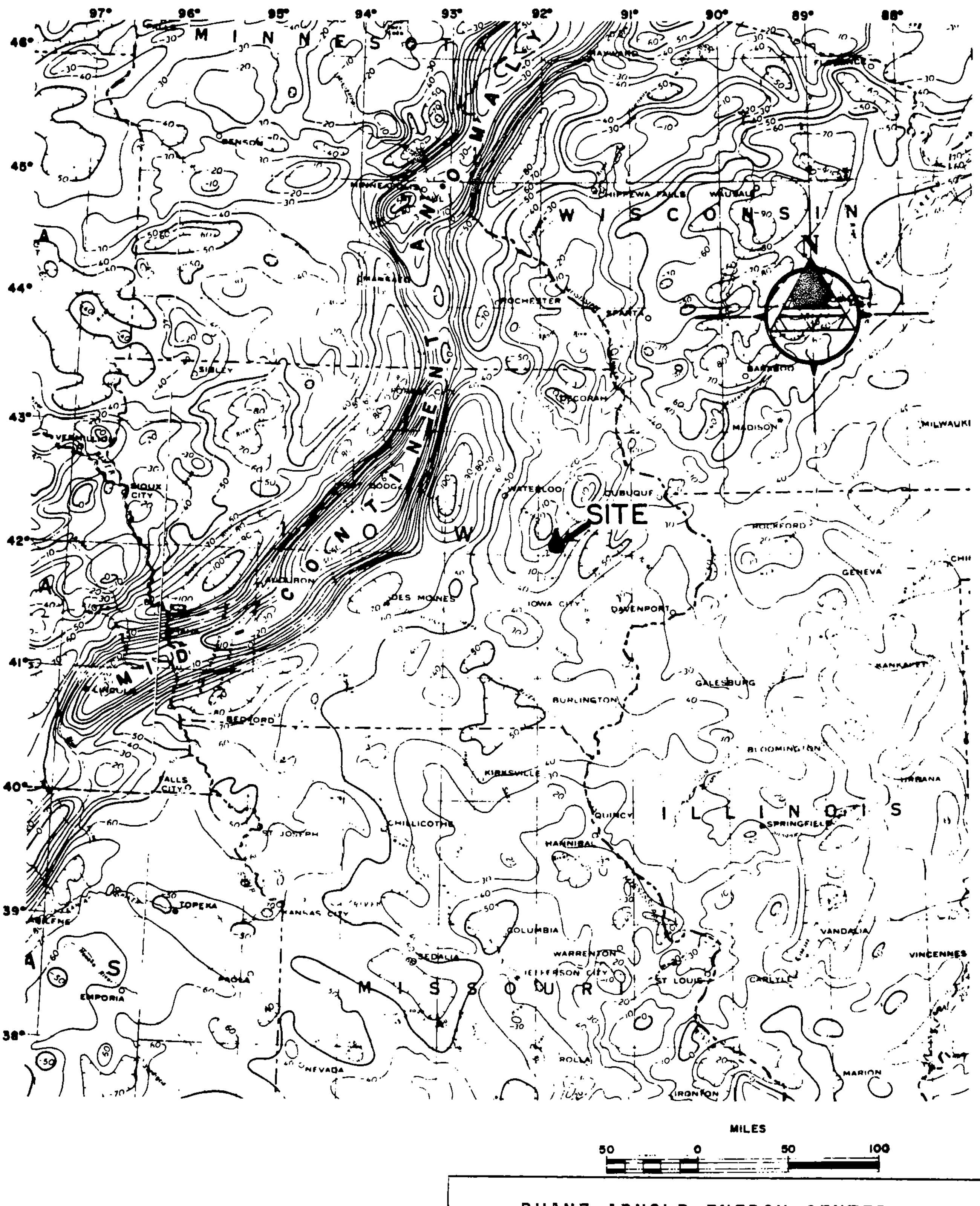
Regional Tectonic Map

Figure 2.5-4



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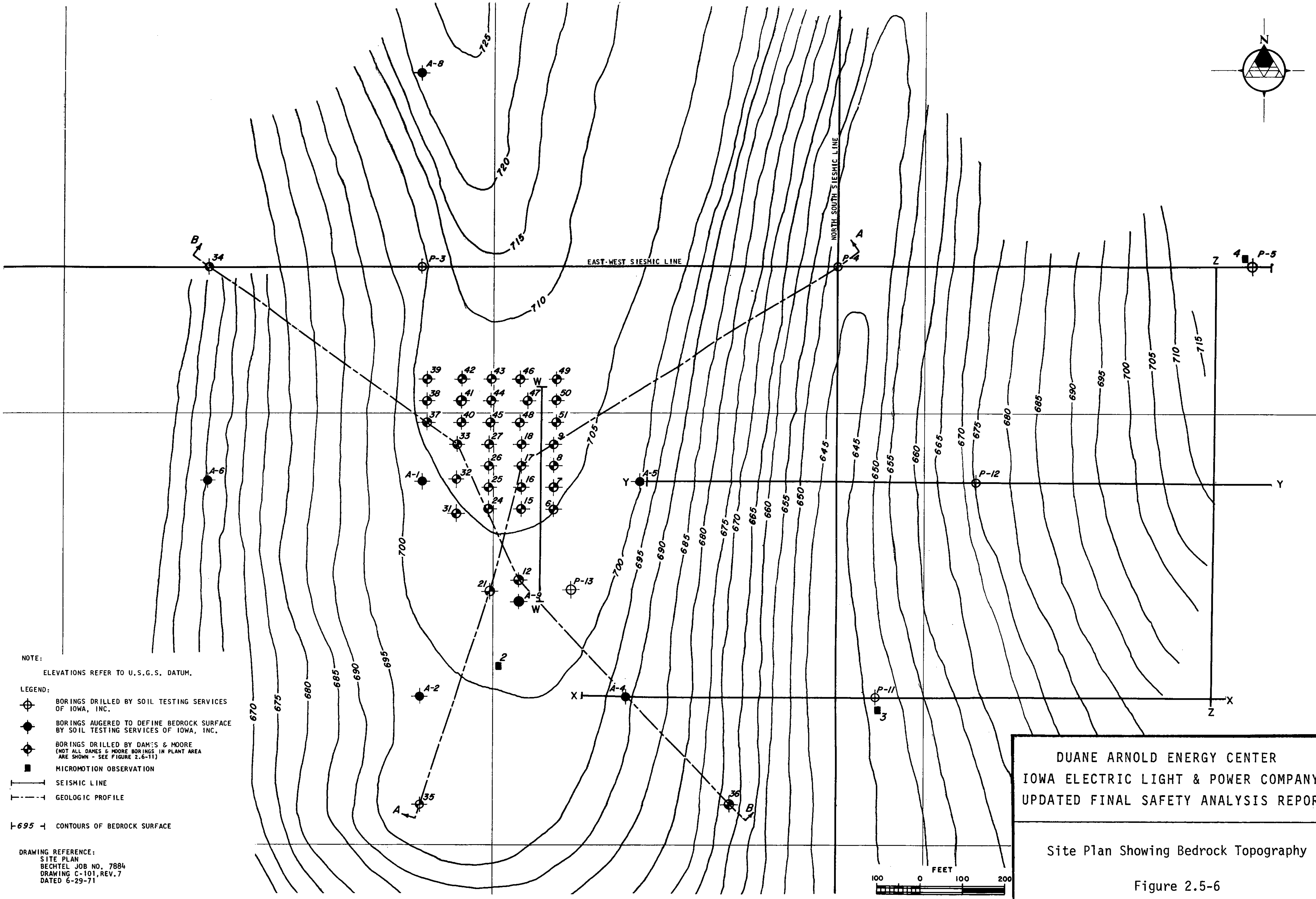
Regional Bouguer Gravity Map

Figure 2.5-5

REFERENCE:

THIS MAP WAS PREPARED FROM A PORTION OF "BOUGUER GRAVITY ANOMALY MAP OF THE UNITED STATES" BY A. G. U. AND G. S. 1964.



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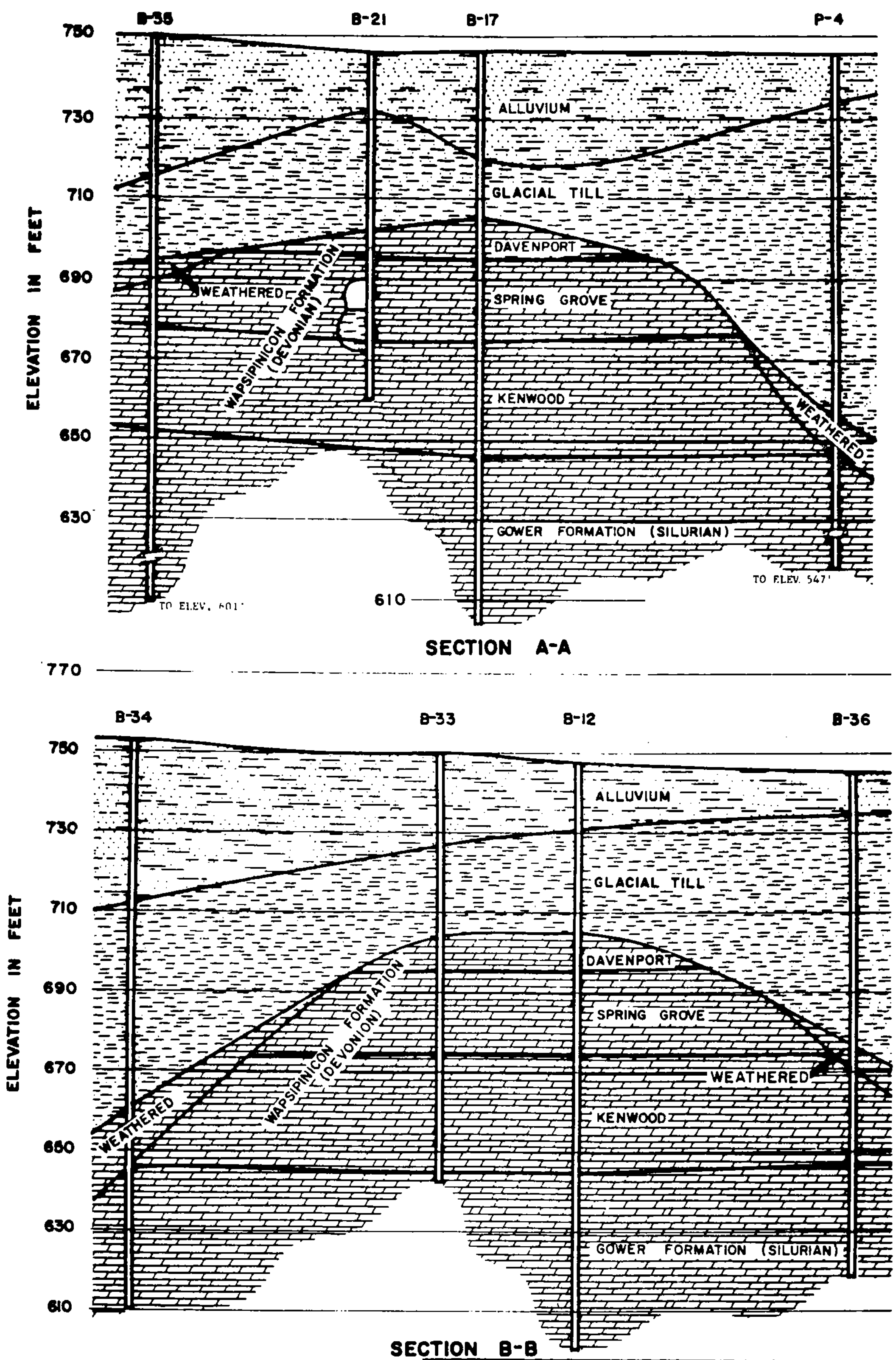
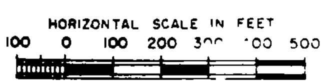
Site Plan Showing Bedrock Topography

Figure 2.5-6





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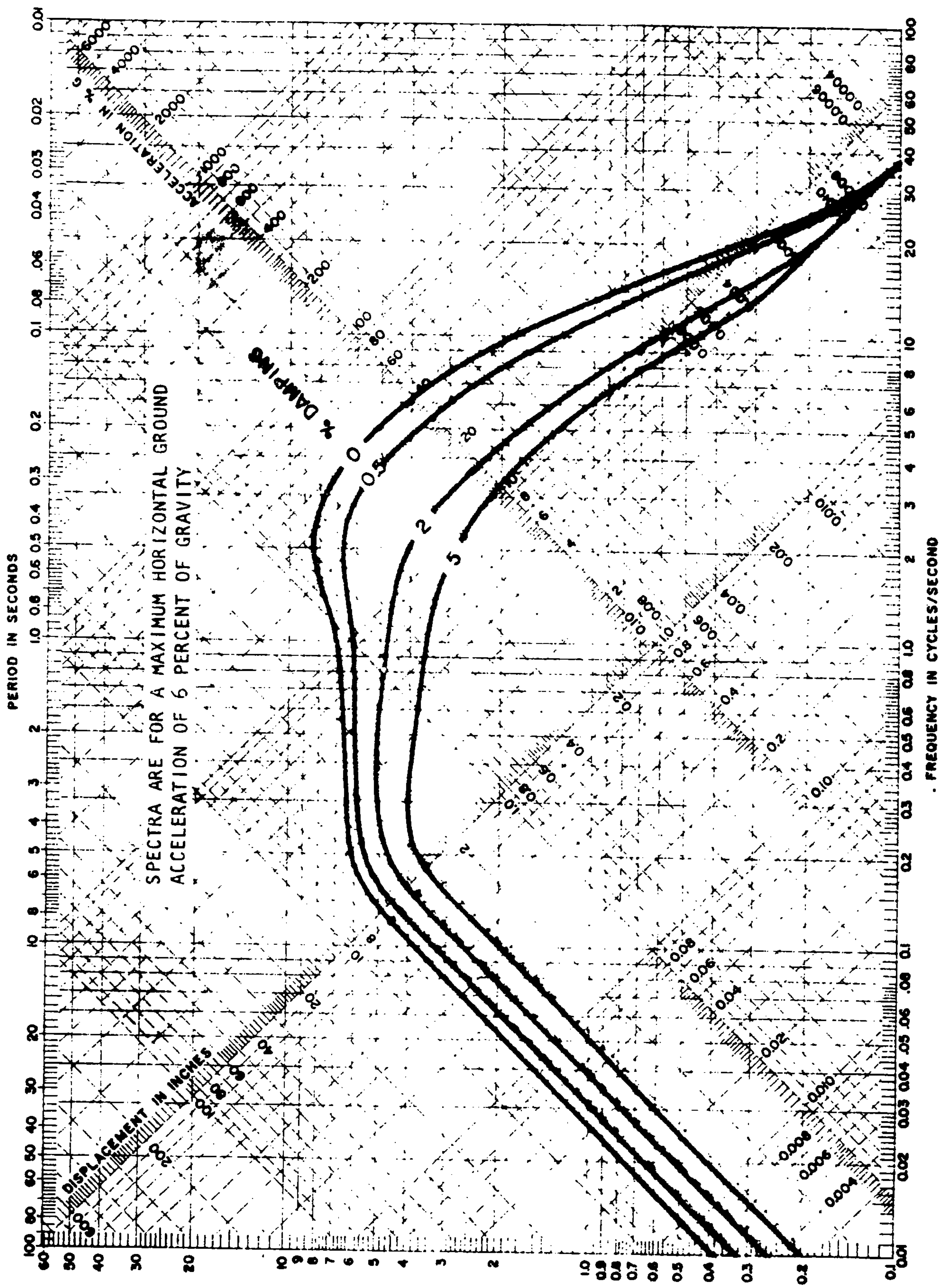
Geologic Profiles

Figure 2.5-7



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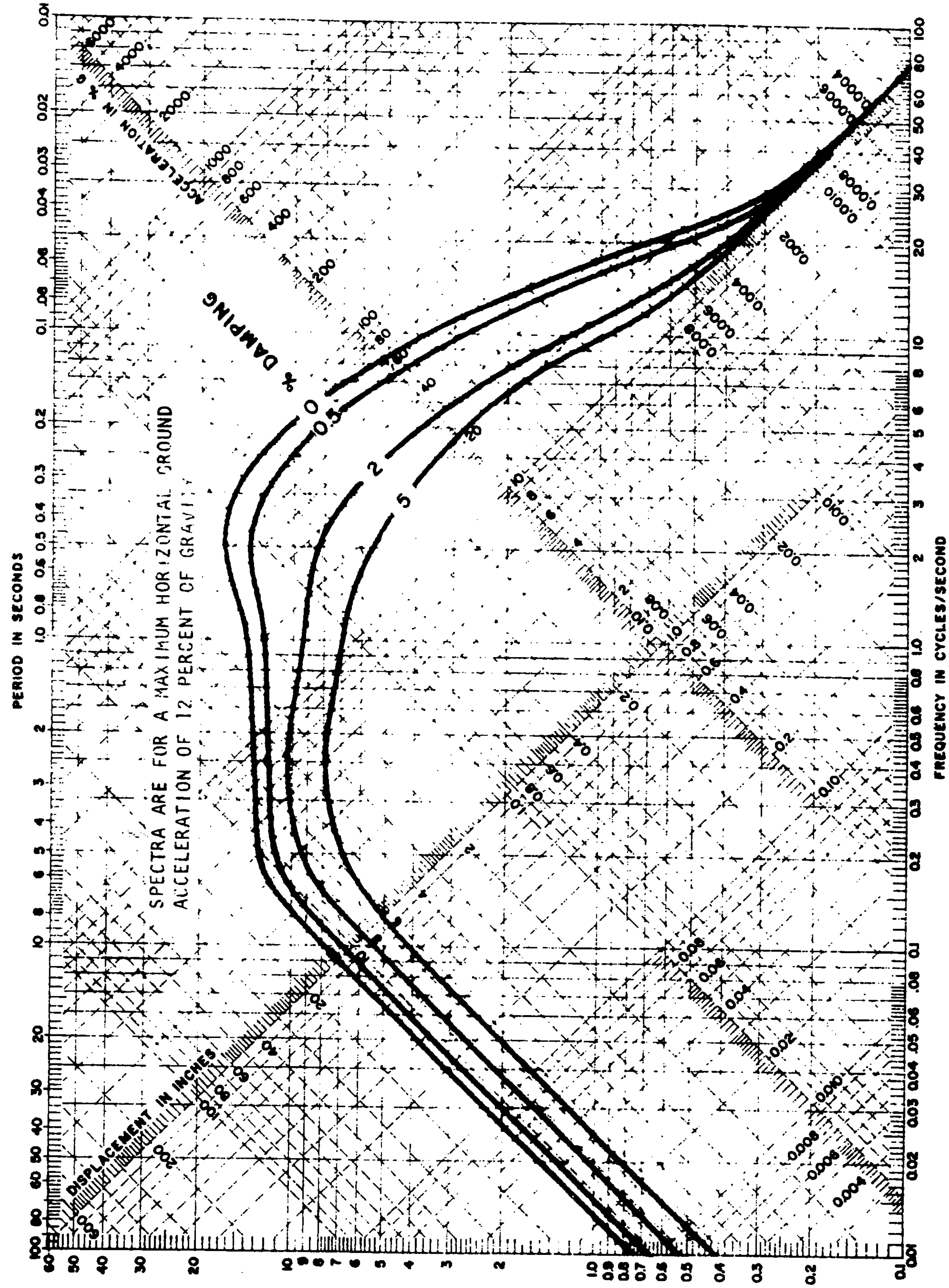
Foundation Level Response Spectra Structures Supported on Rock or Lean Concrete Fill Operating Basis Earthquake

Figure 2.5-8, Sheet 1



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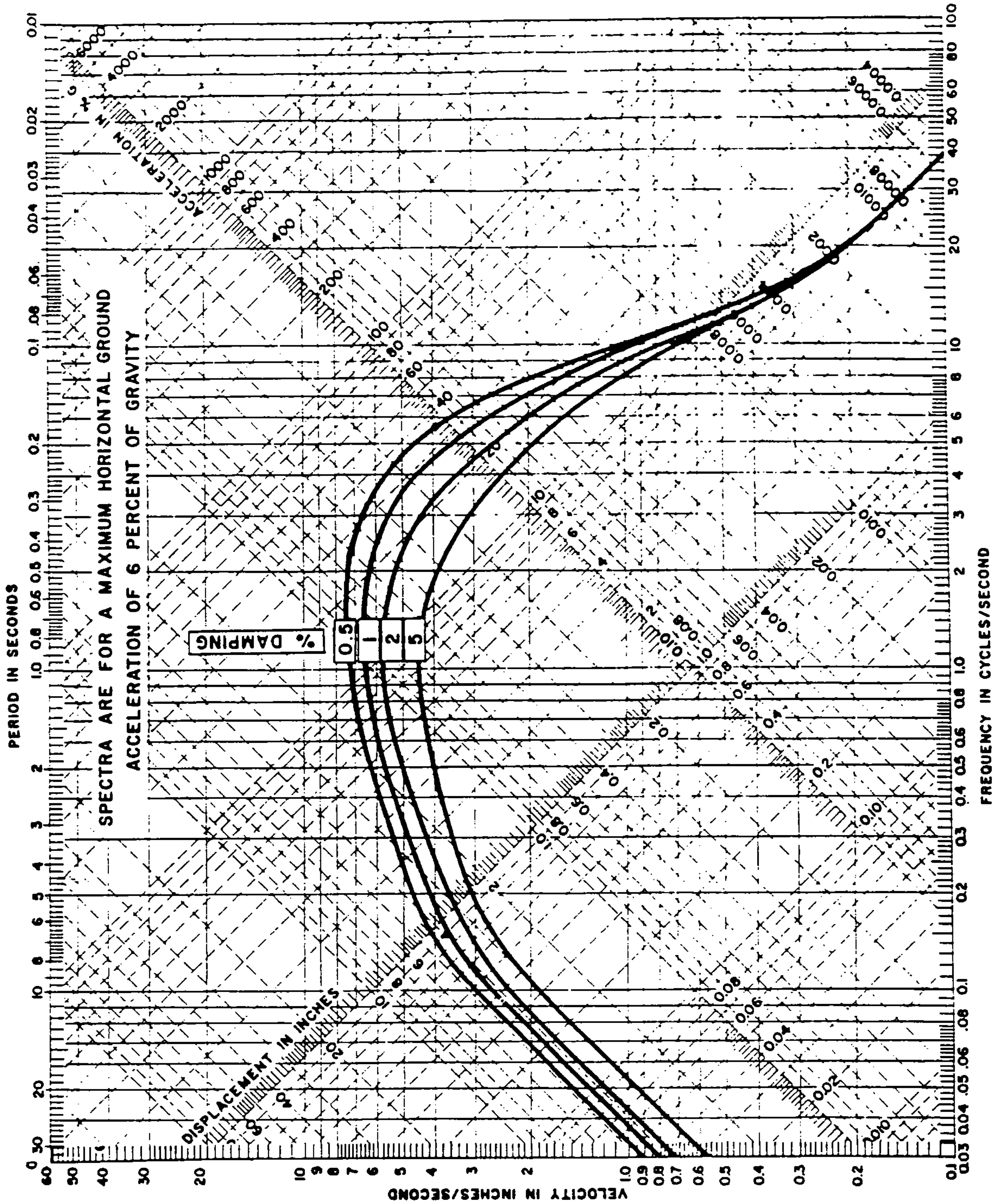
Foundation Level Response Spectra Structures
Supported on Rock or Lean Concrete Fill
Design Basis Earthquake

Figure 2.5-8, Sheet 2



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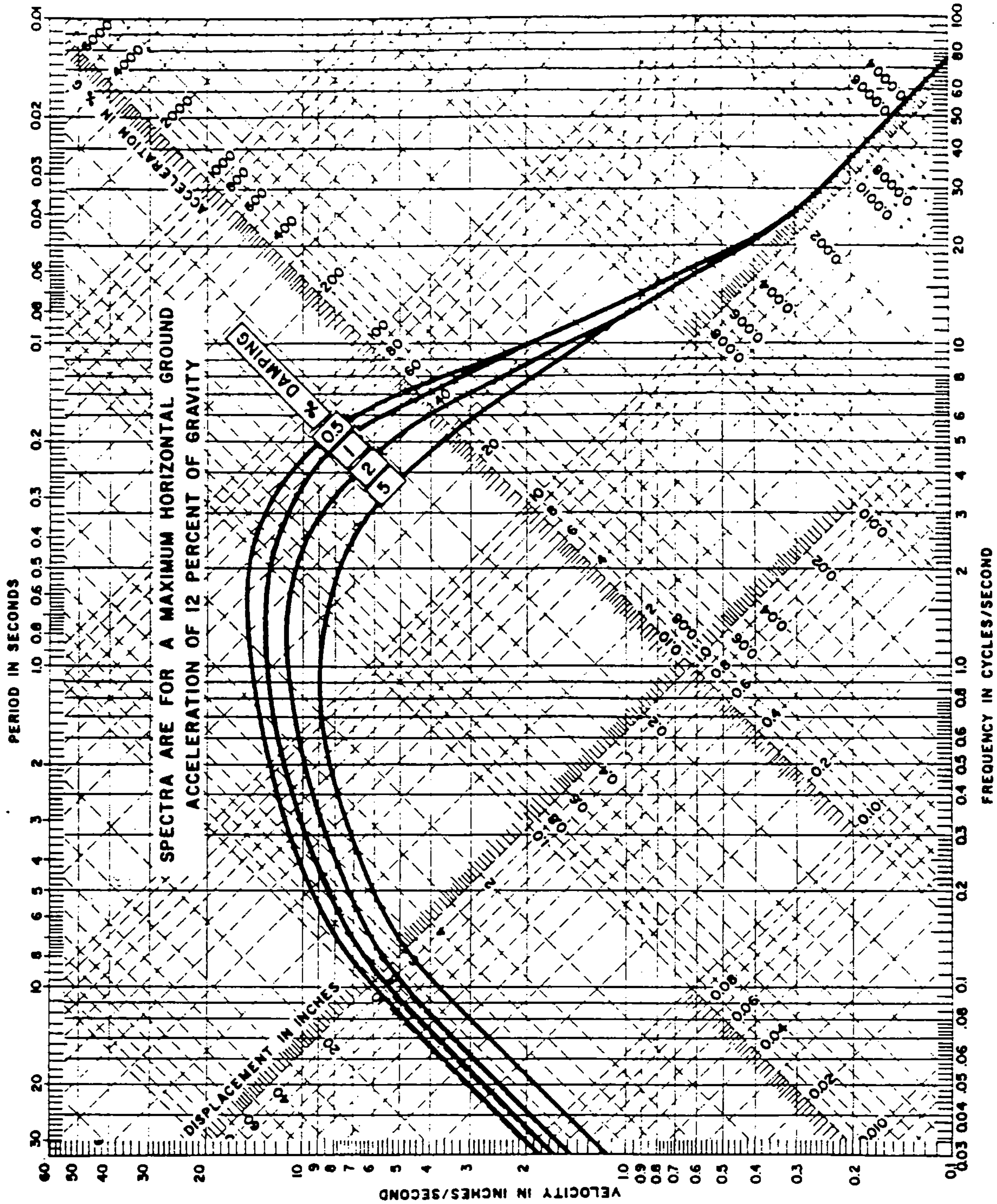
Foundation Level Response Spectra
Structures Supported on About 10 ft. of
Soil Over Bedrock
Operating Basis Earthquake

Figure 2.5-8, Sheet 3



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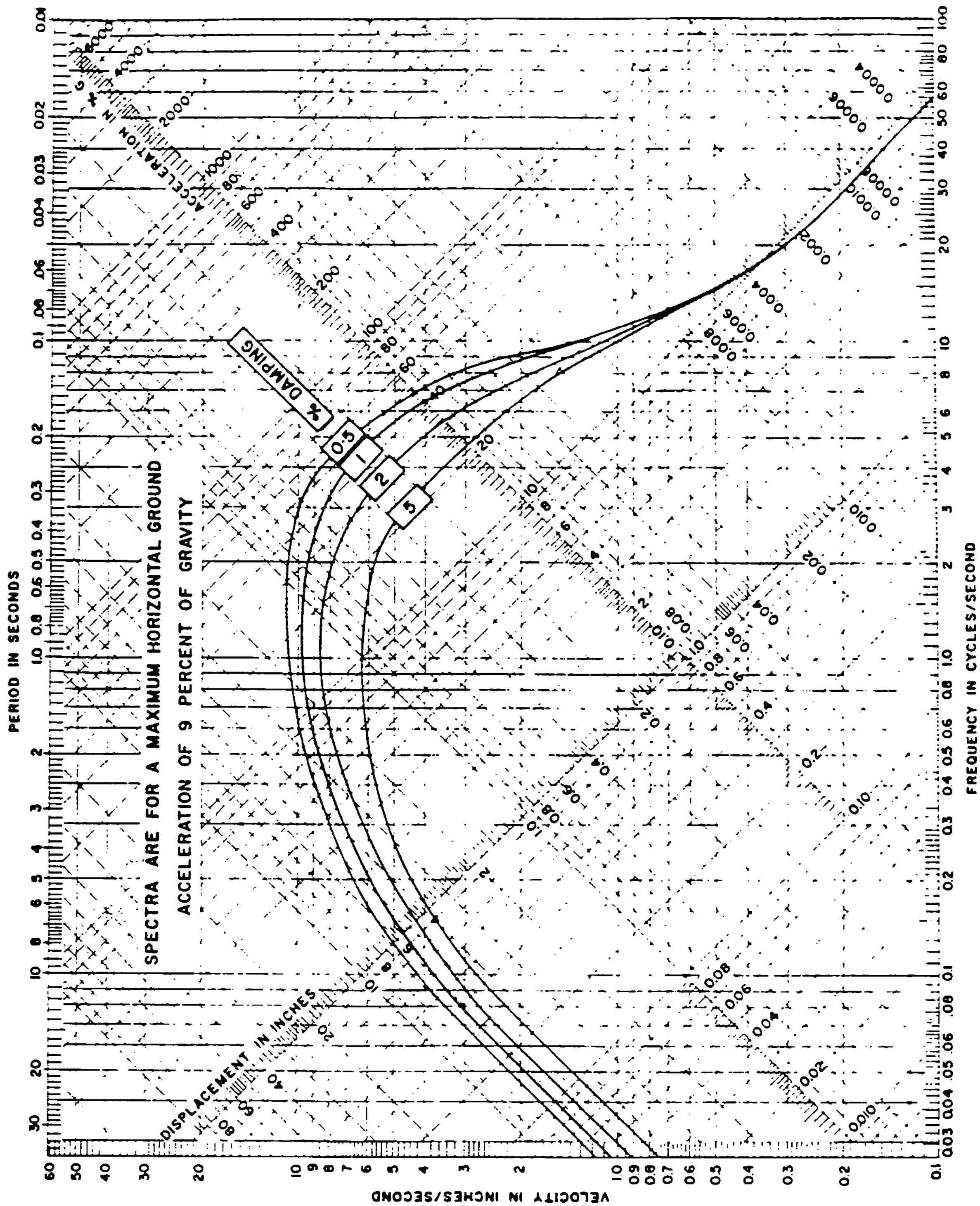
Foundation Level Response Spectra
Structures Supported on About 10 ft. of
Soil Over Bedrock
Design Basis Earthquake

Figure 2.5-8, Sheet 4



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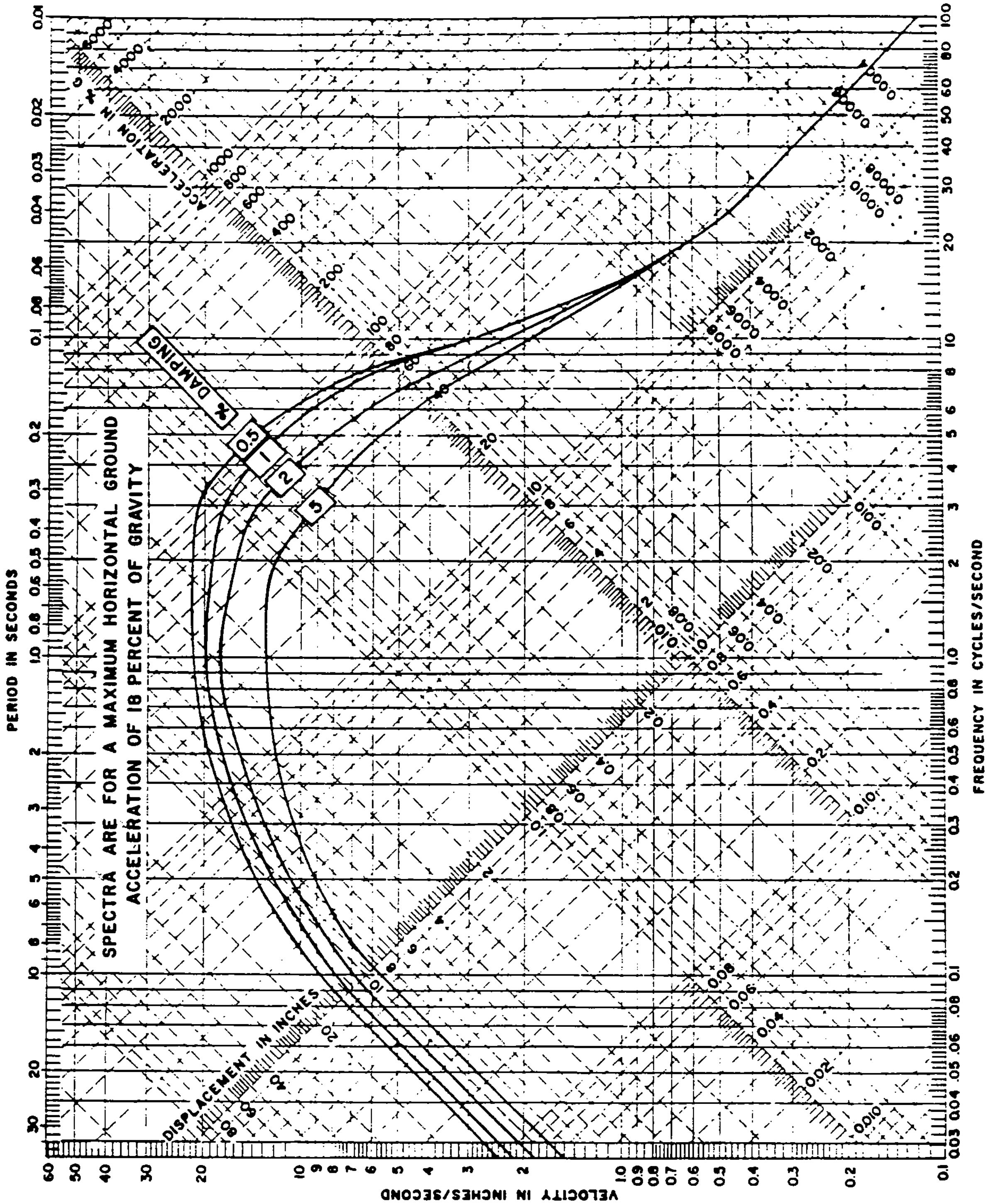
Foundation Level Response Spectra
Structures Supported on About 30 to 50
ft. of Soil Over Bedrock
Operating Basis Earthquake

Figure 2.5-8, Sheet 5



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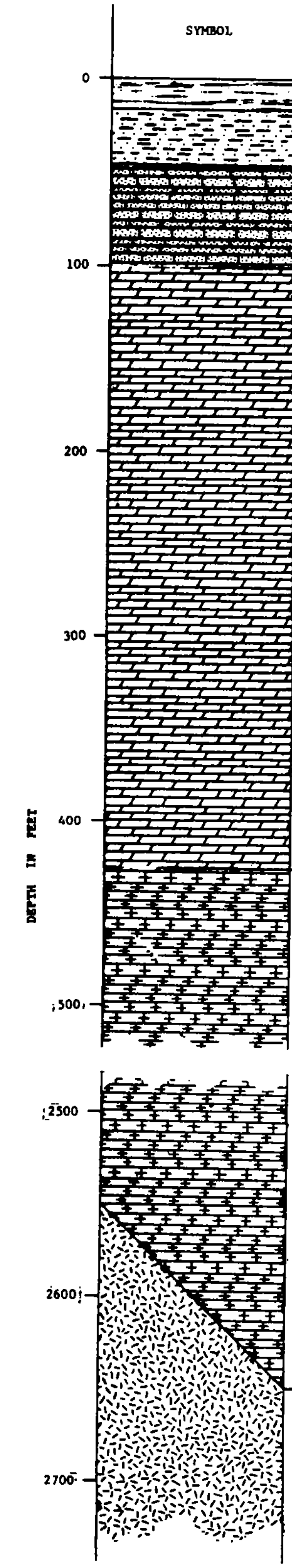
Foundation Level Response Spectra
Structures Supported on About 30 to 50
ft. of Soil Over Bedrock
Design Basis Earthquake

Figure 2.5-8, Sheet 6



DESCRIPTION	COMPRESSIONAL WAVE VELOCITY (FT/SEC)	POISSON'S RATIO (ESTIMATED)	SHEAR WAVE VELOCITY (FT/SEC) (COMPUTED)	TOTAL UNIT WEIGHT (LBS/CU. FT.)
ALLUVIAL	1140	0.37	500	120
SAND	5500±300			
GLACIAL TILL	6100±300	0.45	1800	135
WAPSIPINICON FORMATION (LIMESTONE AND DOLOMITE)	14,000±500	0.20	8600	160
SILURIAN (LIMESTONE AND DOLOMITE)	14,000±500	0.20	8600	160 (ESTIMATED)
ORDOVICIAN AND CAMBRIAN SEDIMENTS (PREDOMINATELY DOLOMITE)	16000±2000 (ESTIMATED)	.20	9800	155 (ESTIMATED)
PRECAMBRIAN BASEMENT (IGNEOUS AND METAMORPHIC ROCKS)	18,000±2000 (ESTIMATED)	.15	11500	175 (ESTIMATED)

NOTE:
PHYSICAL PROPERTIES ARE MEASURED VALUES UNLESS NOTED OTHERWISE.

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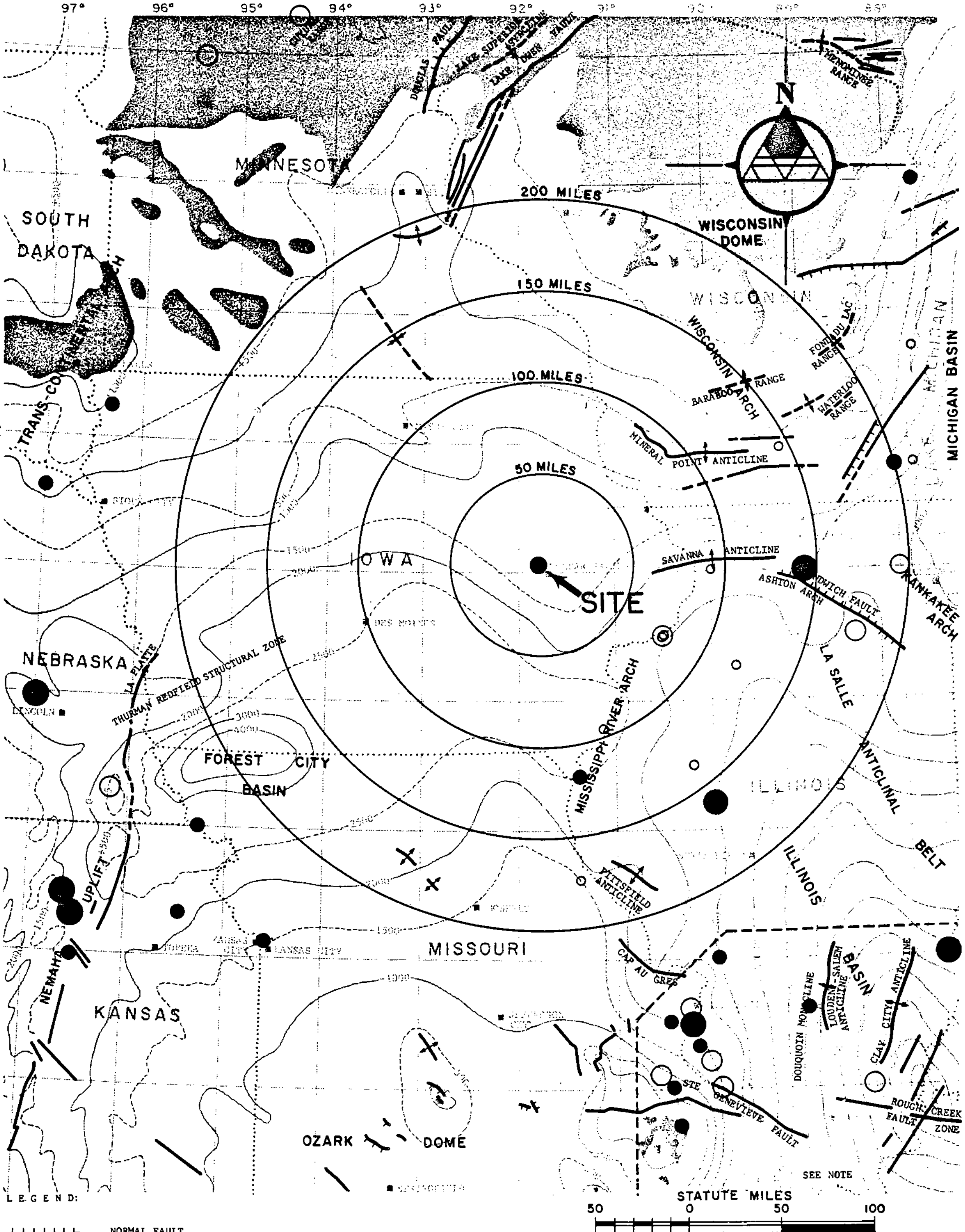
Stratigraphic Section Showing Geophysical Data

Figure 2.5-9



LEGEND:

NORMAL FAULT

ANTICLINAL AXIS

SYNCLINAL AXIS

CONTOURS ON BASEMENT SURFACE ARE USGS DATUM

PRE-CAMBRIAN ROCK OUTCROPS

INTENSITY IV OR LESS

INTENSITY V

INTENSITY VI

INTENSITY VII

REFERENCE:

THIS MAP WAS PREPARED FROM A PORTION OF THE FOLLOWING USGS "IFR WALL PLANNING CHART EAST AND WEST 1968". TECTONIC FEATURES TAKEN FROM A PORTION OF THE "TECTONIC MAP OF THE UNITED STATES" BY USGS AND AAPG, 1962. BASEMENT FEATURES TAKEN FROM "BASEMENT MAP OF NORTH AMERICA" BY USGS, 1967.

NOTE:

THE AREA BOUNDED BY THE DASHED LINE HAS A HIGHER DEGREE OF SEISMICITY THAN THE SITE AREA. ONLY EARTHQUAKES WITH EPICENTRAL INTENSITIES OF V AND GREATER ARE INDICATED.

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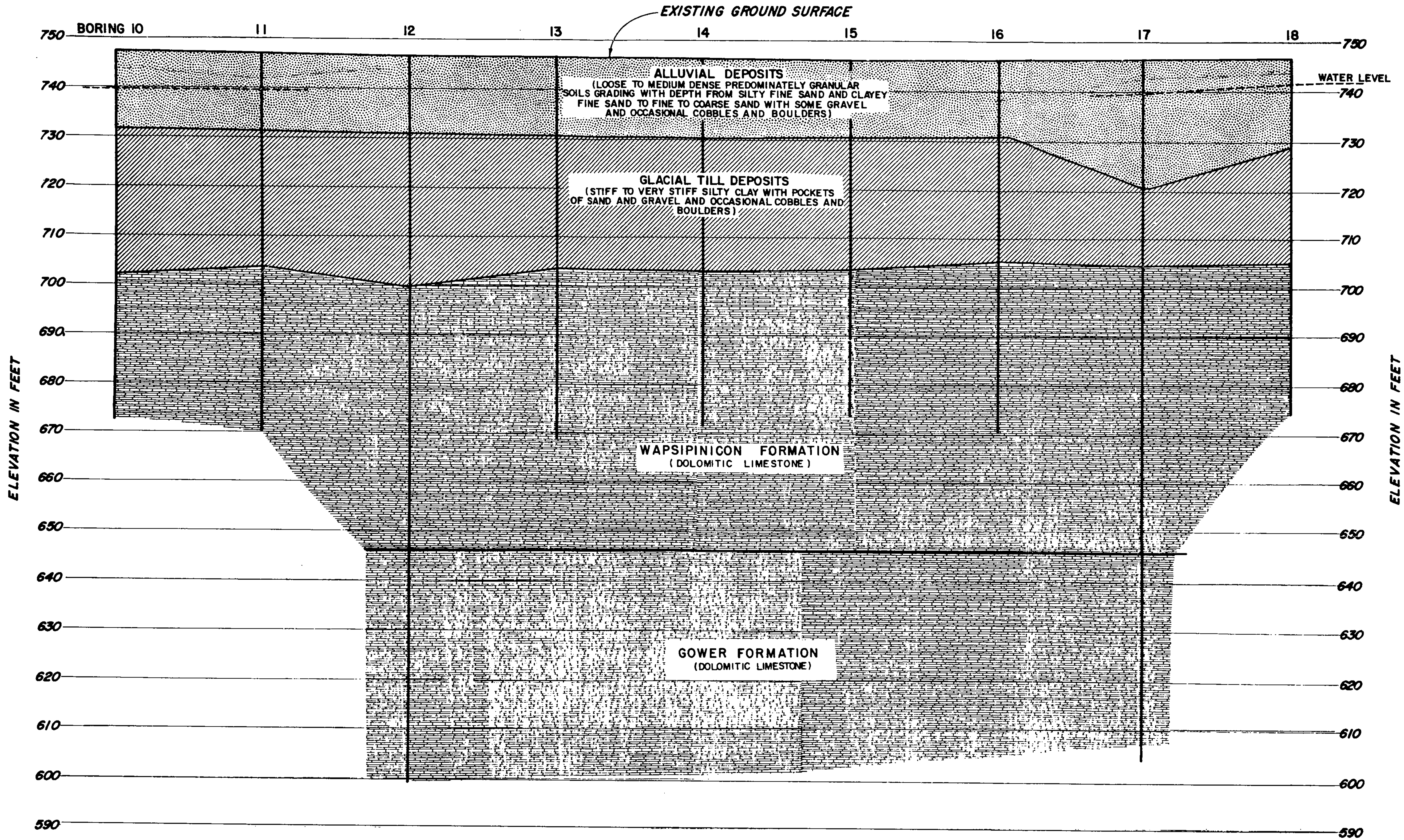
Epicenter Map

Figure 2.5-10

[redacted]



NOTES.

ELEVATIONS REFER TO U.S.G.S. DATUM.

GROUND SURFACE ELEVATIONS ARE CORRECT ONLY AT TEST BORING LOCATIONS.

THE DEPTH AND THICKNESS OF THE SOIL STRATA AND THE DEPTH OF THE ROCK STRATA INDICATED ON THE SUBSURFACE SECTION WERE OBTAINED BY INTERPOLATING BETWEEN TEST BORINGS. INFORMATION ON ACTUAL SOIL AND ROCK CONDITIONS EXISTS ONLY AT THE TEST BORING LOCATIONS AND IT IS POSSIBLE THAT THE SOIL AND ROCK CONDITIONS BETWEEN THE TEST BORINGS MAY VARY FROM THOSE INDICATED.

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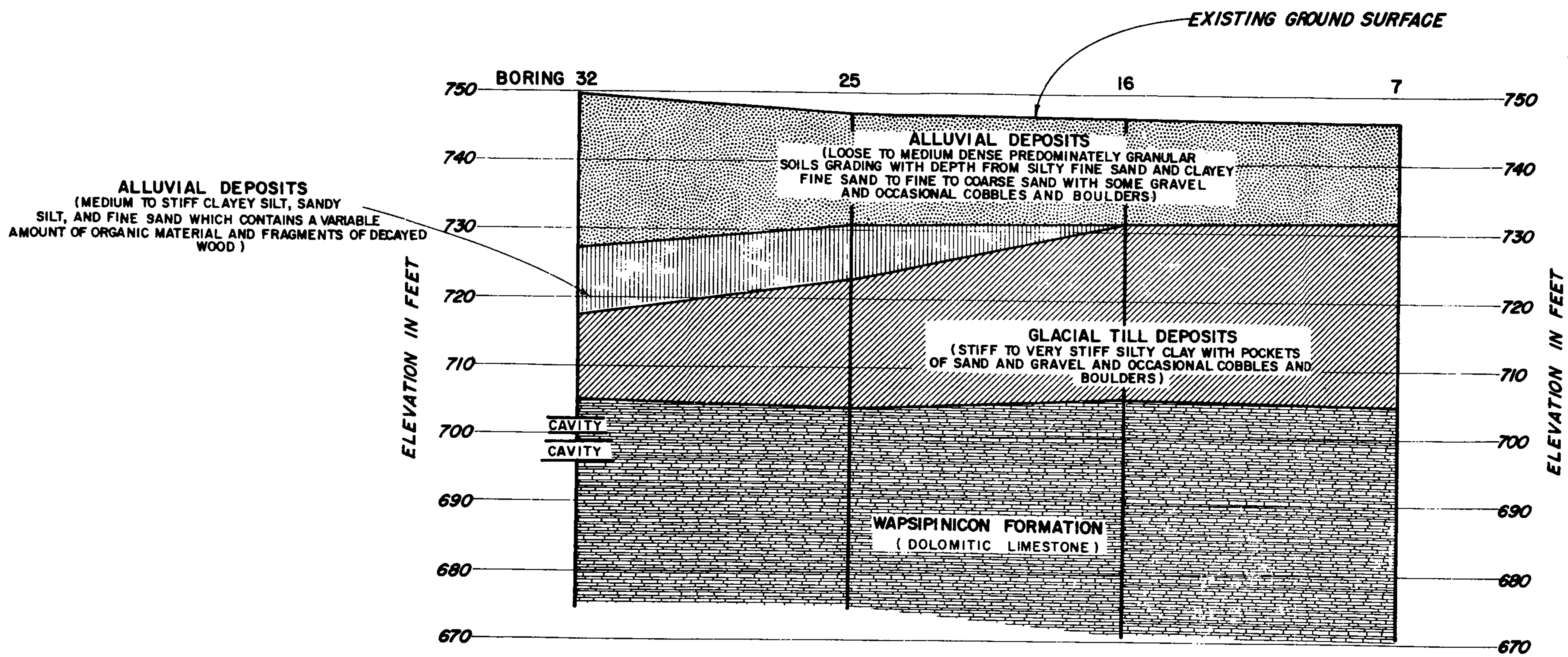
Generalized Subsurface Section A-A

Figure 2.5-12

Revision 5 - 6/87

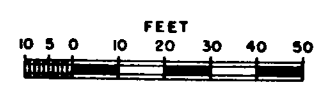


NOTES:

ELEVATIONS REFER TO U.S.G.S. DATUM.

GROUND SURFACE ELEVATIONS ARE CORRECT ONLY AT TEST BORING LOCATIONS.

THE DEPTH AND THICKNESS OF THE SOIL STRATA AND THE DEPTH OF THE ROCK STRATA INDICATED ON THE SUBSURFACE SECTION WERE OBTAINED BY INTERPOLATING BETWEEN TEST BORINGS. INFORMATION ON ACTUAL SOIL AND ROCK CONDITIONS EXISTS ONLY AT THE TEST BORING LOCATIONS AND IT IS POSSIBLE THAT THE SOIL AND ROCK CONDITIONS BETWEEN THE TEST BORINGS MAY VARY FROM THOSE INDICATED.



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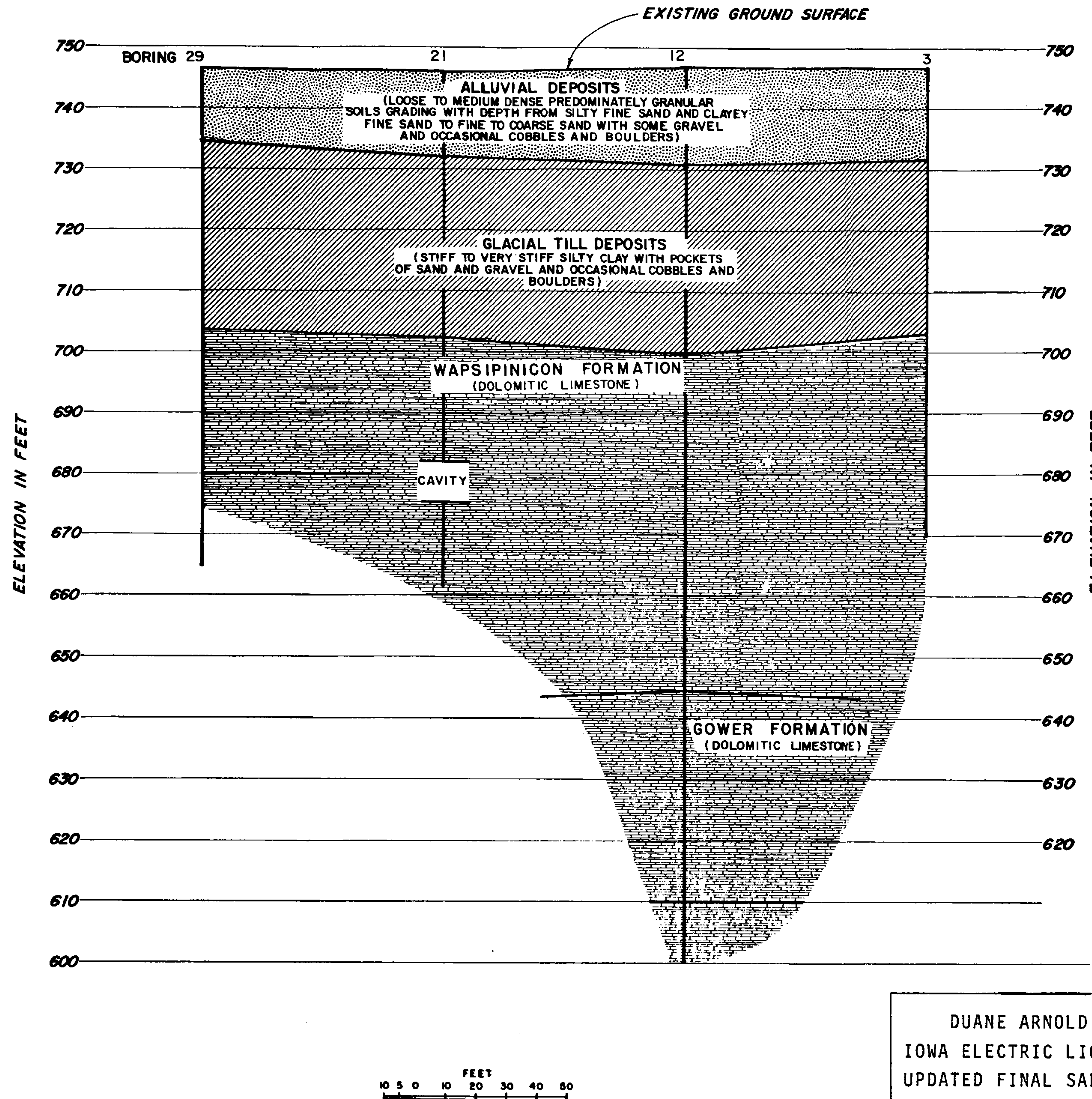
Generalized Subsurface Section B-B

Figure 2.5-13



NOTES:

ELEVATIONS REFER TO U.S.G.S. DATUM.

GROUND SURFACE ELEVATIONS ARE CORRECT ONLY AT TEST BORING LOCATIONS.

THE DEPTH AND THICKNESS OF THE SOIL STRATA AND THE DEPTH OF THE ROCK STRATA INDICATED ON THE SUBSURFACE SECTION WERE OBTAINED BY INTERPOLATING BETWEEN TEST BORINGS. INFORMATION ON ACTUAL SOIL AND ROCK CONDITIONS EXISTS ONLY AT THE TEST BORING LOCATIONS AND IT IS POSSIBLE THAT THE SOIL AND ROCK CONDITIONS BETWEEN THE TEST BORINGS MAY VARY FROM THOSE INDICATED.

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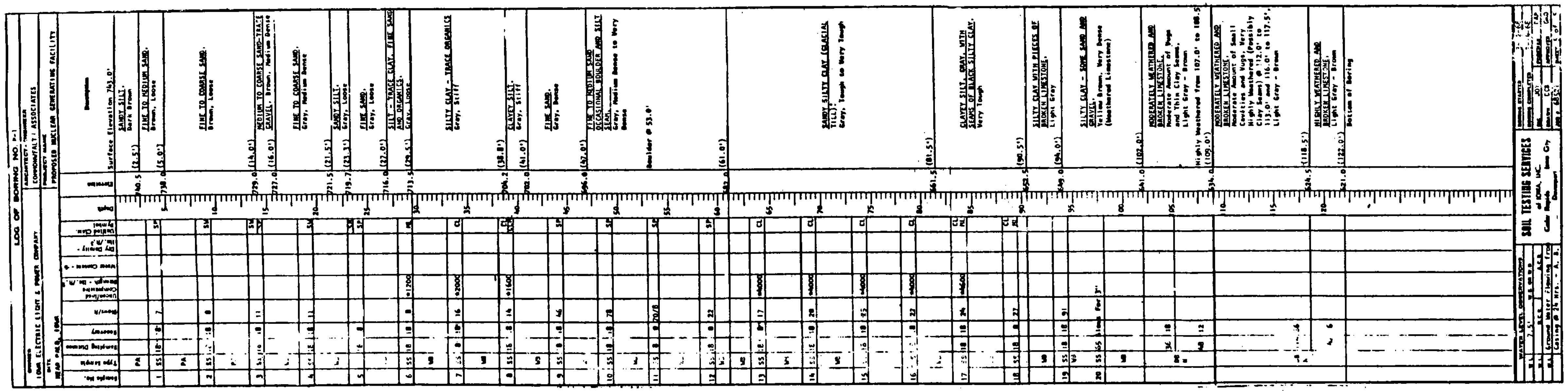
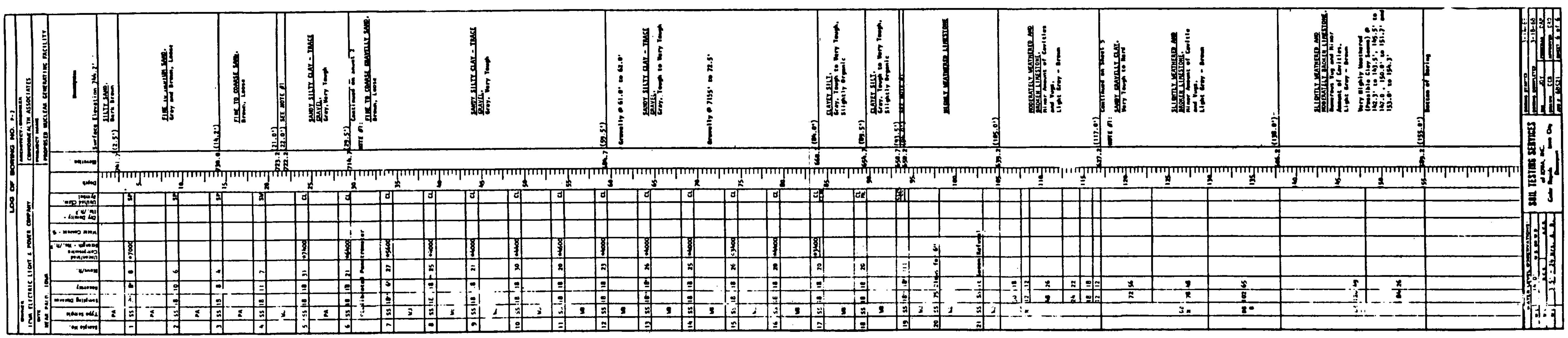
Generalized Subsurface Section C-C

Figure 2.5-14

[redacted]





NOTE:

ELEVATIONS REFER TO U.S.G.S. DATUM.

SS IN THE SAMPLE TYPE COLUMN INDICATES A STANDARD SPLIT SPOON SAMPLE (2 INCH OUTSIDE DIAMETER).

ST IN THE SAMPLE TYPE COLUMN INDICATES A SHELBY TUBE SAMPLE (3 INCH OUTSIDE DIAMETER).

RB IN THE SAMPLE TYPE COLUMN INDICATES THAT THE BEDROCK WAS DRILLED WITH A ROCK BIT.

DB IN THE SAMPLE TYPE COLUMN INDICATES THAT THE BEDROCK WAS CORED WITH A BX OR NX CORE BARREL HAVING A DIAMOND BIT.

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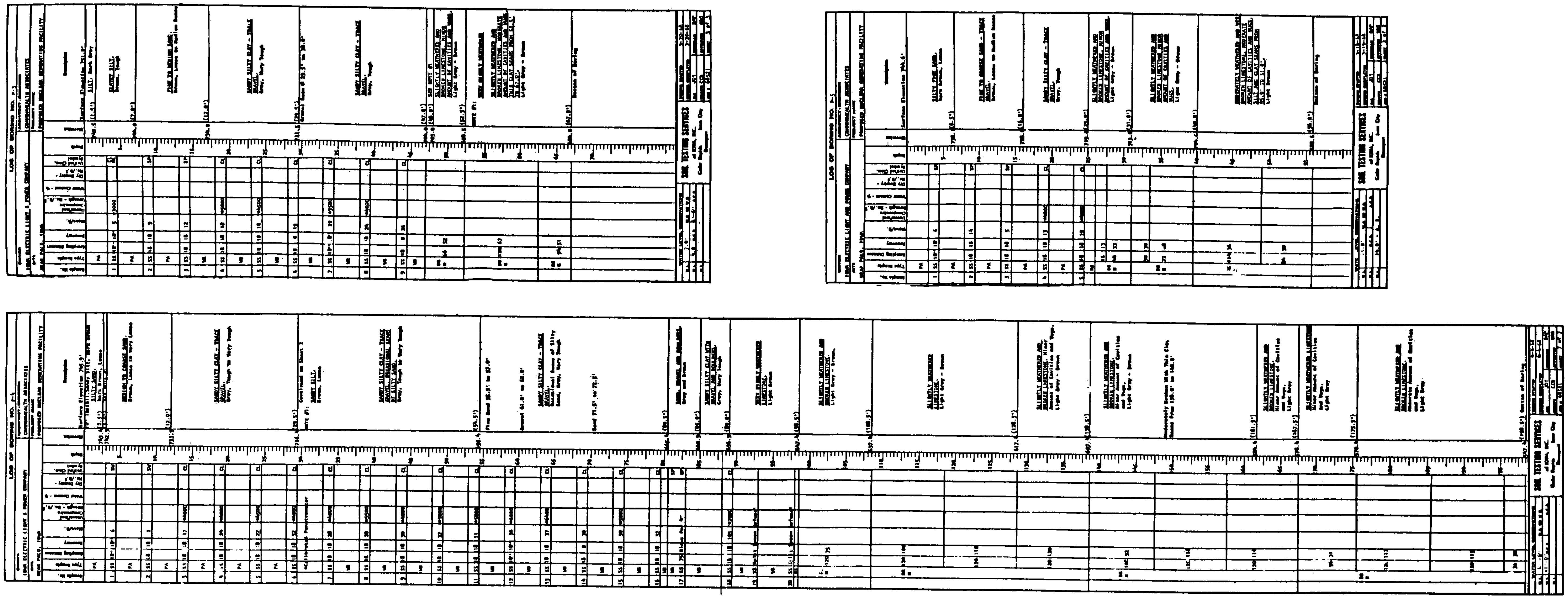
Log of Borings - Numbers P-1 and P-2

Figure 2.5-17



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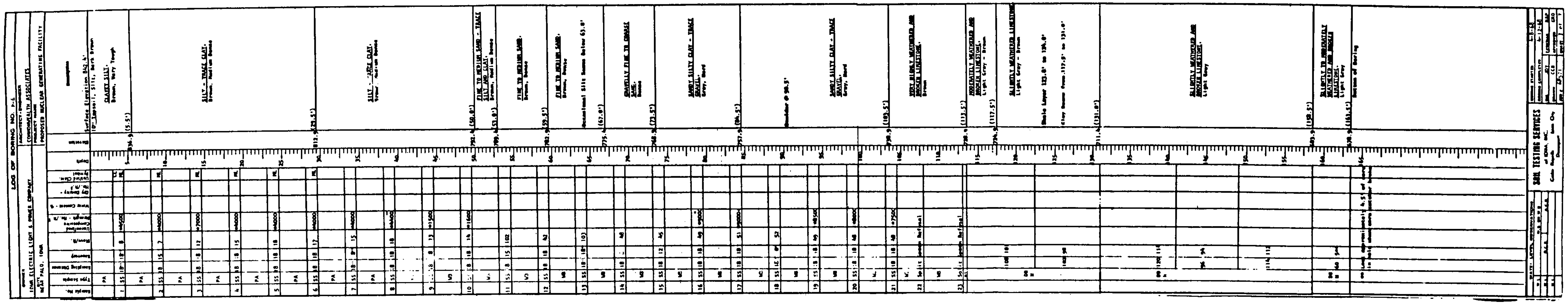
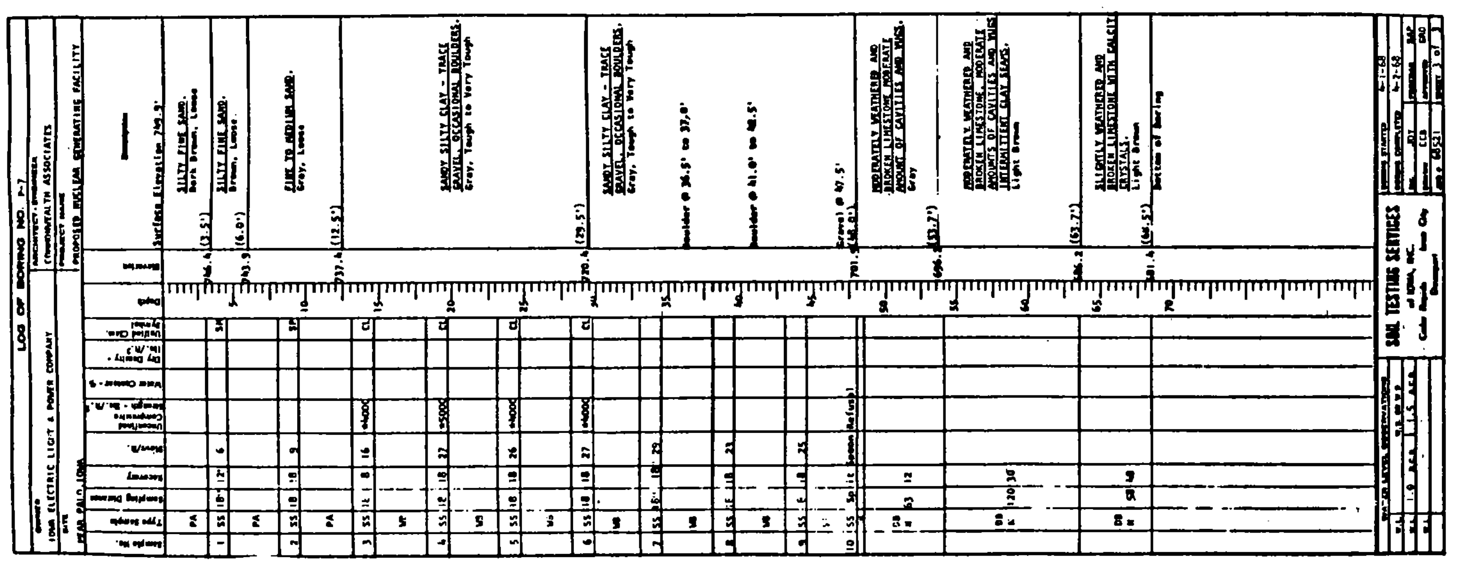
Log of Borings - Numbers P-3 and P-4

Figure 2.5-18





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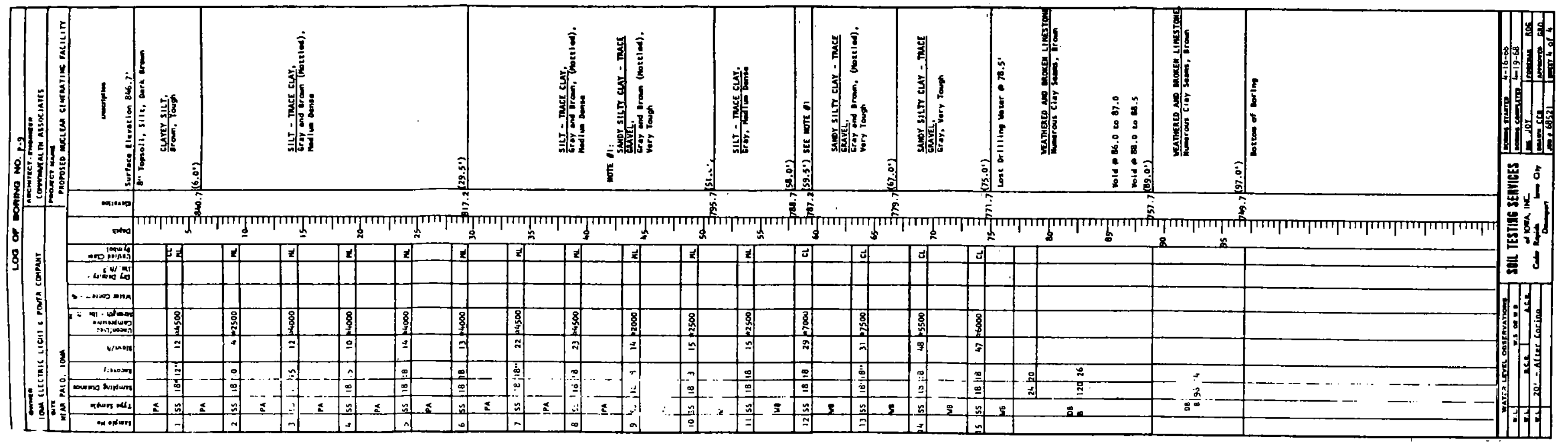
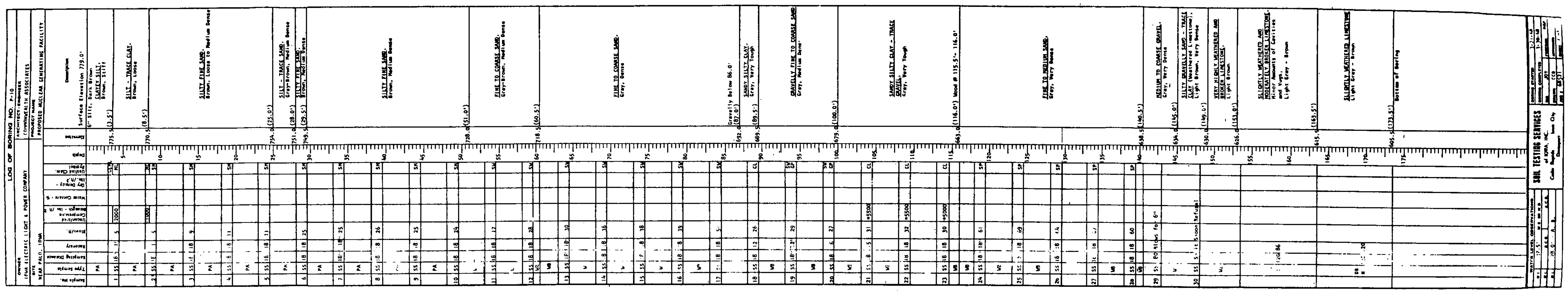
Log of Borings - Number P-6 and P-7

Figure 2.5-19





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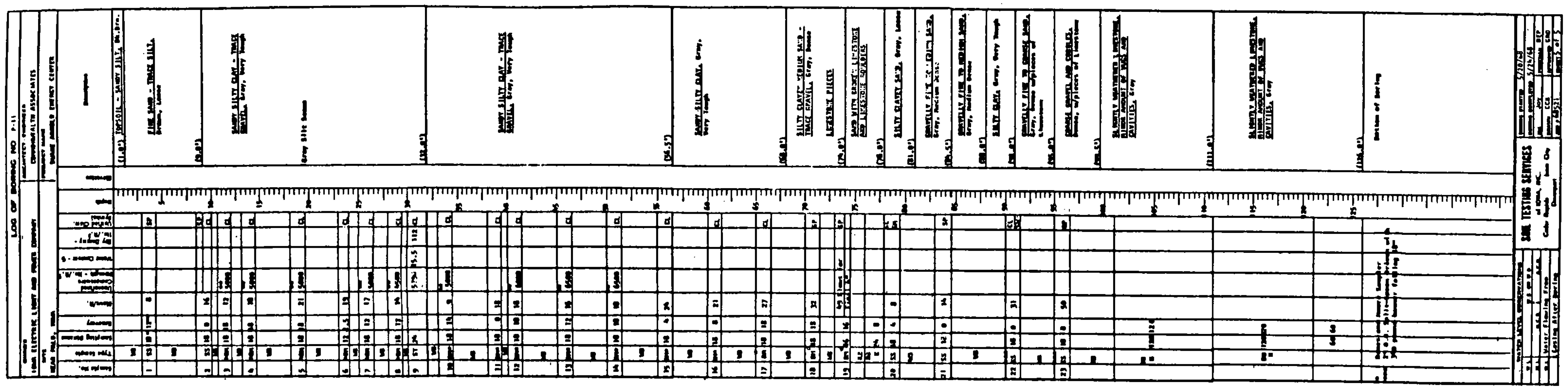
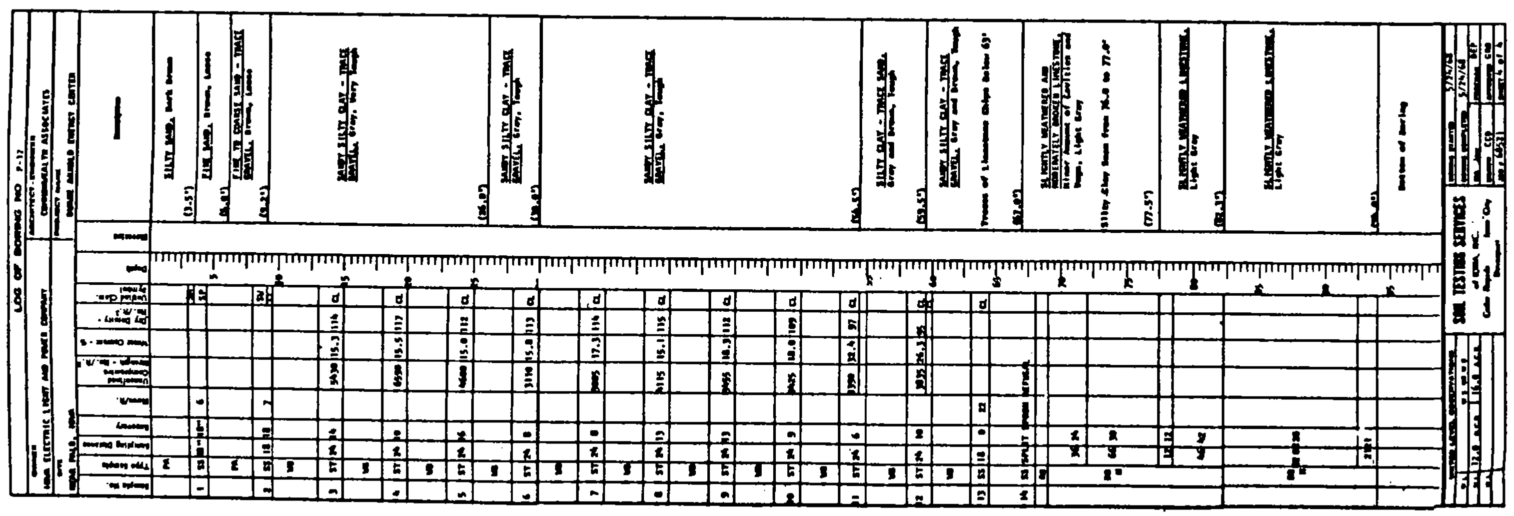
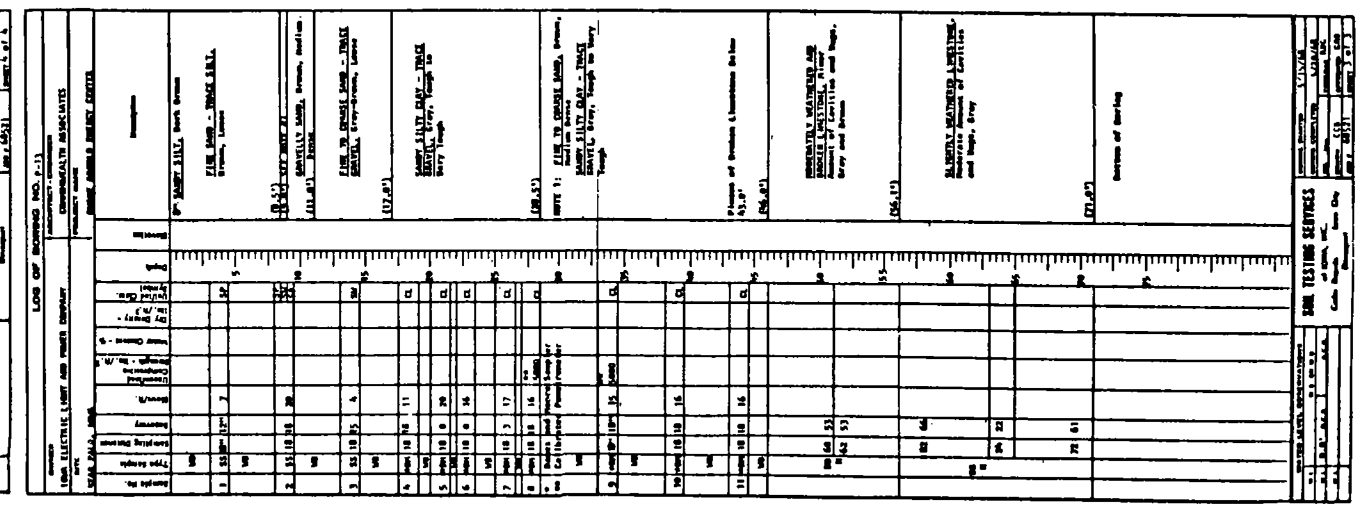
Log of Borings - Numbers P-9 and P-10

Figure 2.5-20







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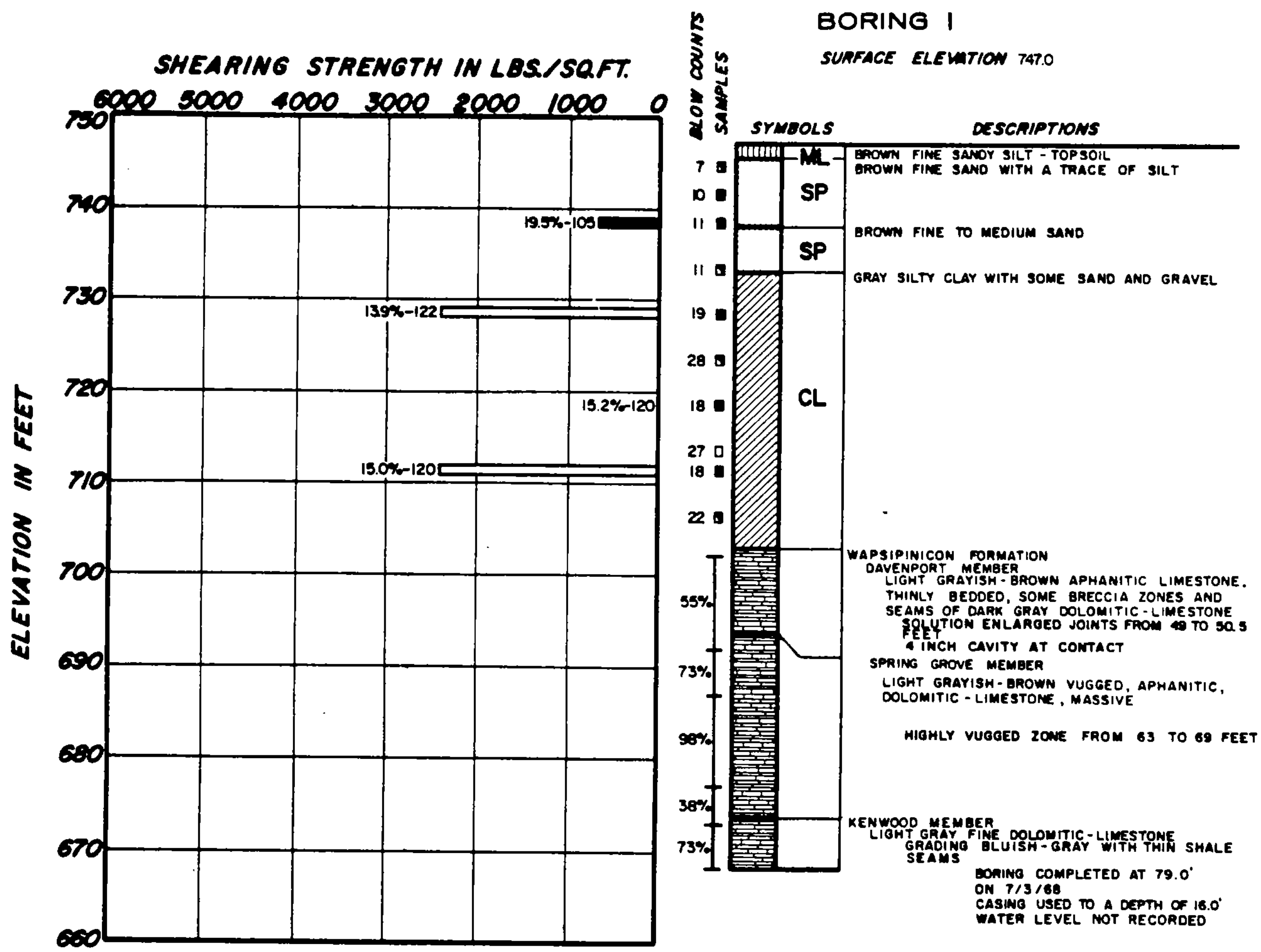
Log of Borings - Numbers P-11 and P-12

Figure 2.5-21



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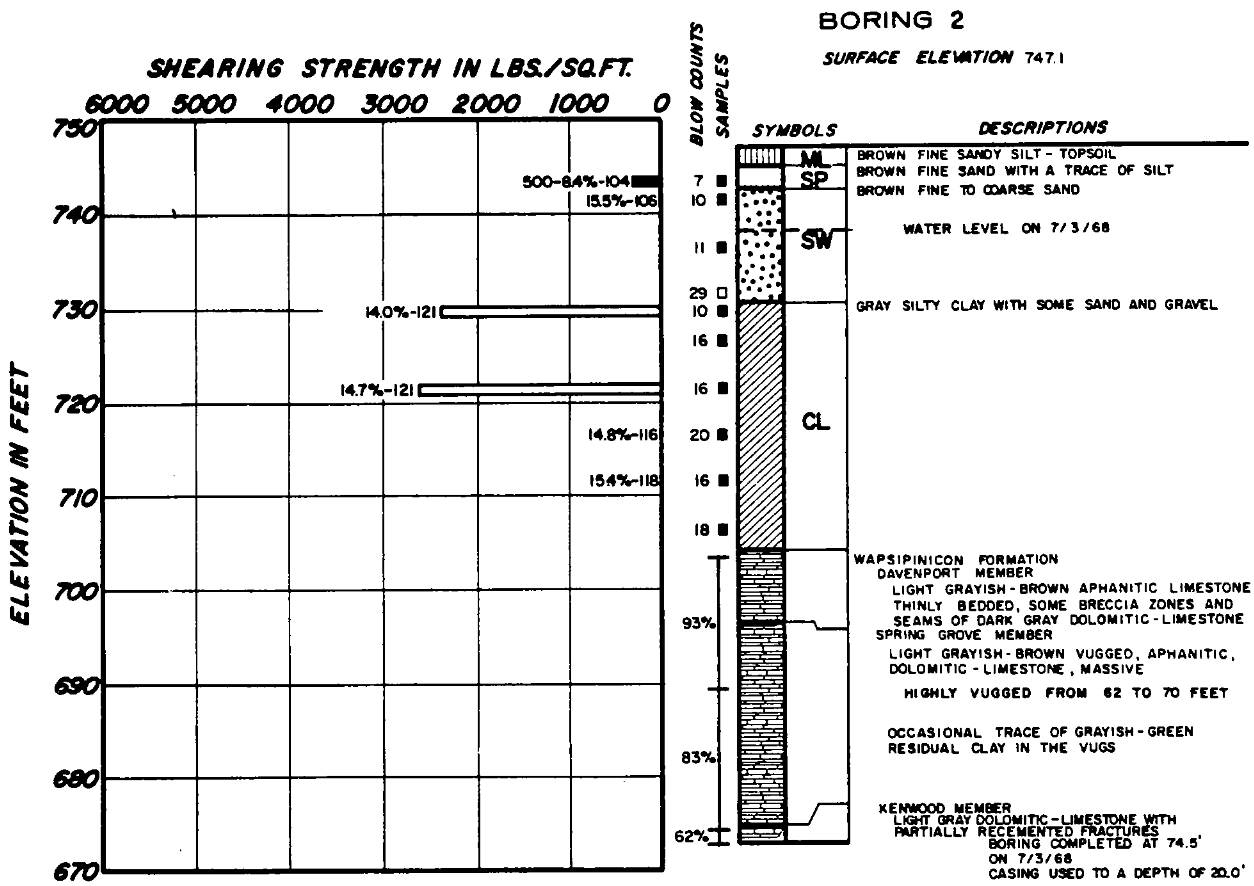
Log of Borings - Boring 1

Figure 2.5-22



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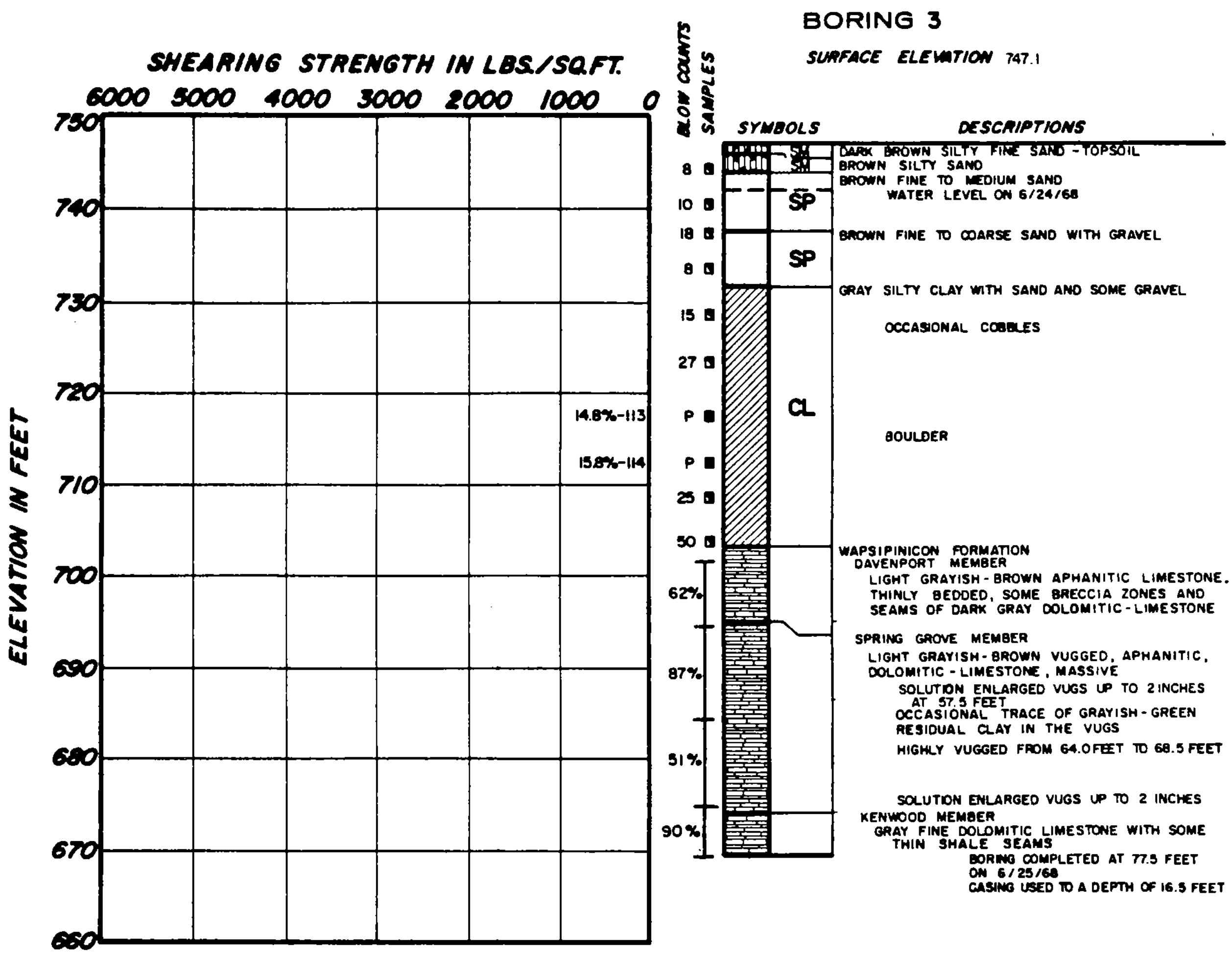
Log of Borings - Boring 2

Figure 2.5-23



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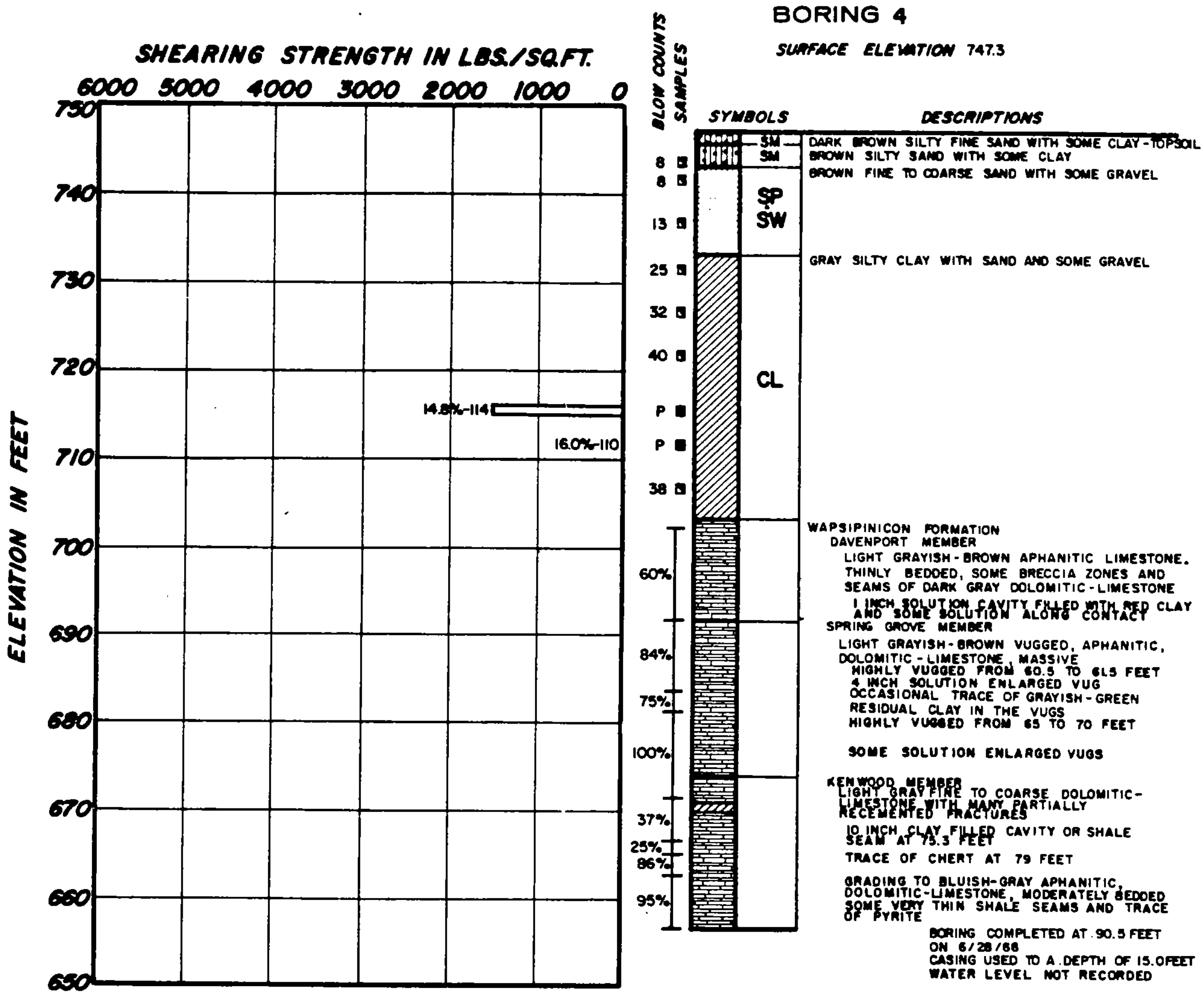
Log of Borings - Boring 3

Figure 2.5-24



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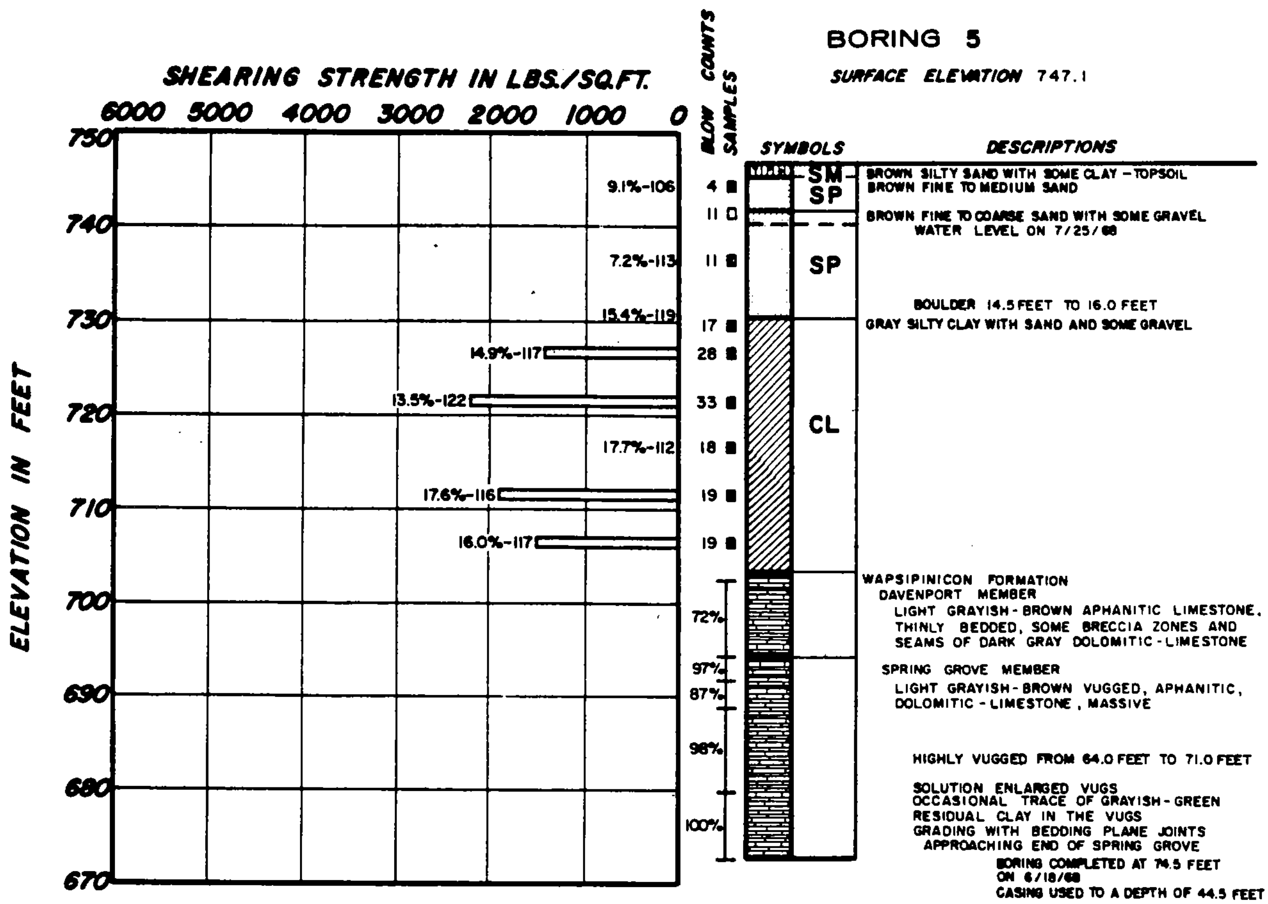
Log of Borings - Boring 4

Figure 2.5-25



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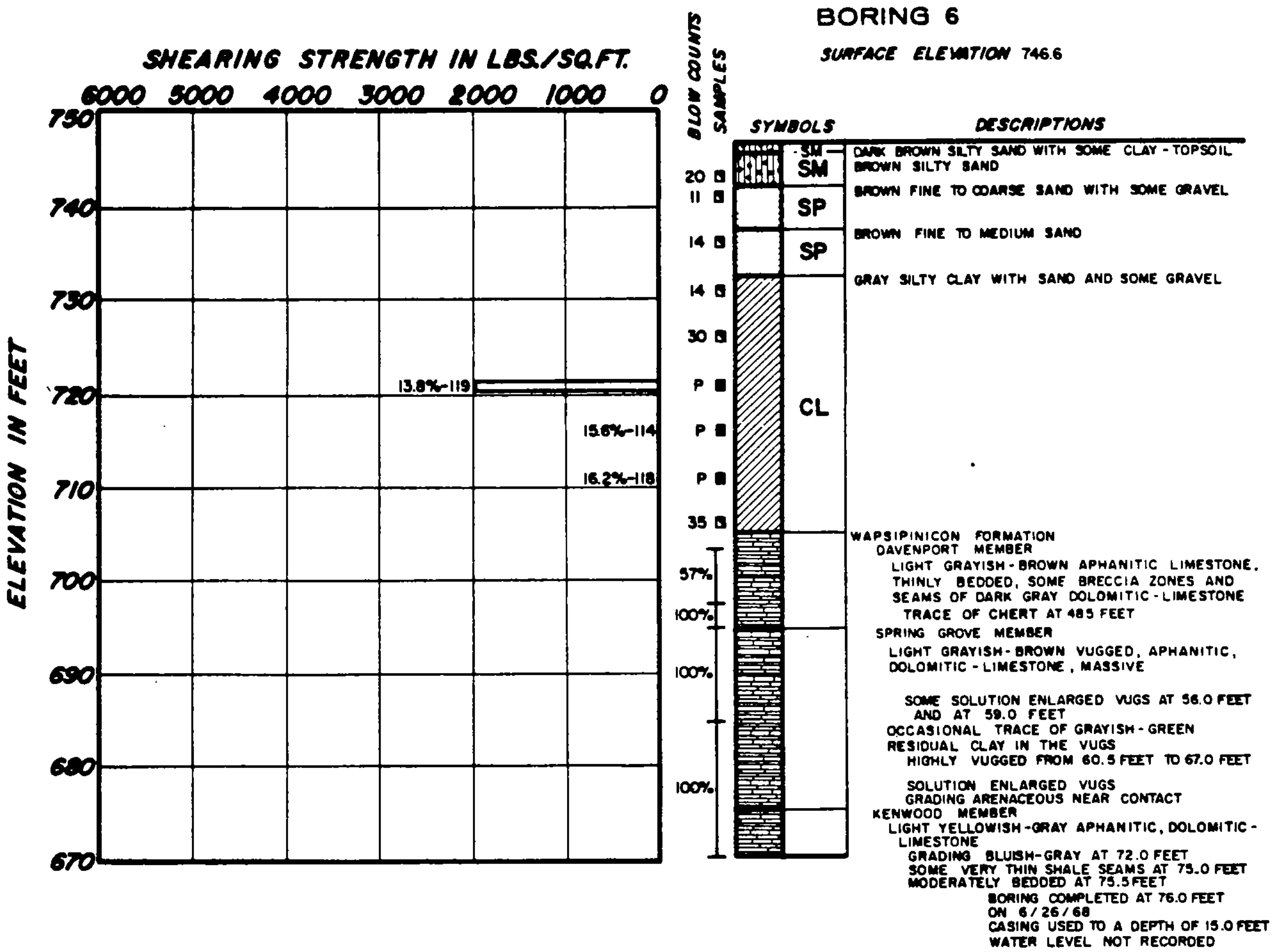
Log of Borings - Boring 5

Figure 2.5-26



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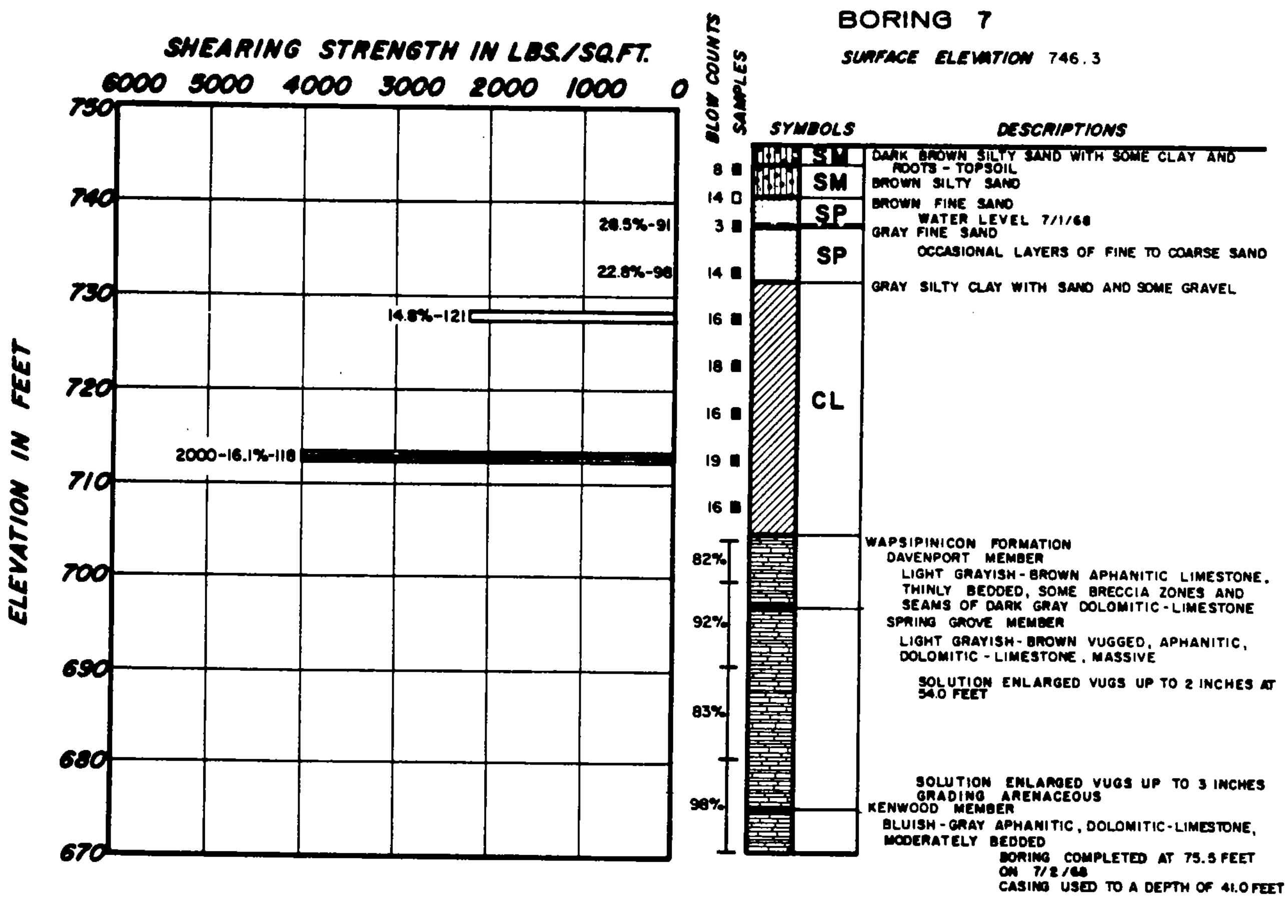
Log of Borings - Boring 6

Figure 2.5-27



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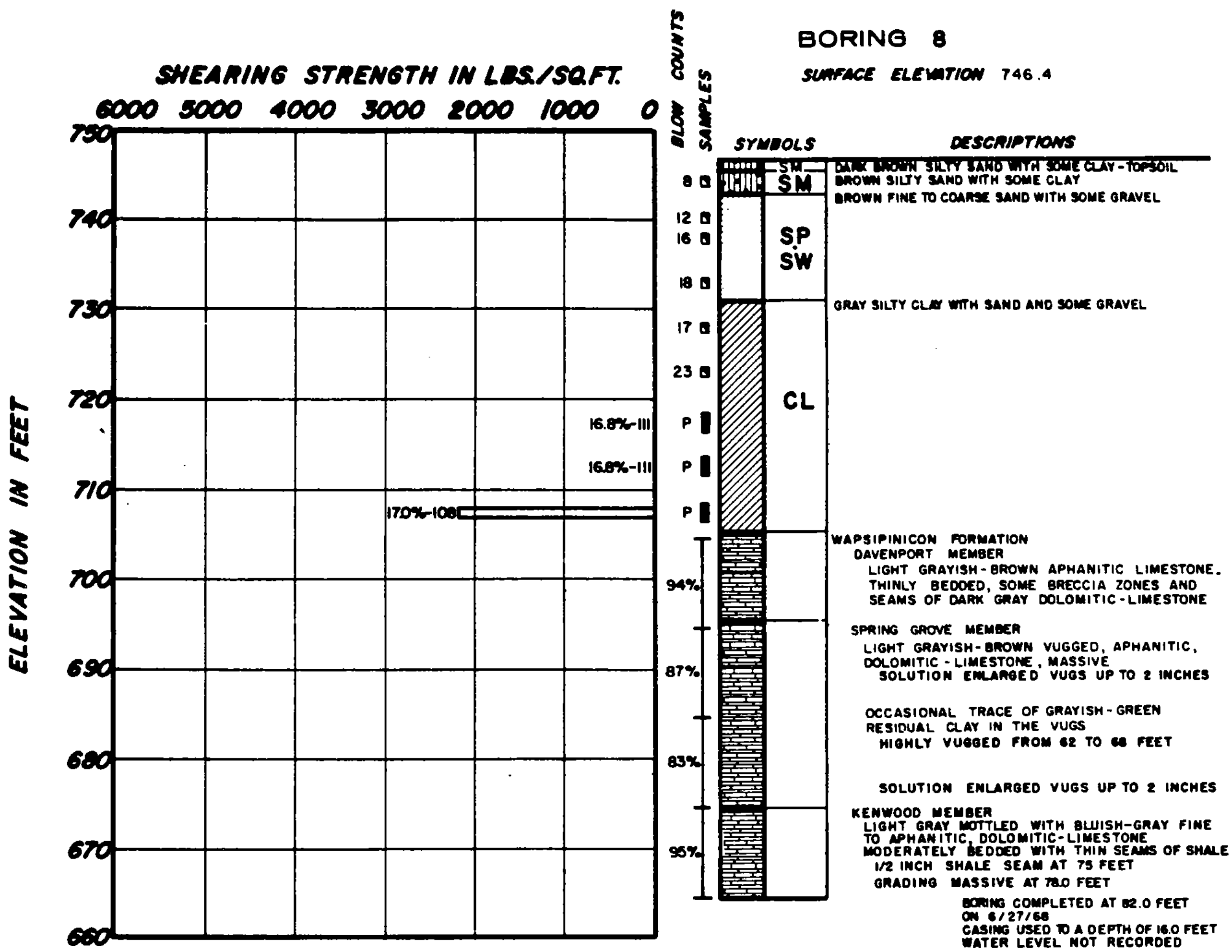
Log of Borings - Boring 7

Figure 2.5-28



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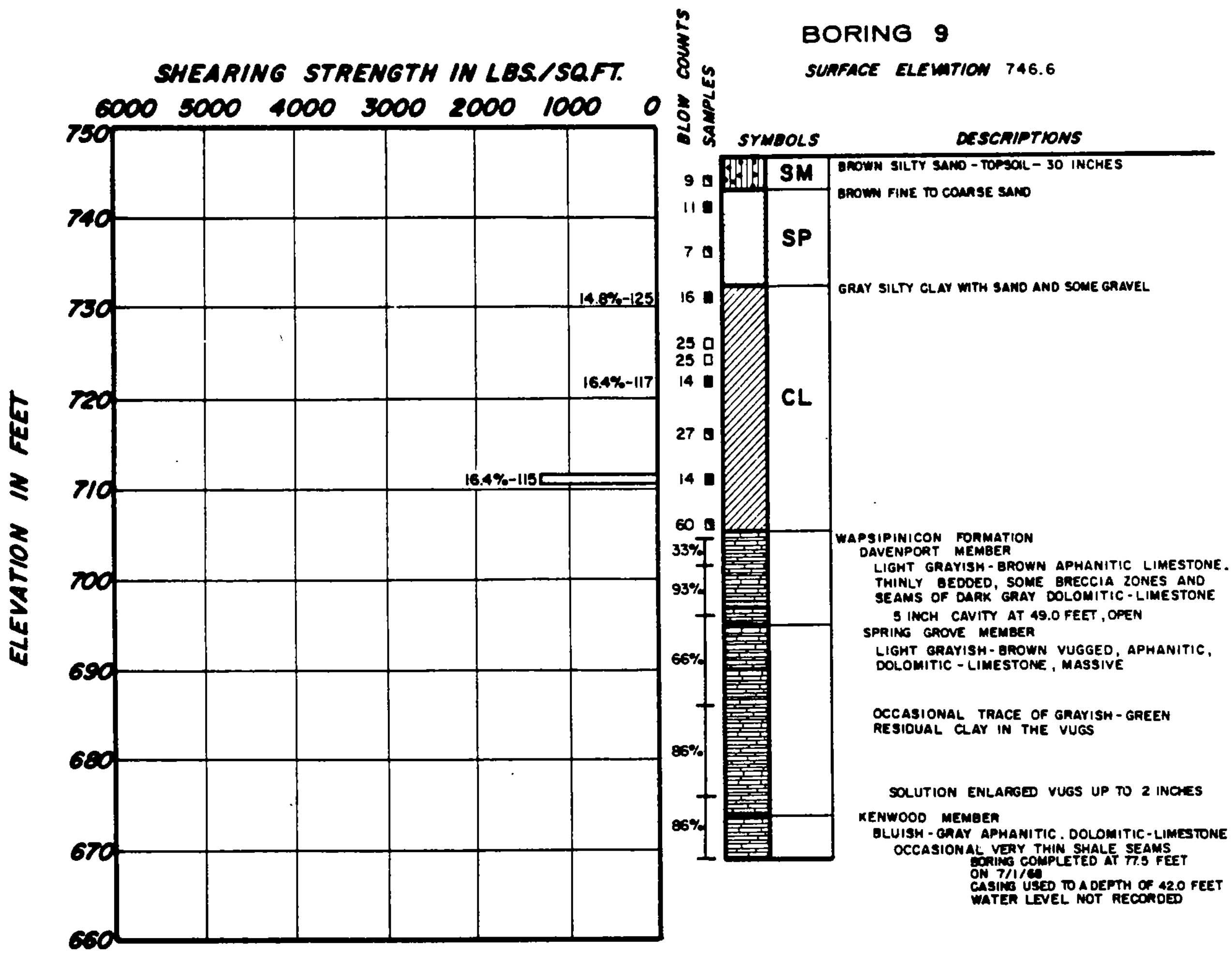
Log of Borings - Boring 8

Figure 2.5-29



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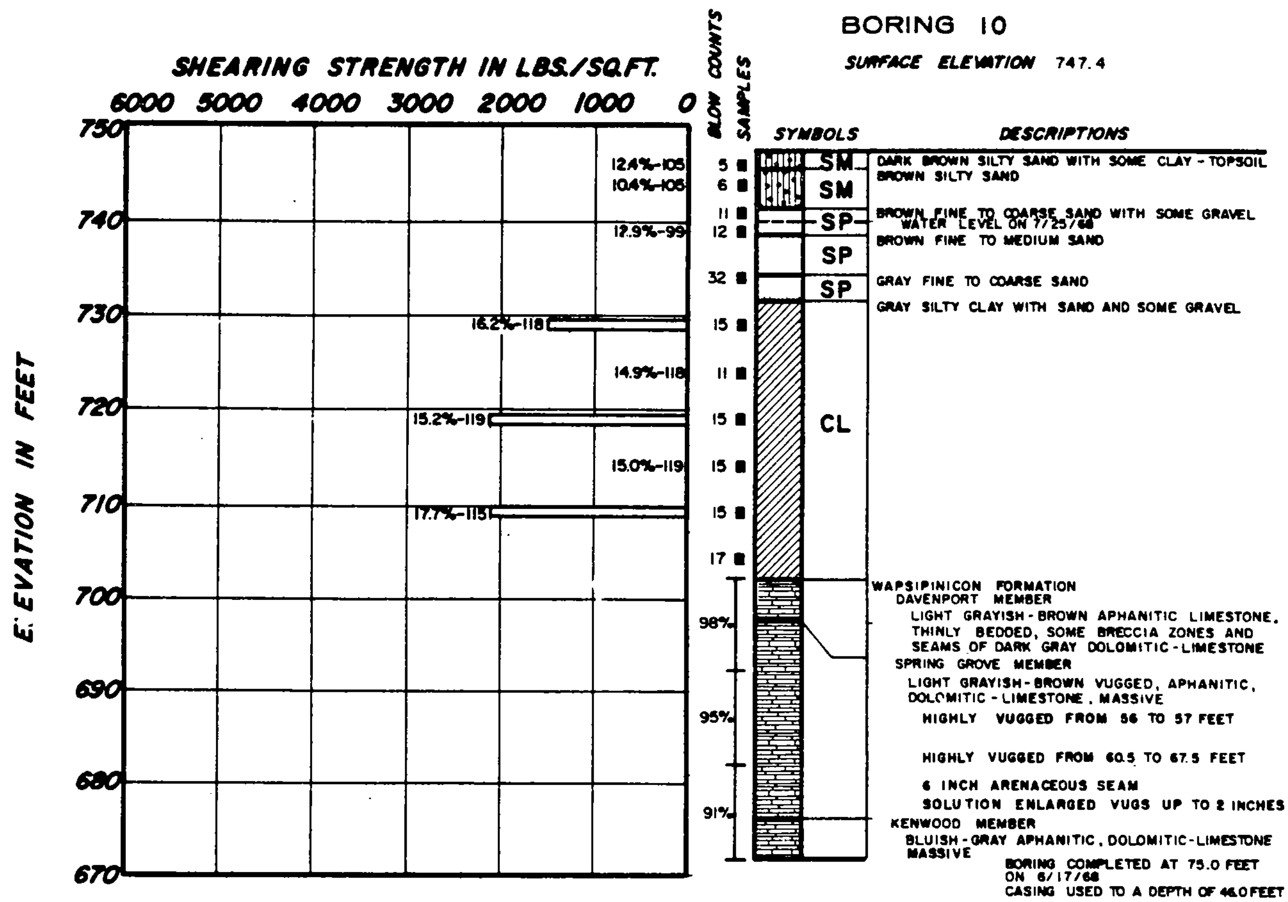
Log of Borings - Boring 9

Figure 2.5-30



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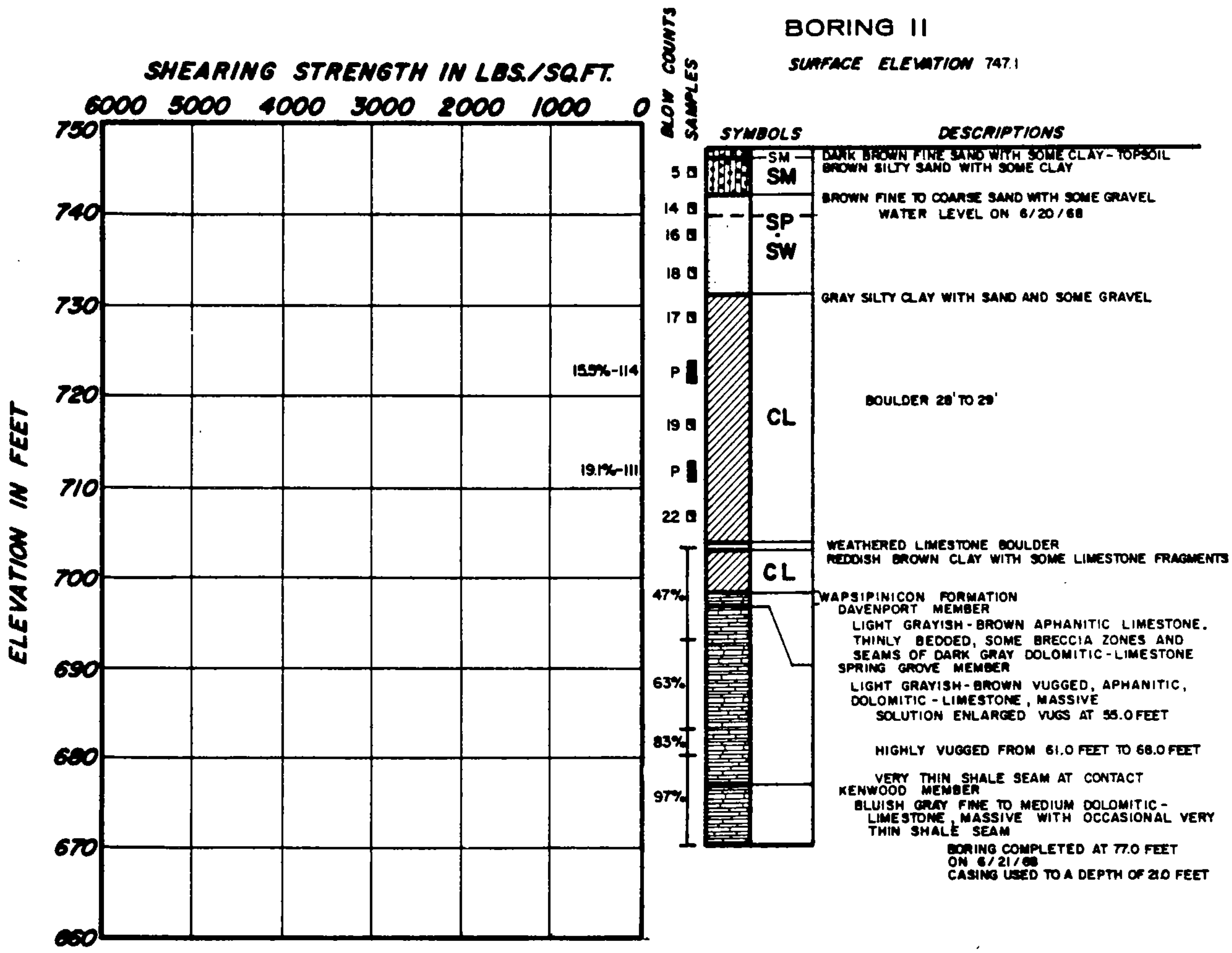
Log of Borings - Boring 10

Figure 2.5-31



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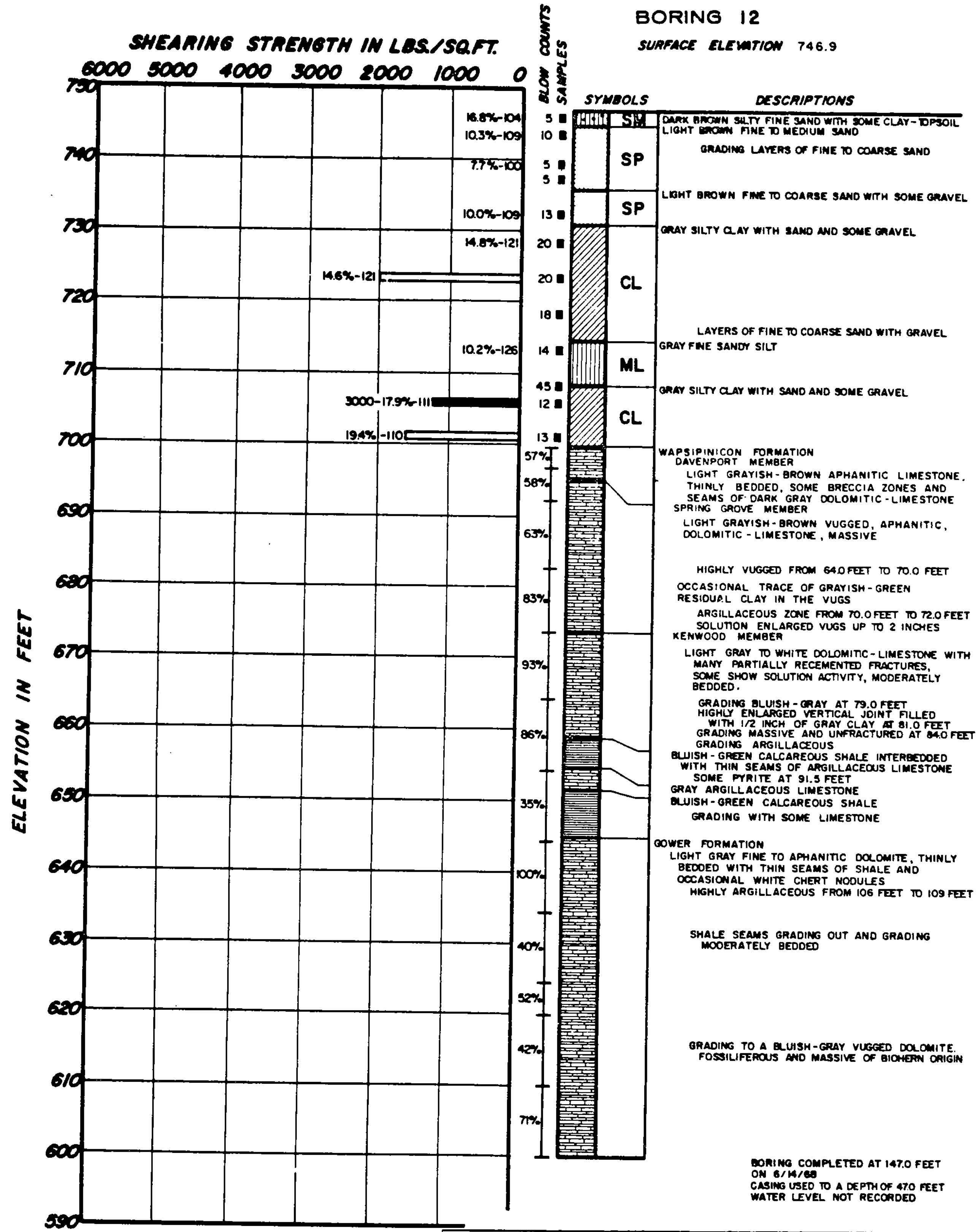
Log of Borings - Boring 11

Figure 2.5-32



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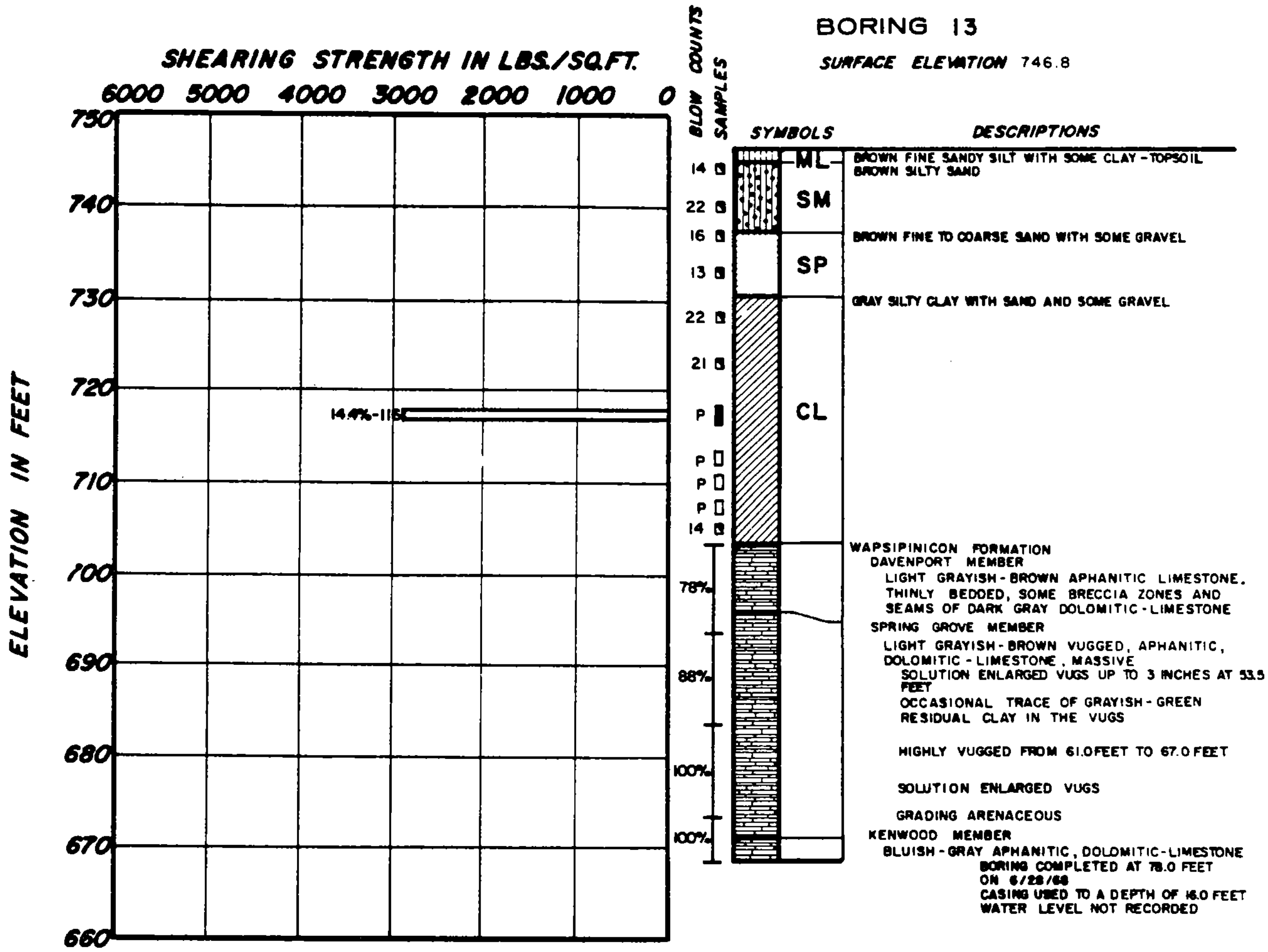
Log of Borings - Boring 12

Figure 2.5-33



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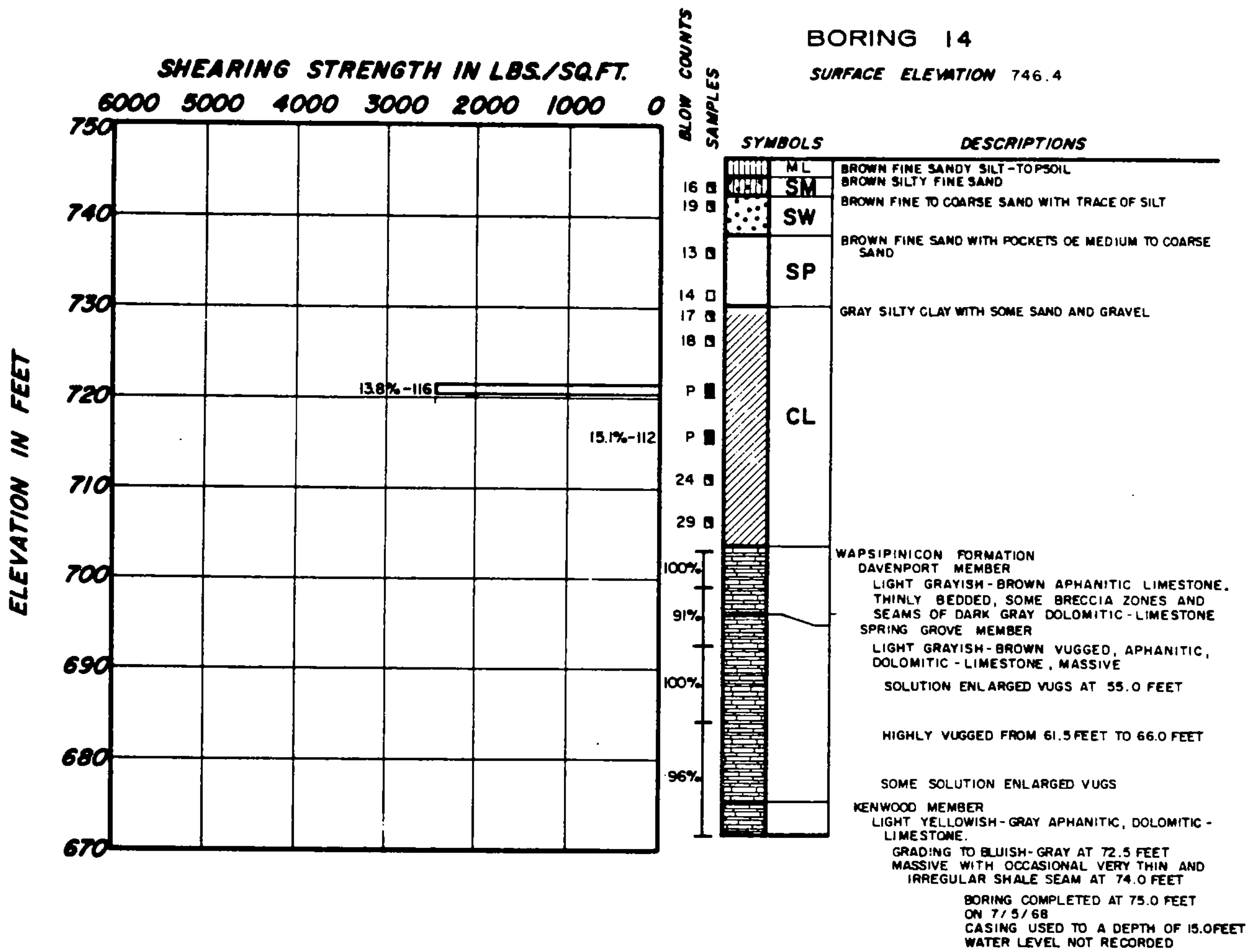
Log of Borings - Boring 13

Figure 2.5-34



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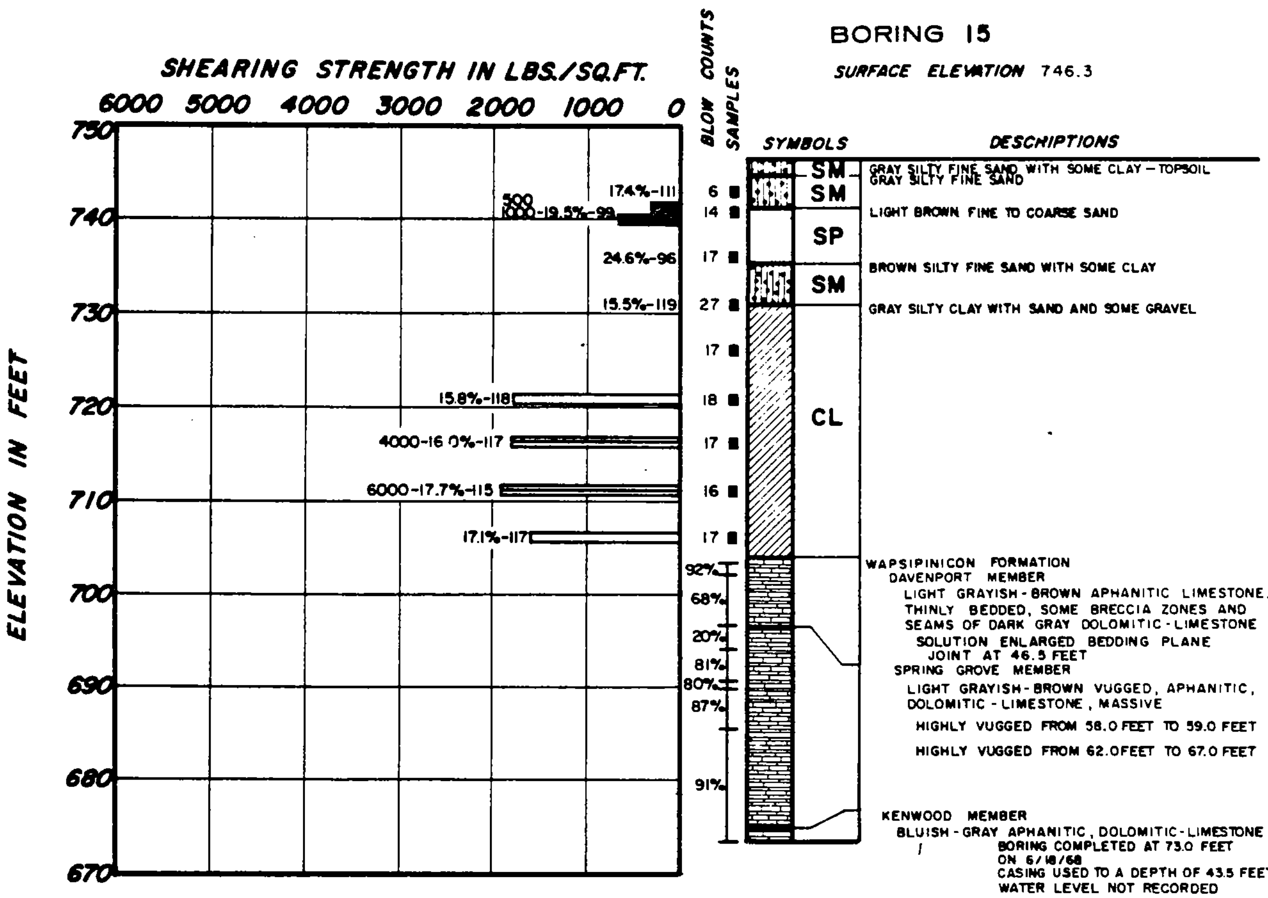
Log of Borings - Boring 14

Figure 2.5-35



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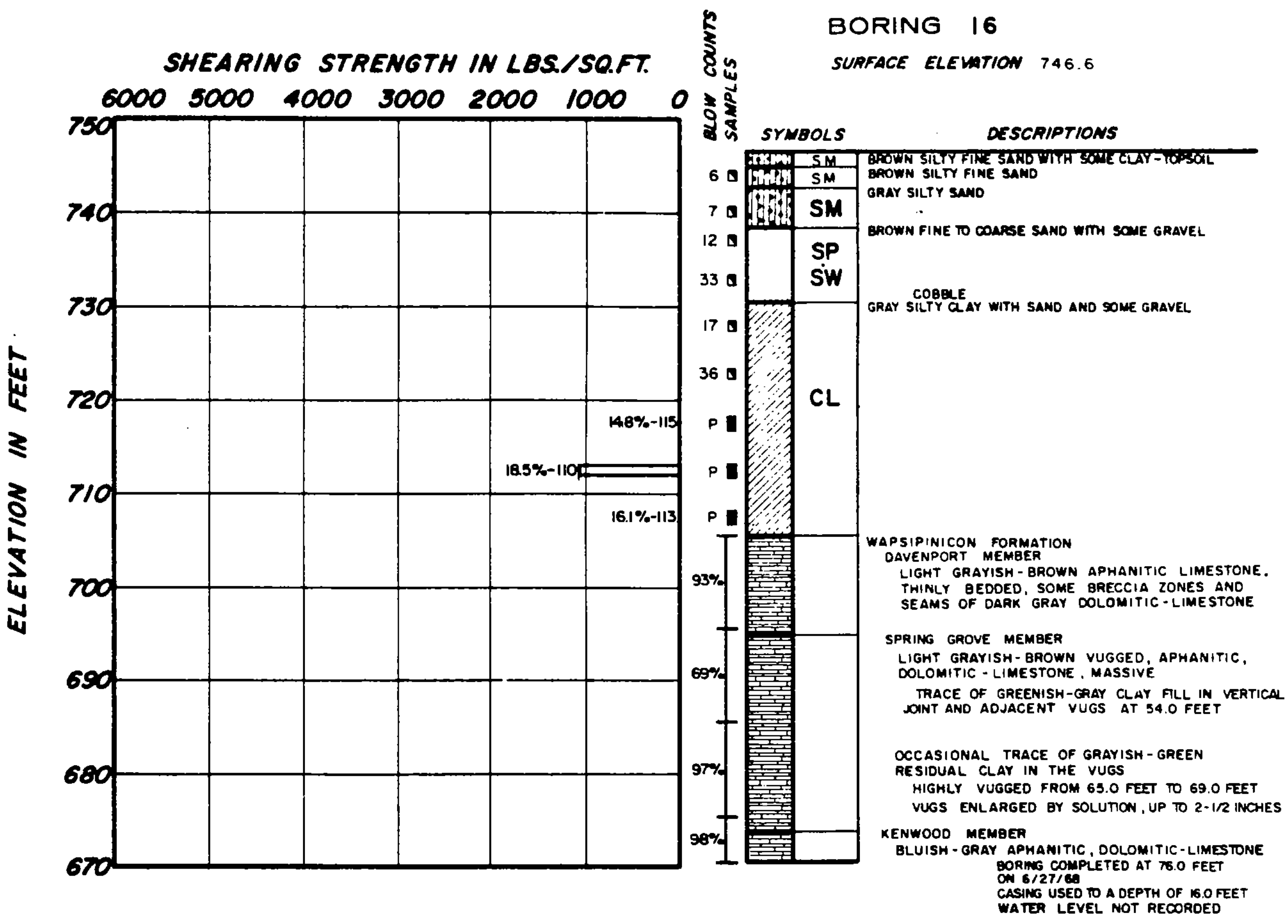
Log of Borings - Boring 15

Figure 2.5-36



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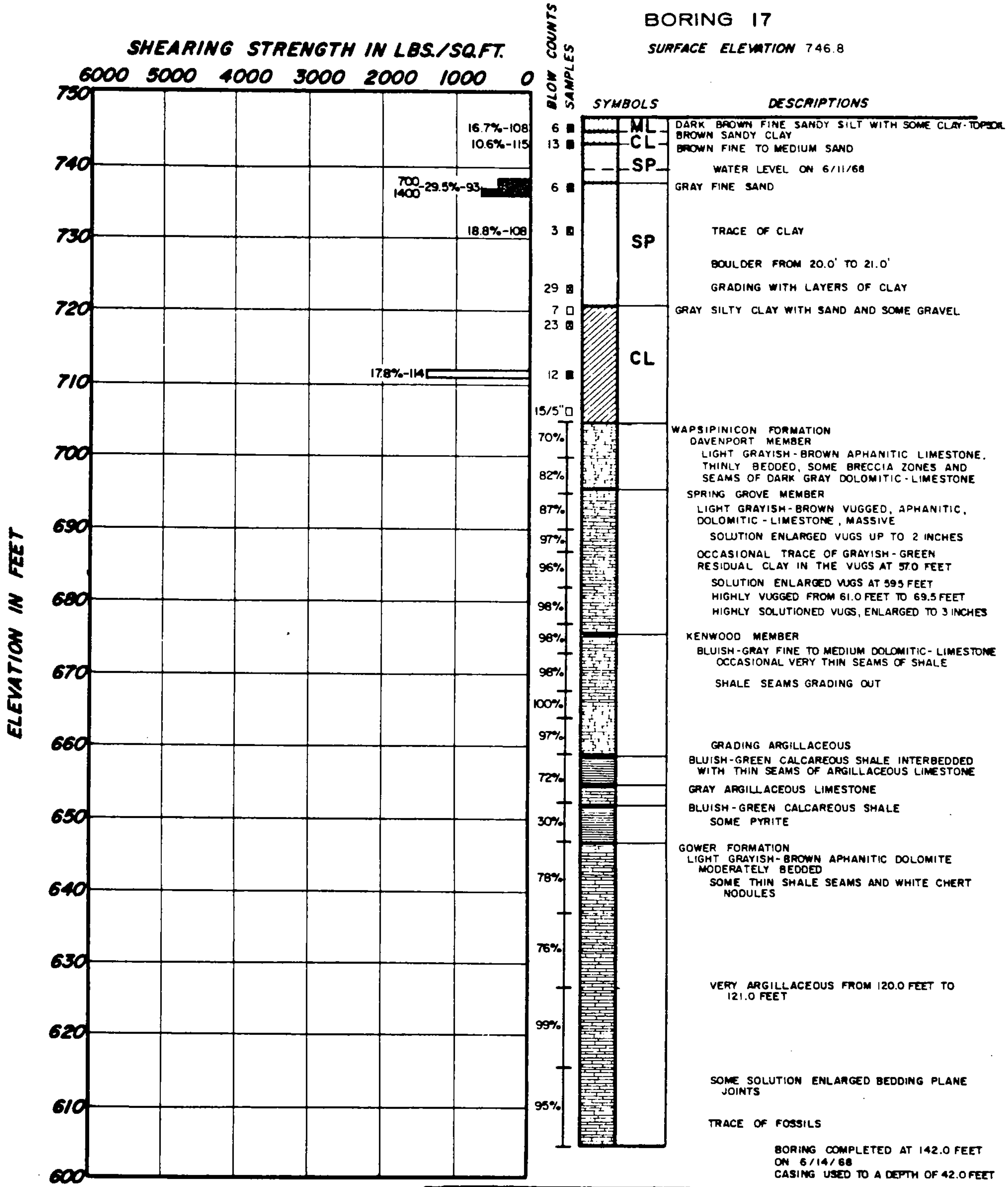
Log of Borings - Boring 16

Figure 2.5-37



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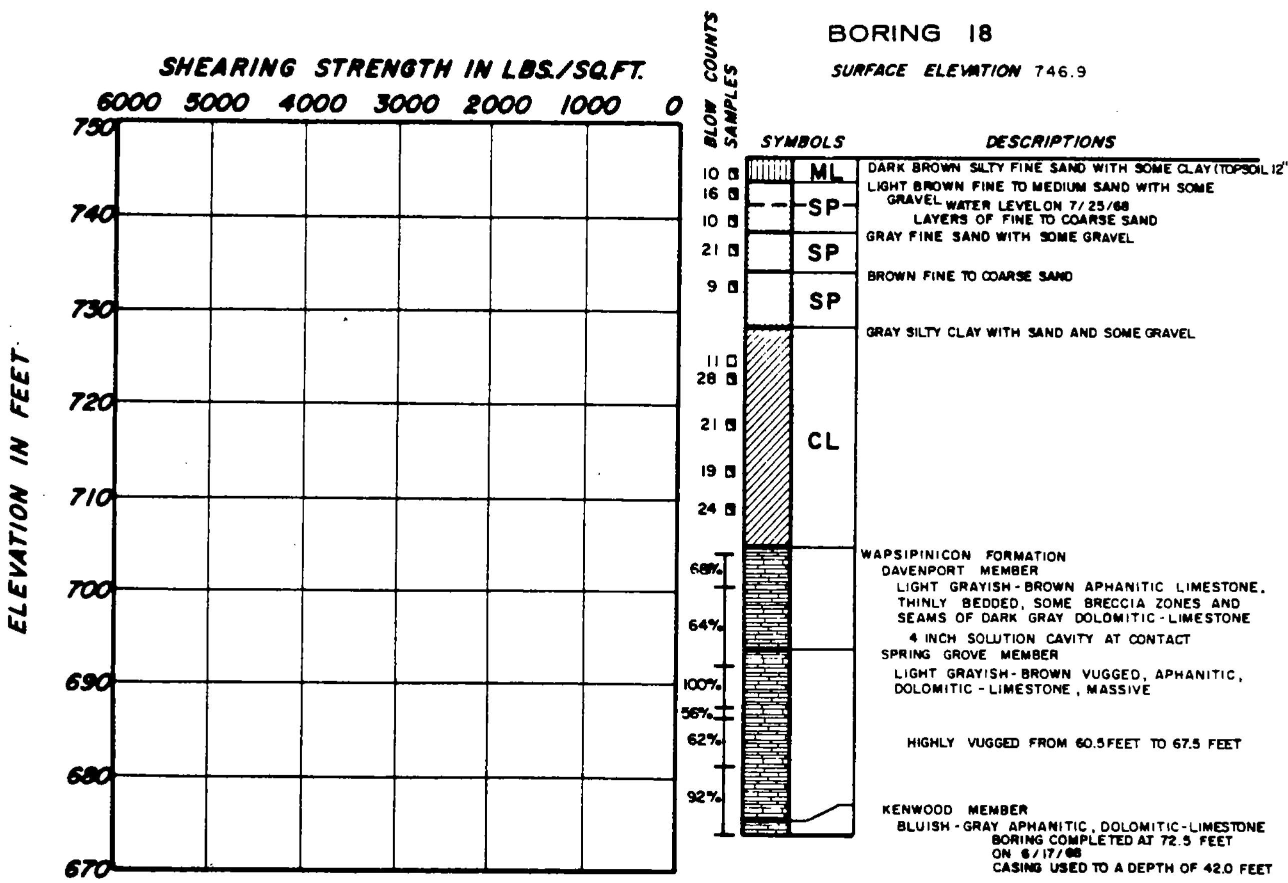
Log of Borings - Boring 17

Figure 2.5-38



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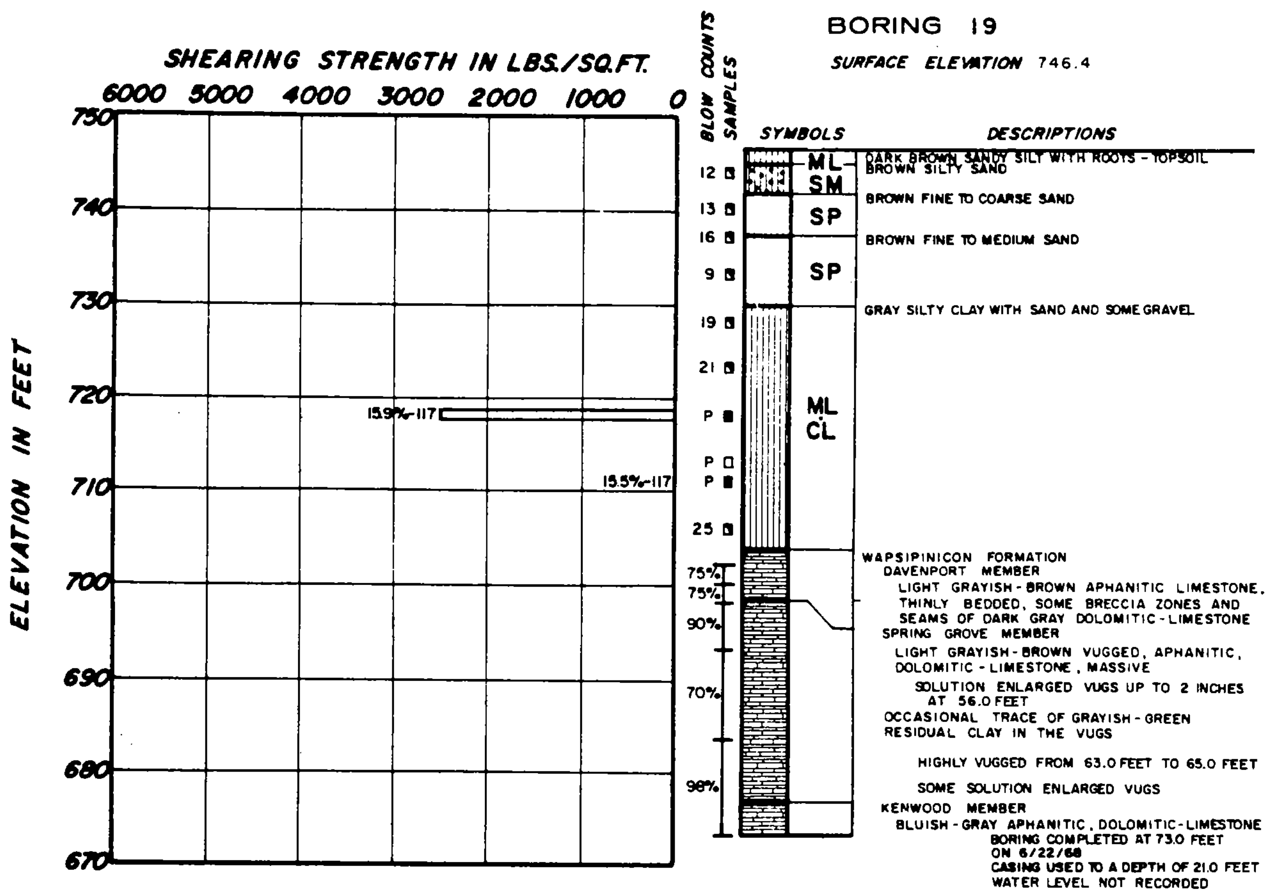
Log of Borings - Boring 18

Figure 2.5-39



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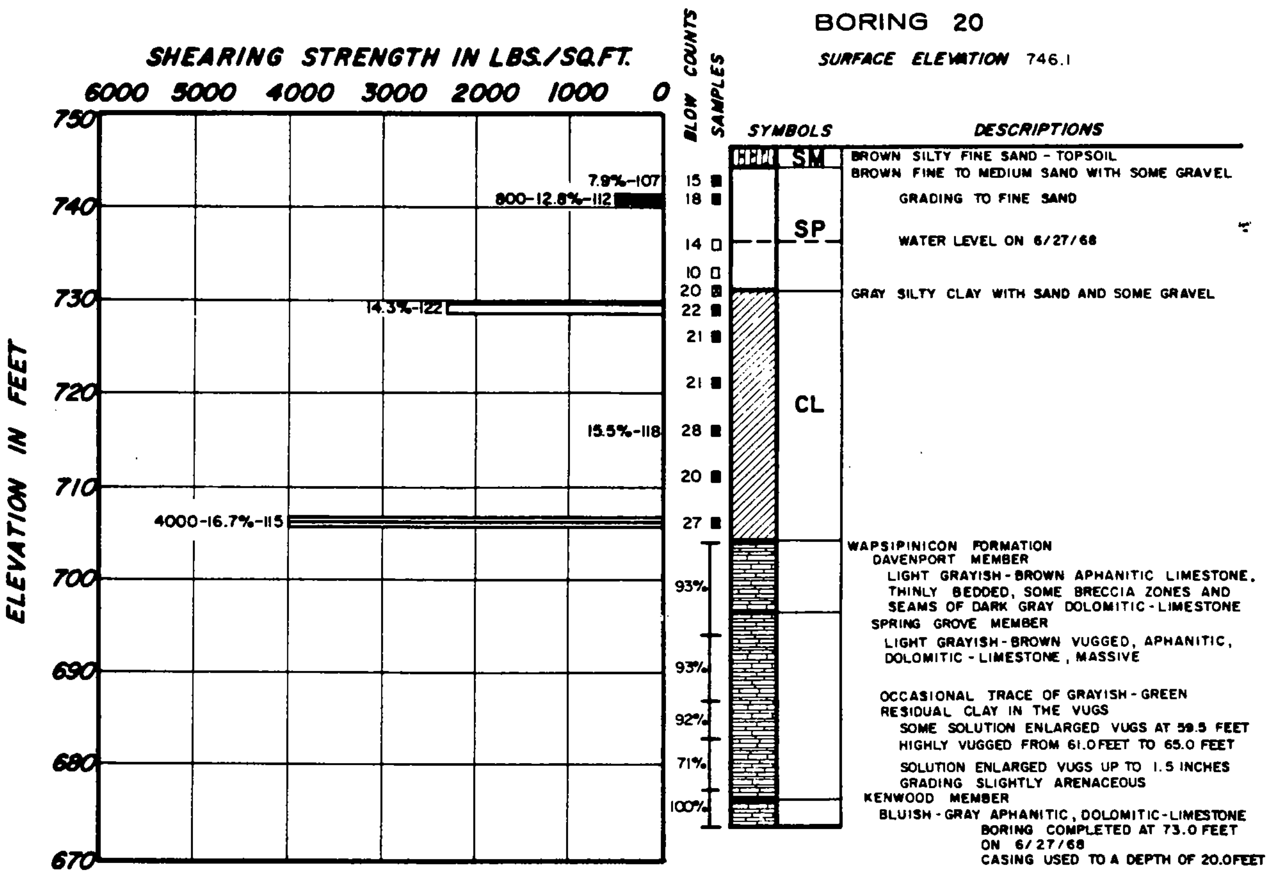
Log of Borings - Boring 19

Figure 2.5-40



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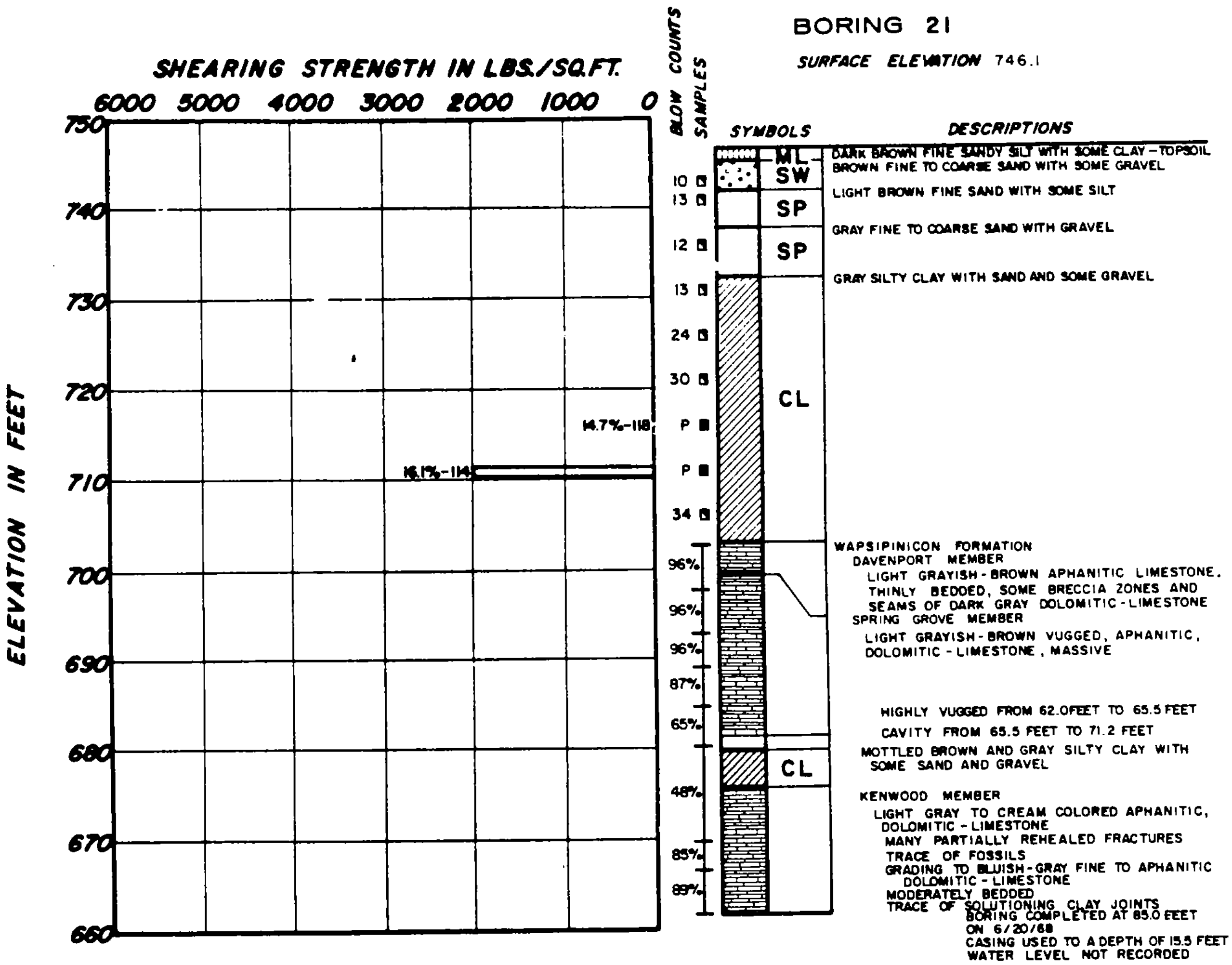
Log of Borings - Boring 20

Figure 2.5-41



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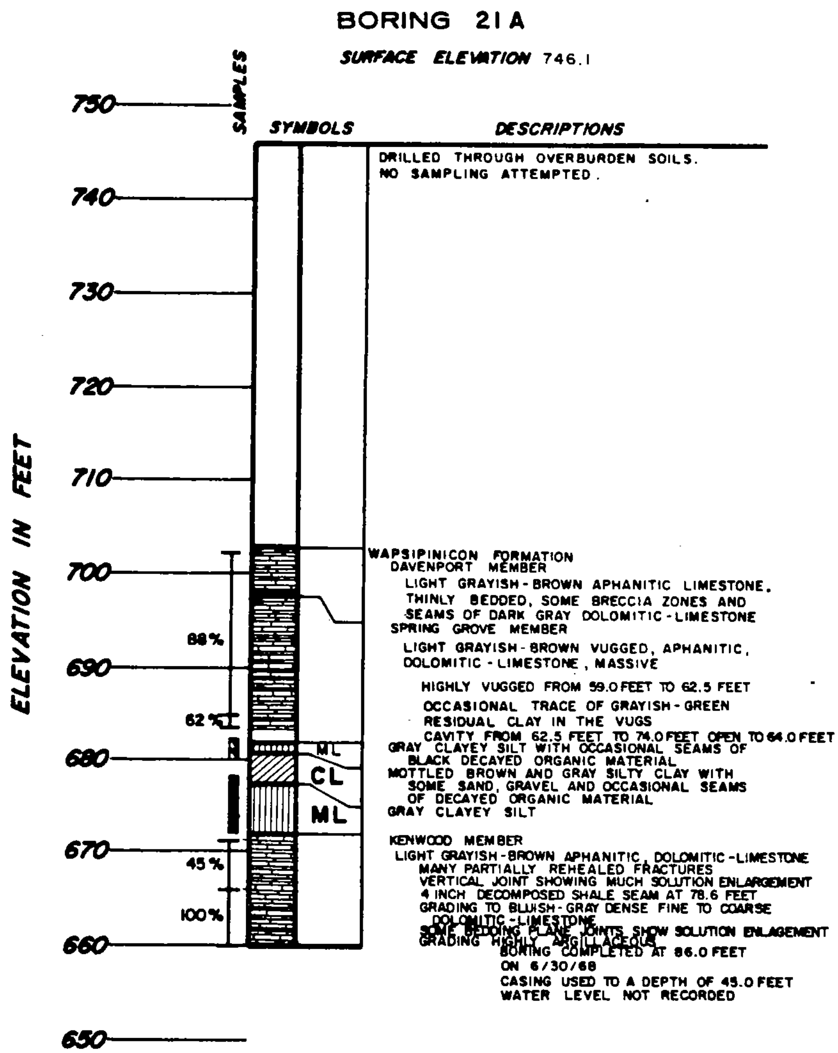
Log of Borings - Boring 21

Figure 2.5-42



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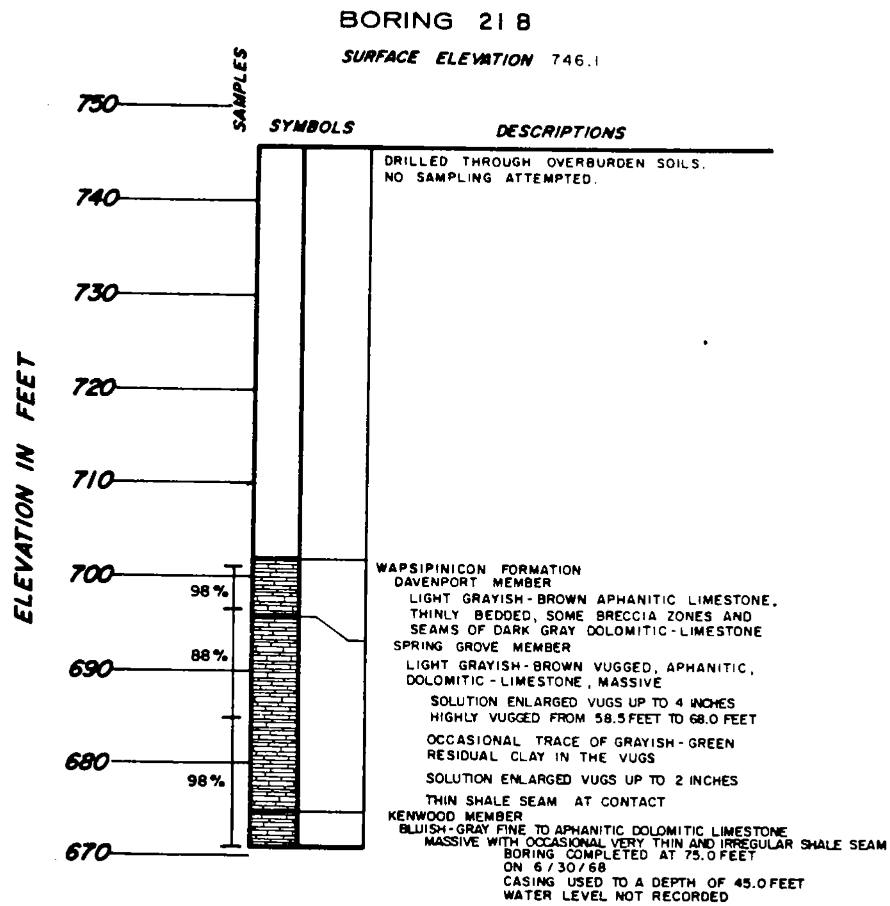
Log of Borings - Boring 21A

Figure 2.5-43



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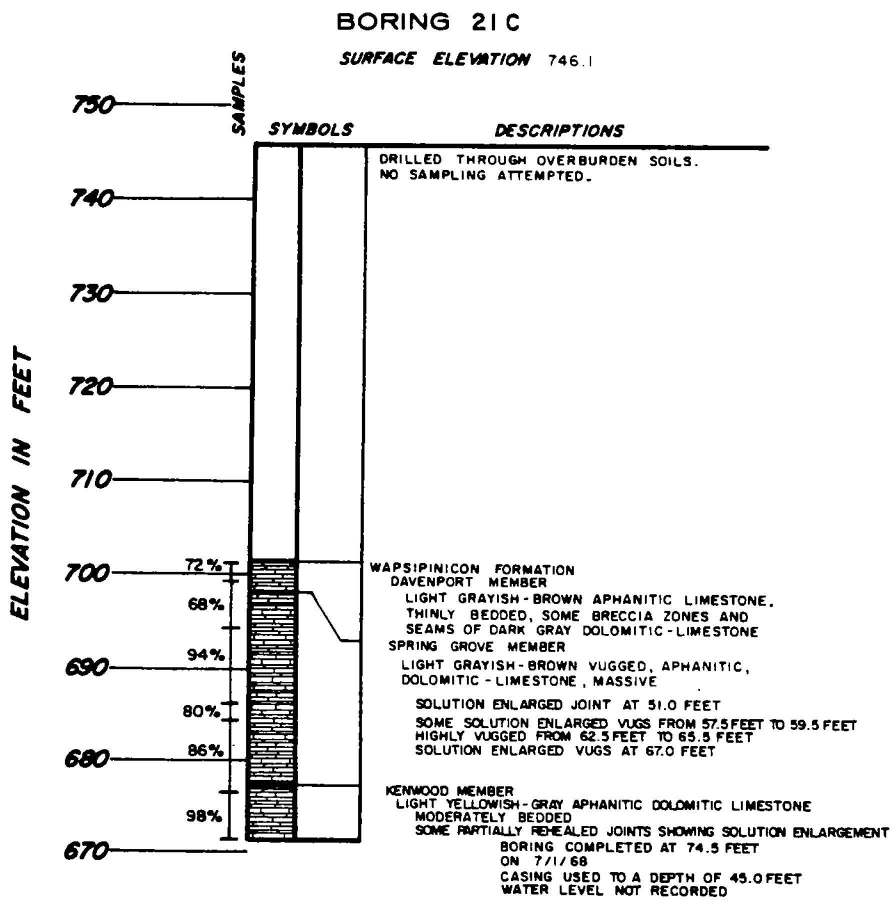
Log of Borings - Boring 21B

Figure 2.5-44



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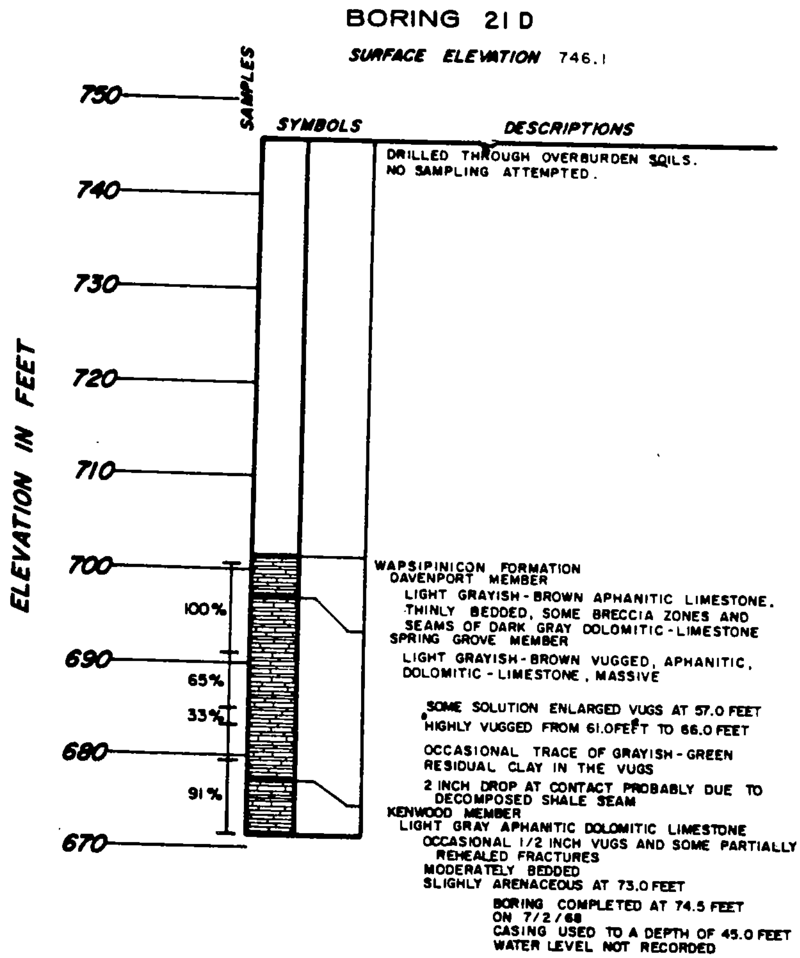
Log of Borings - Boring 21C

Figure 2.5-45



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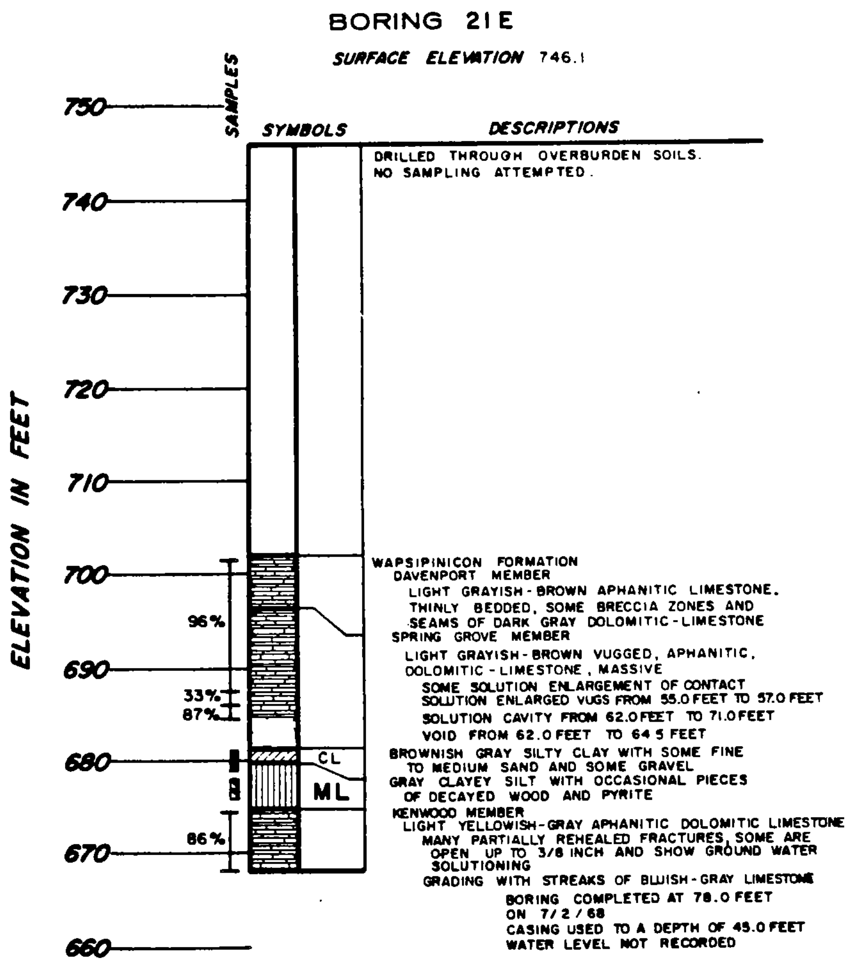
Log of Borings - Boring 21D

Figure 2.5-46



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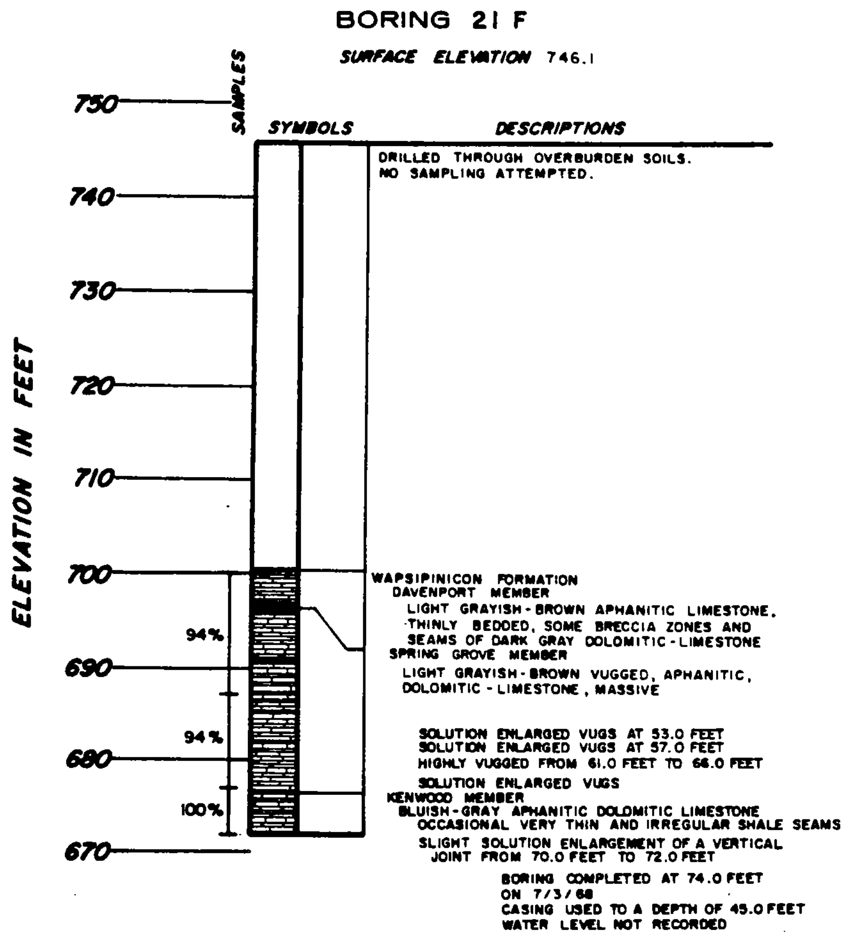
Log of Borings - Boring 21E

Figure 2.5-47



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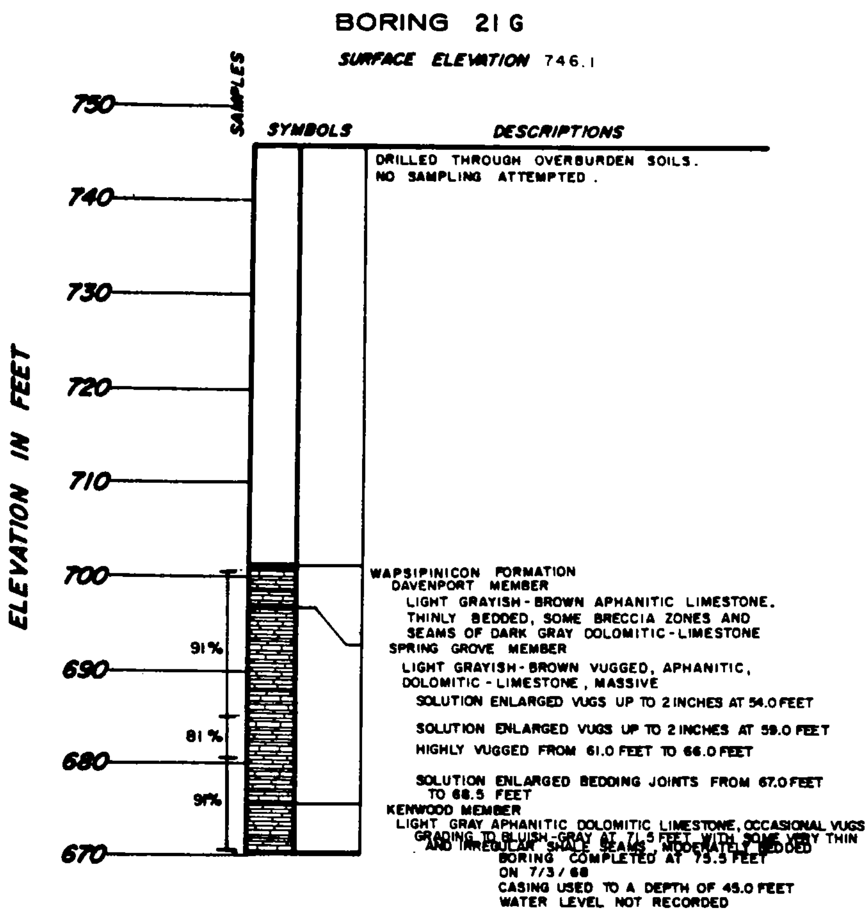
Log of Borings - Boring 21F

Figure 2.5-48



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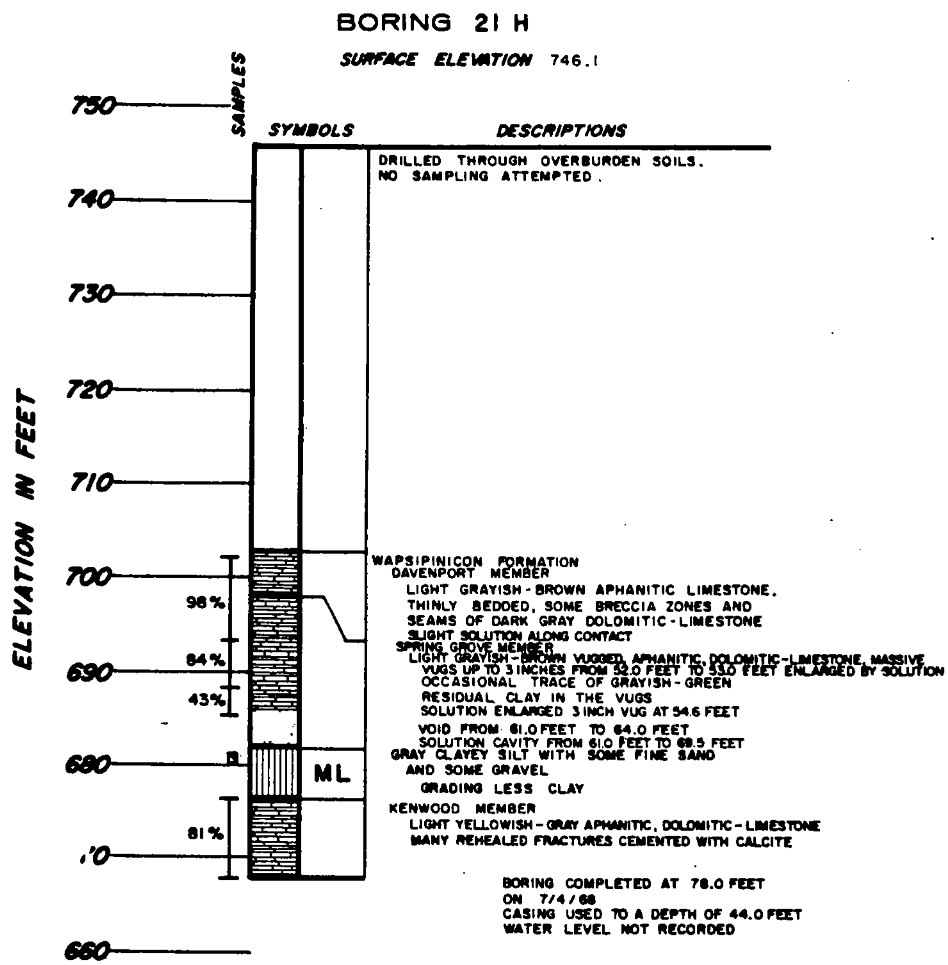
Log of Borings - Boring 21G

Figure 2.5-49



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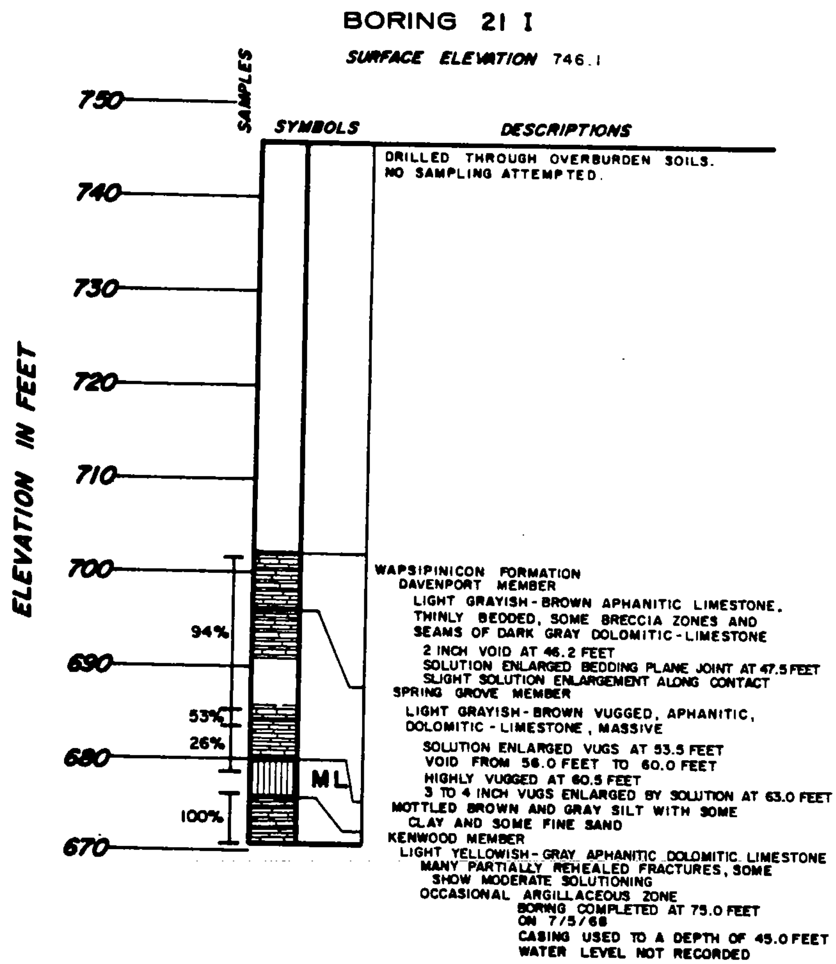
Log of Borings - Boring 21H

Figure 2.5-50



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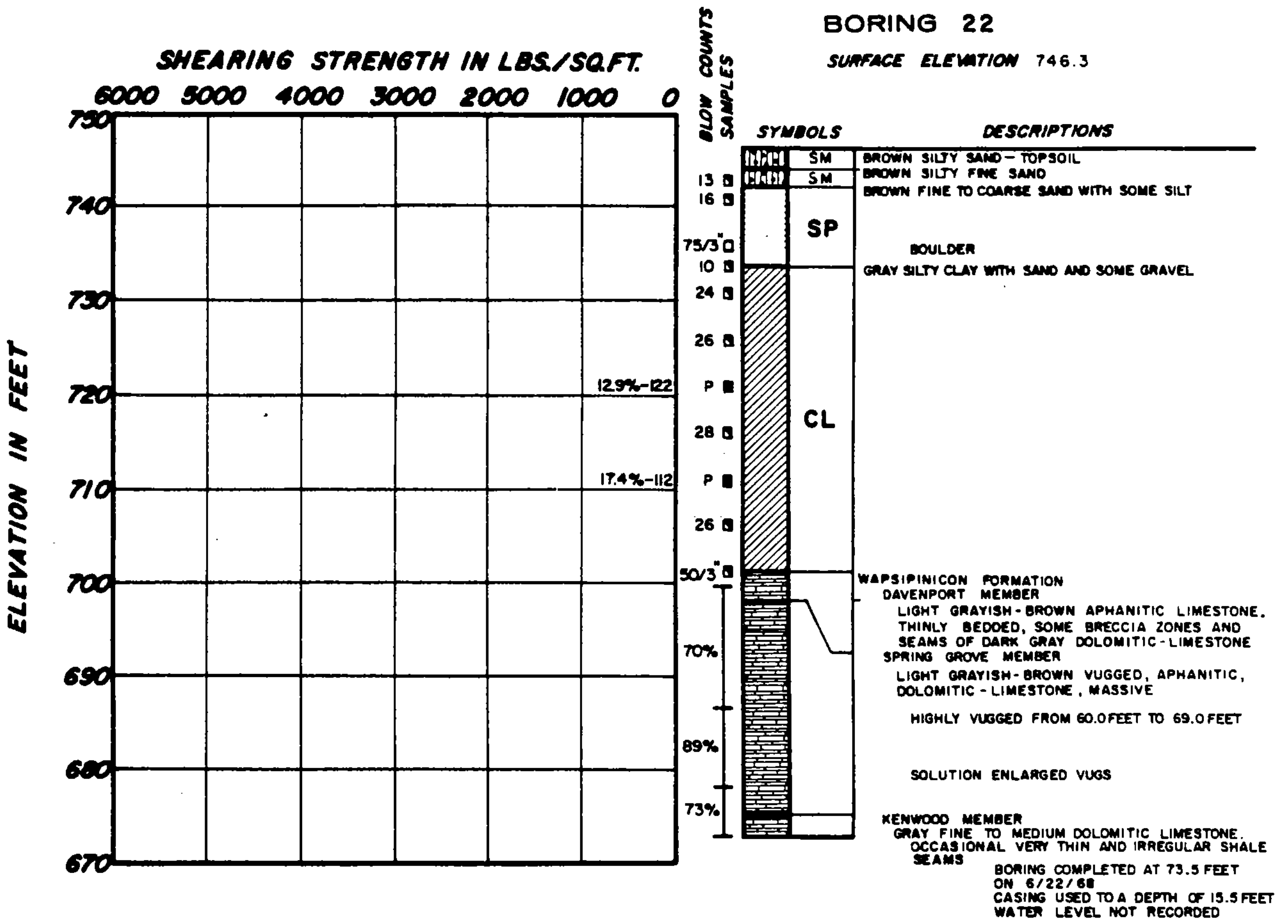
Log of Borings - Boring 21I

Figure 2.5-51



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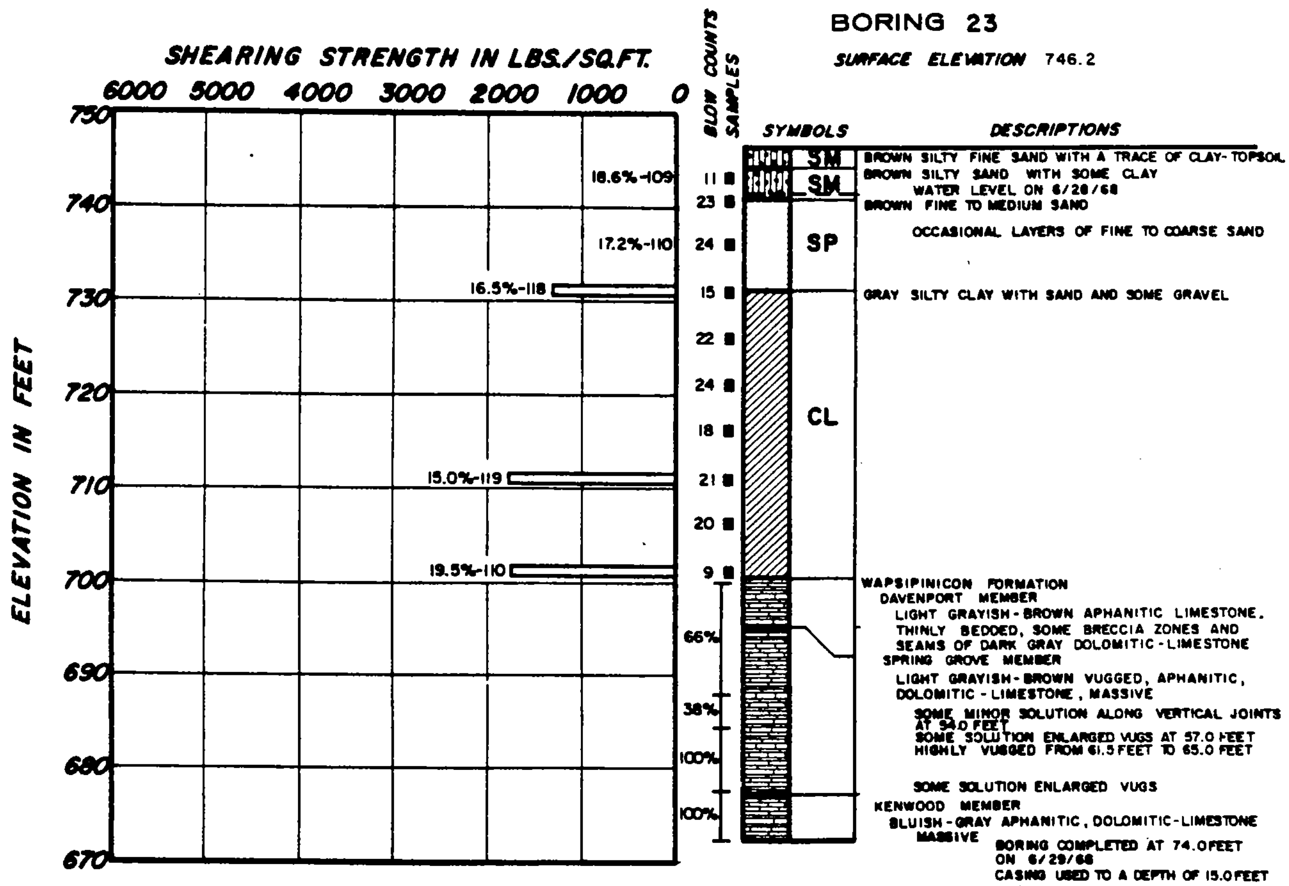
Log of Borings - Boring 22

Figure 2.5-52



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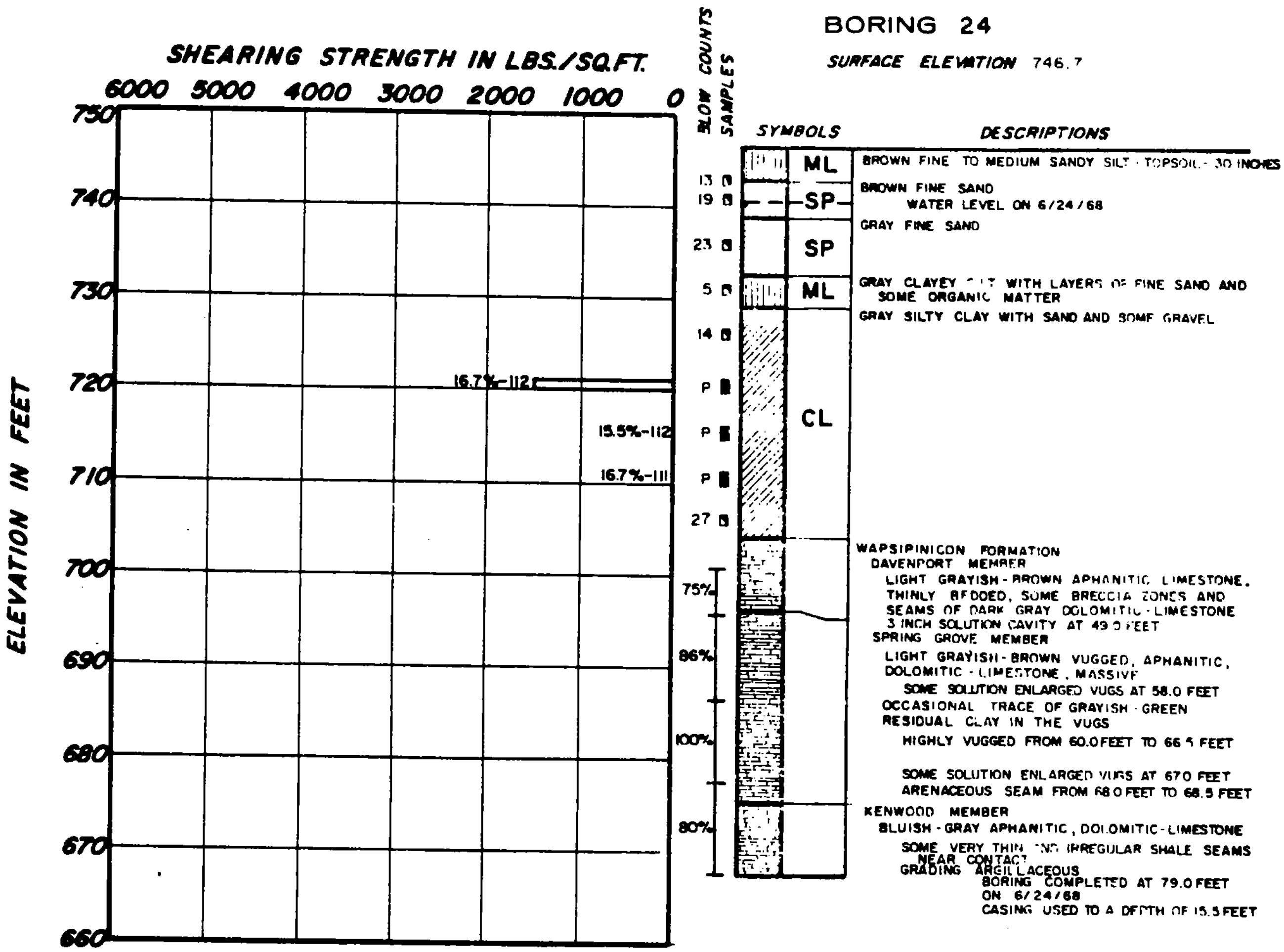
Log of Borings - Boring 23

Figure 2.5-53



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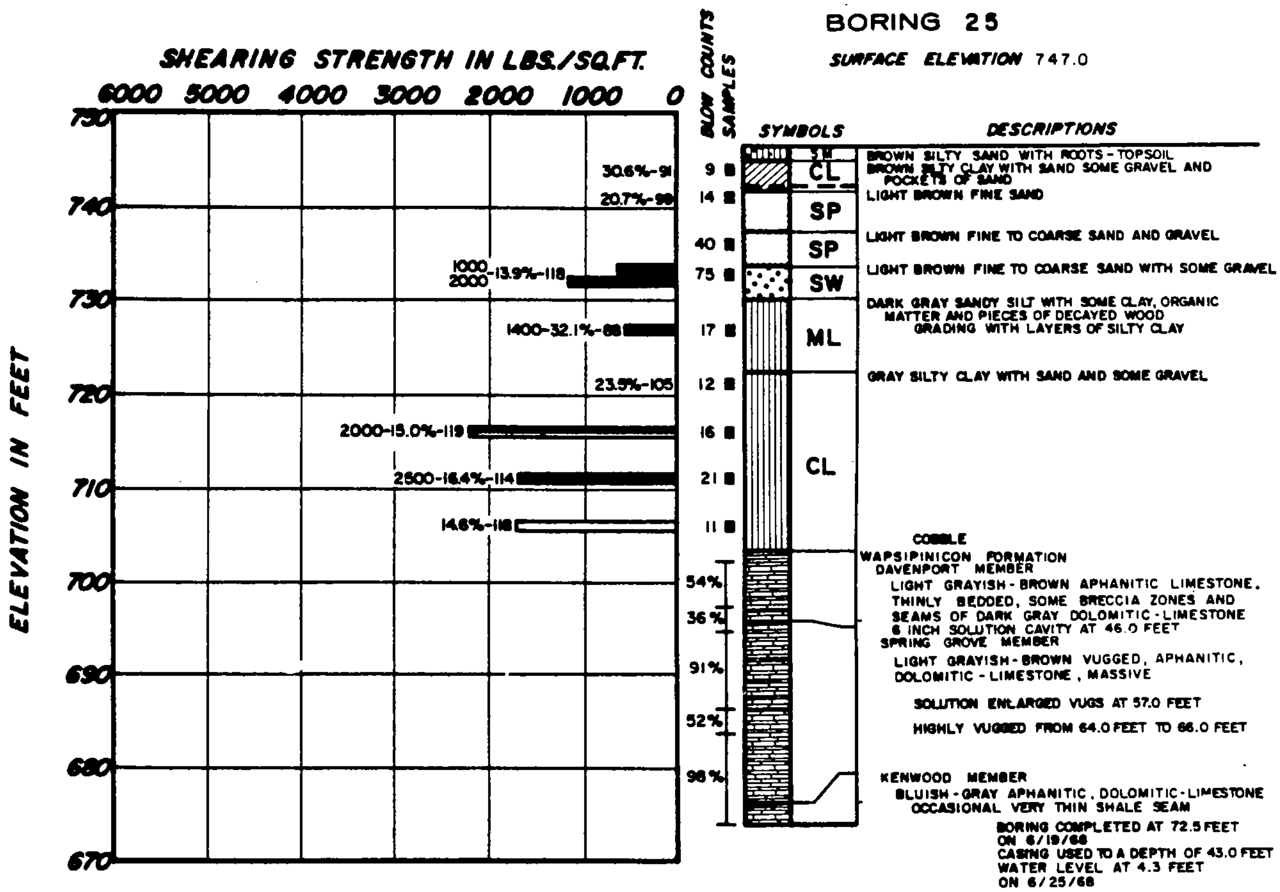
Log of Borings - Boring 24

Figure 2.5-54



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Log of Borings - Boring 25

Figure 2.5-55

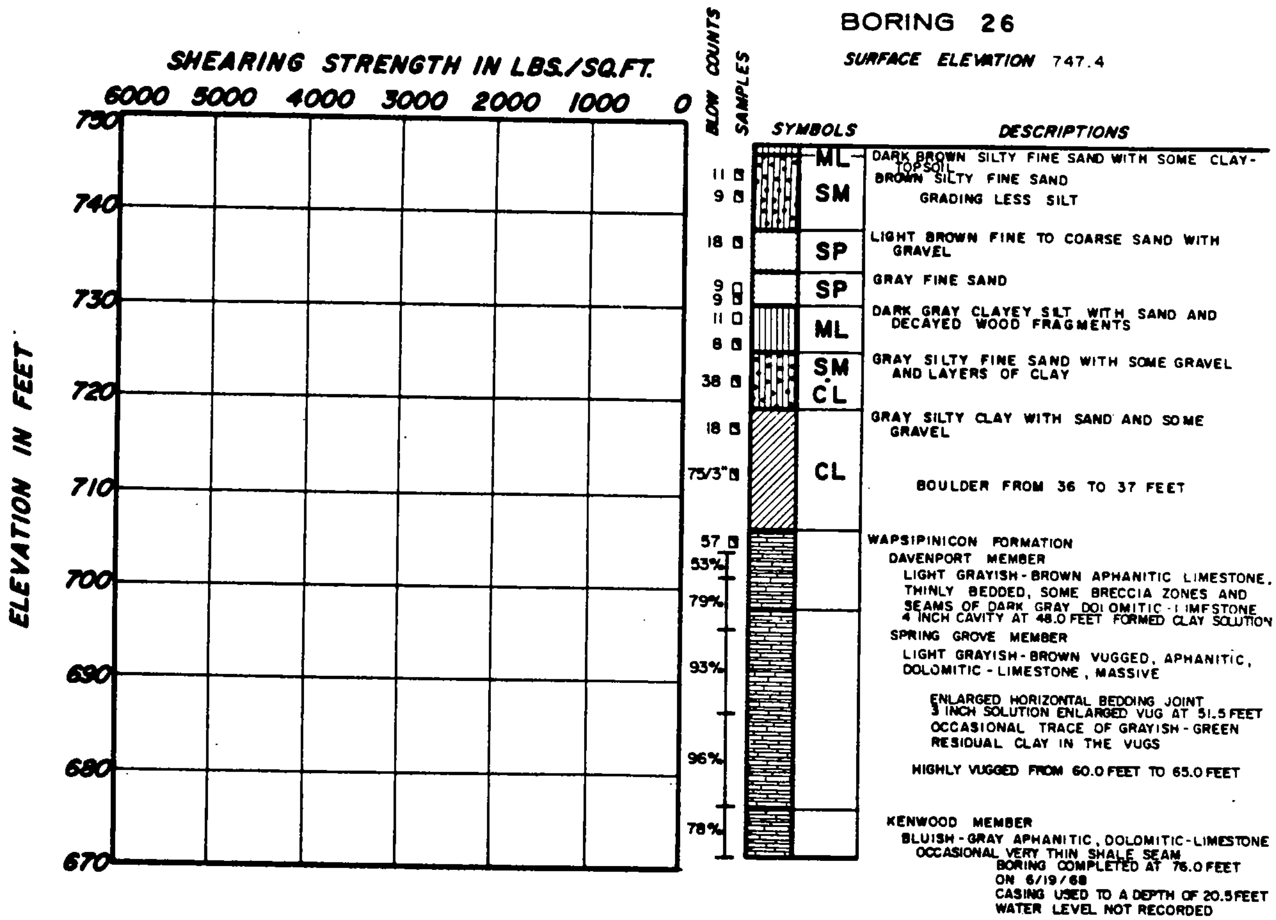

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Figure 2.5-56

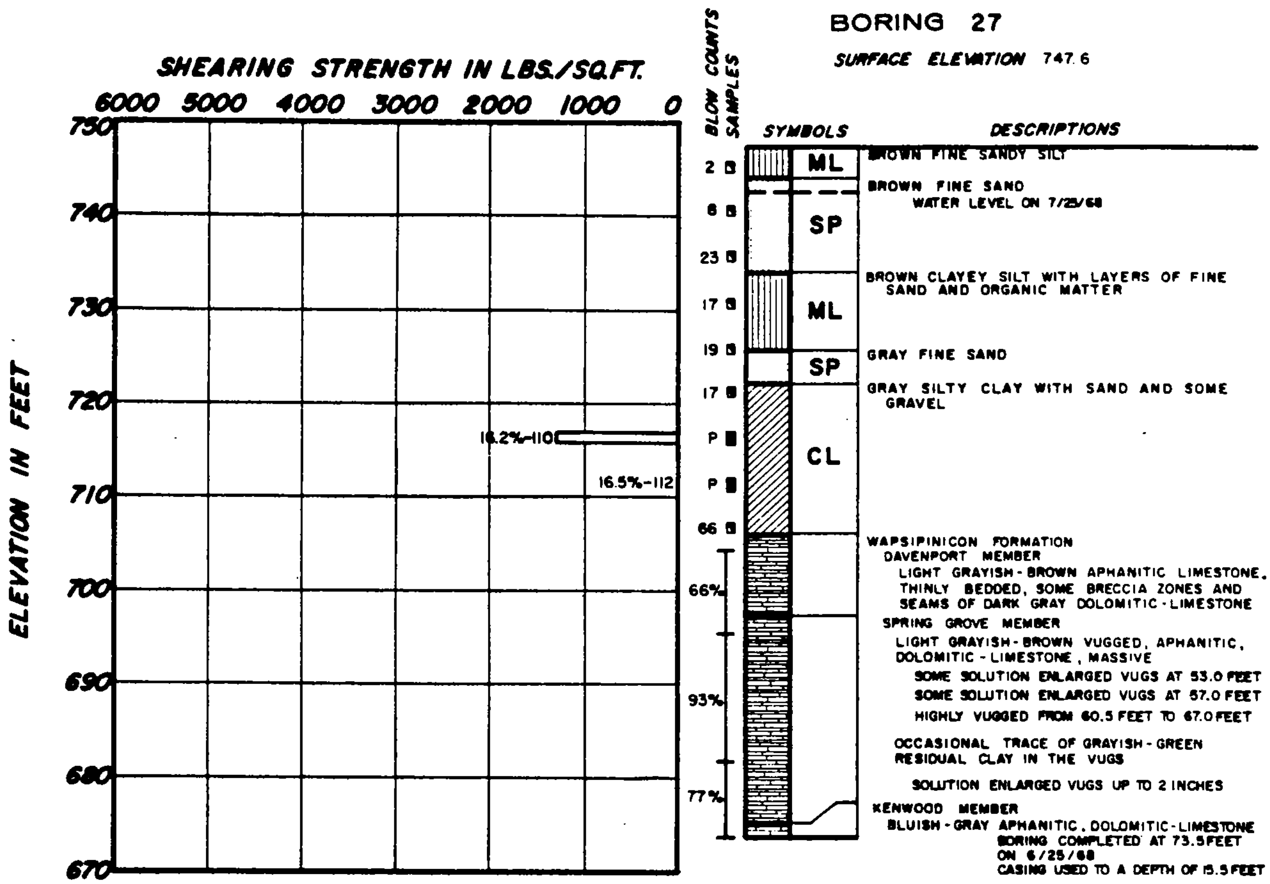

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Figure 2.5-57

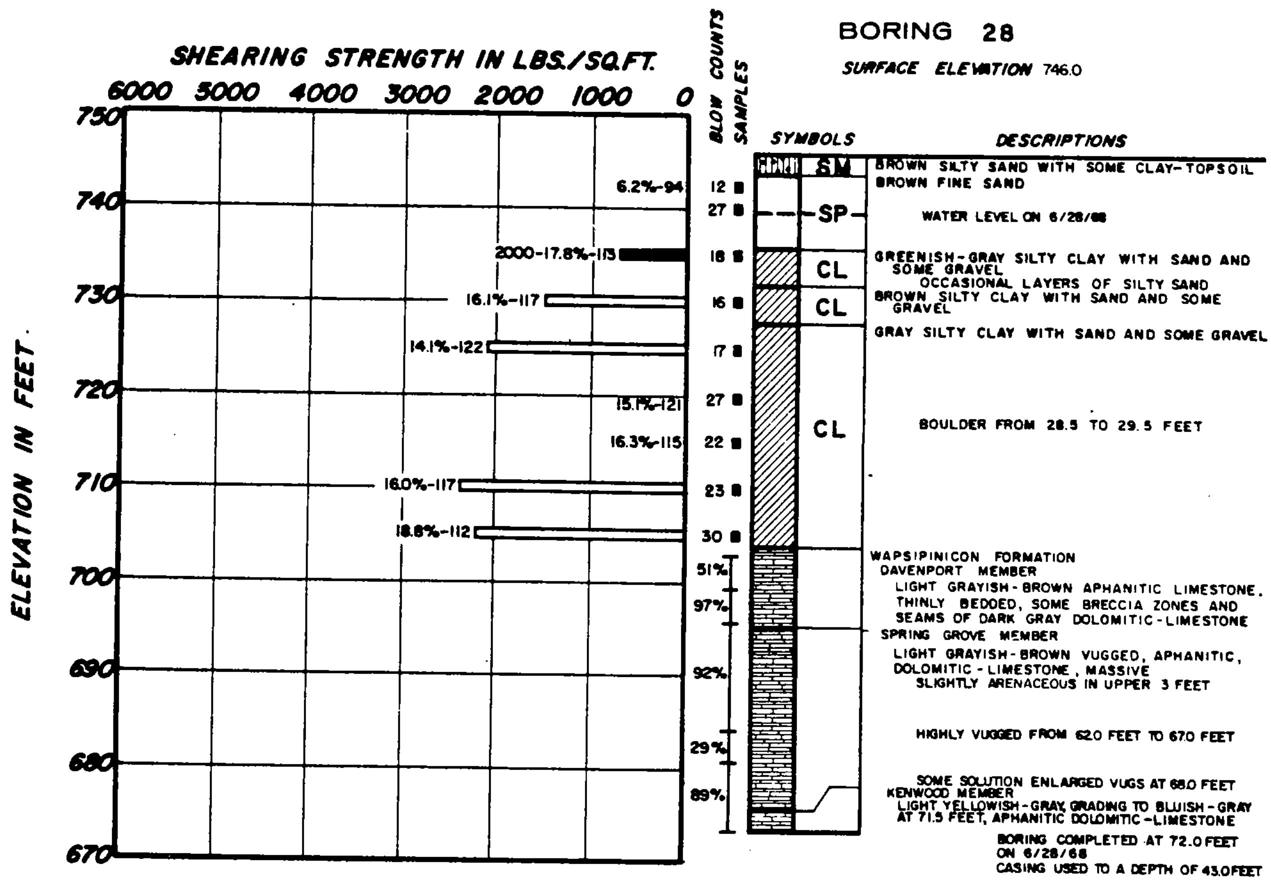

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Figure 2.5-58

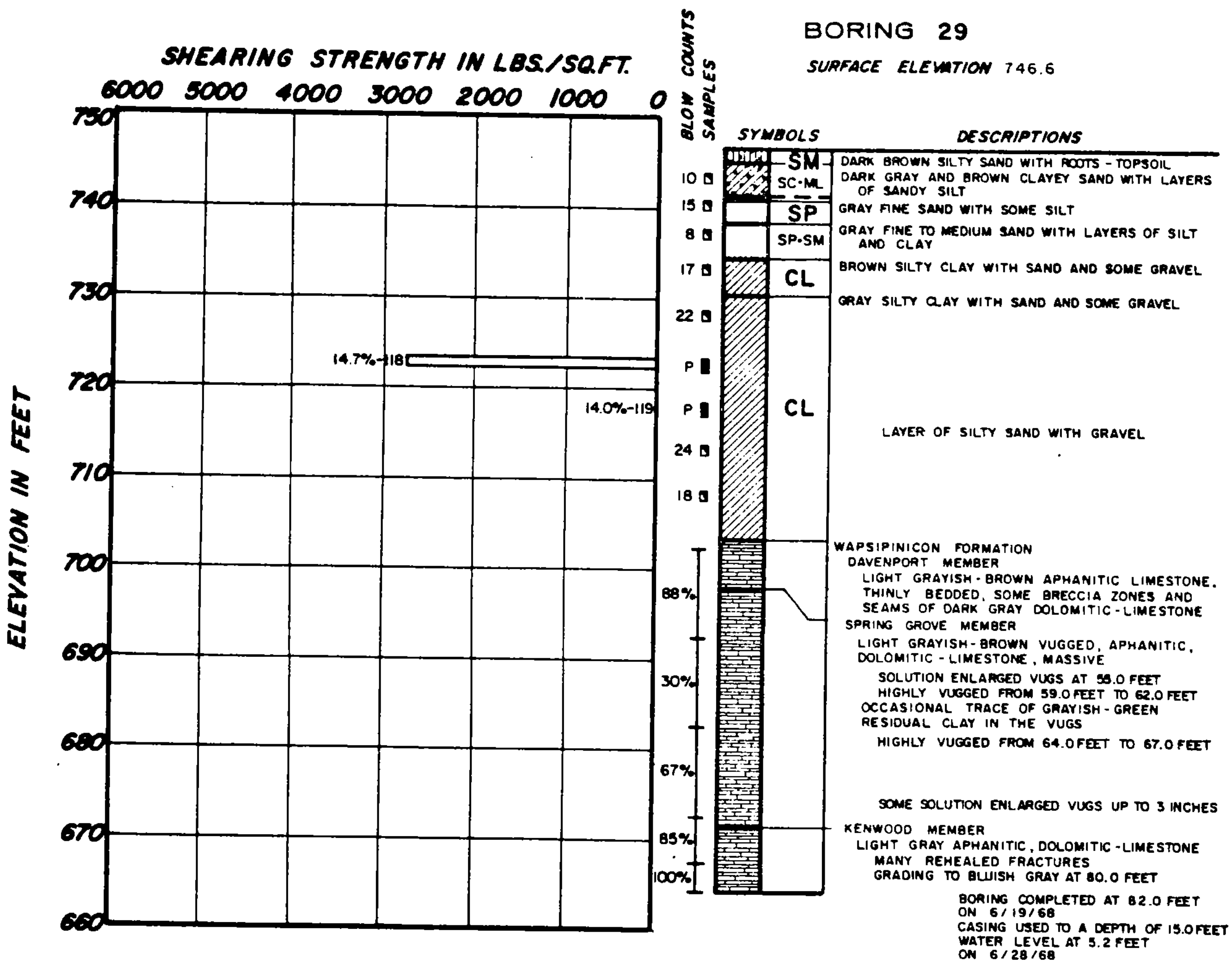

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Figure 2.5-59

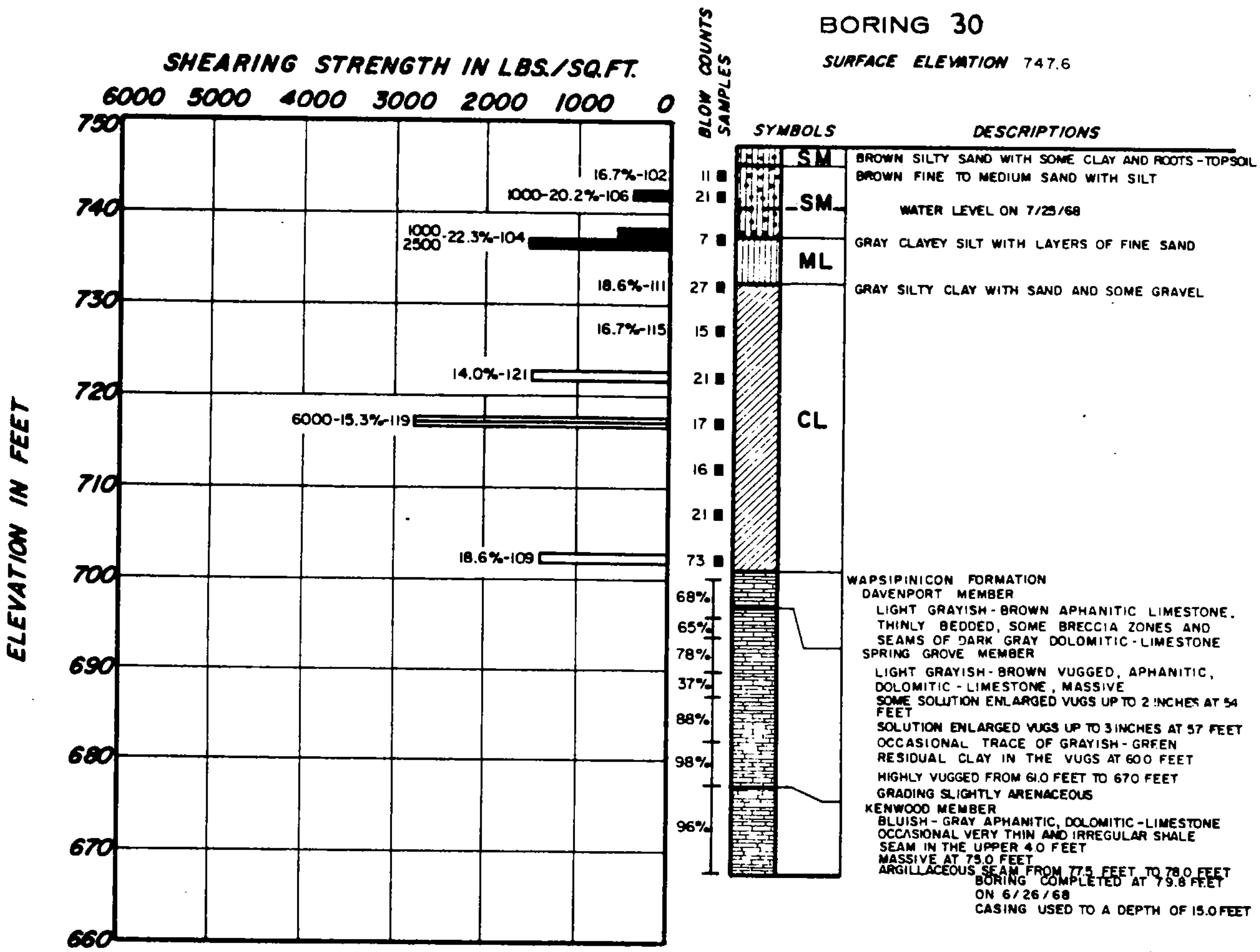

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Figure 2.5-60

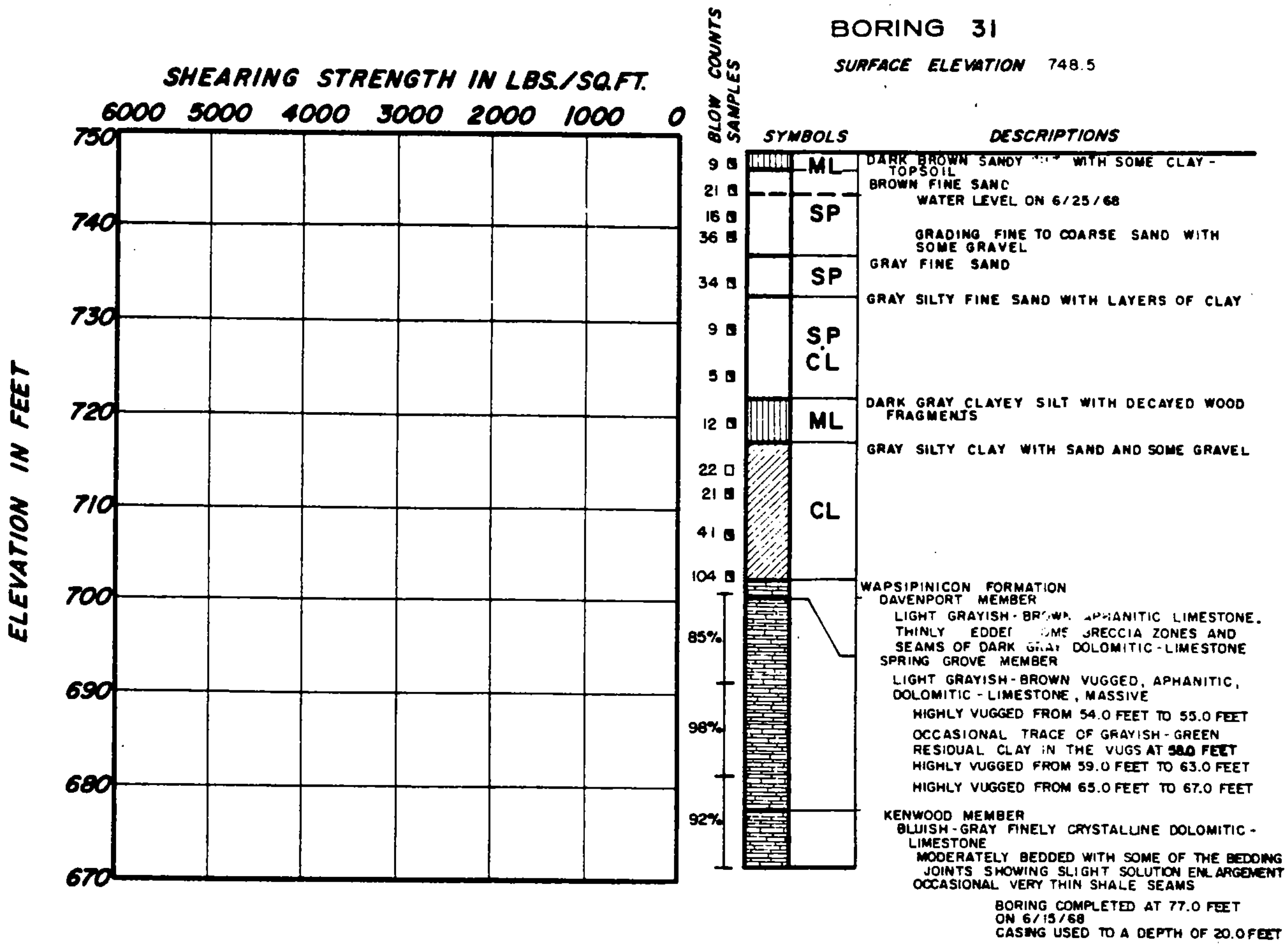

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Figure 2.5-61

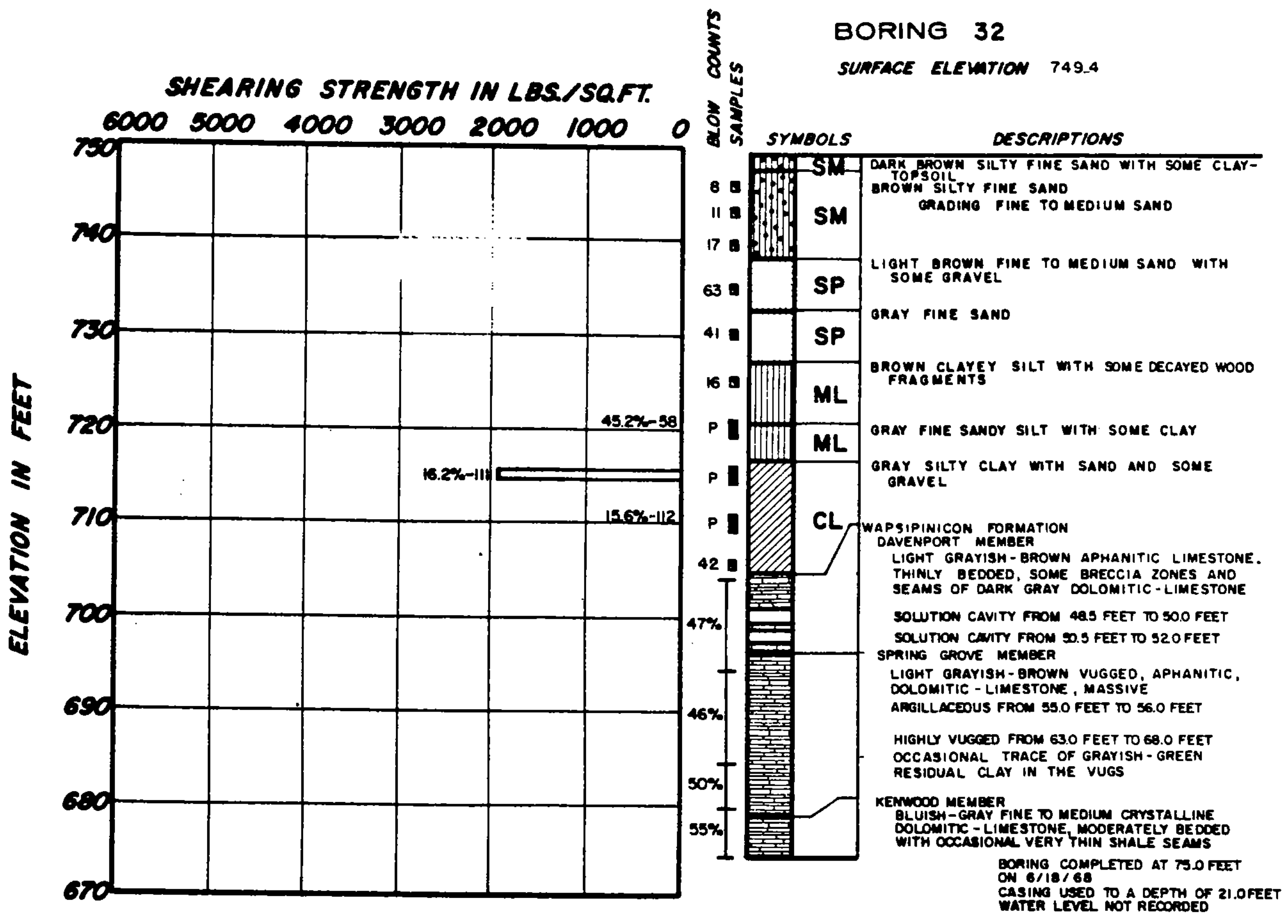

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Figure 2.5-62

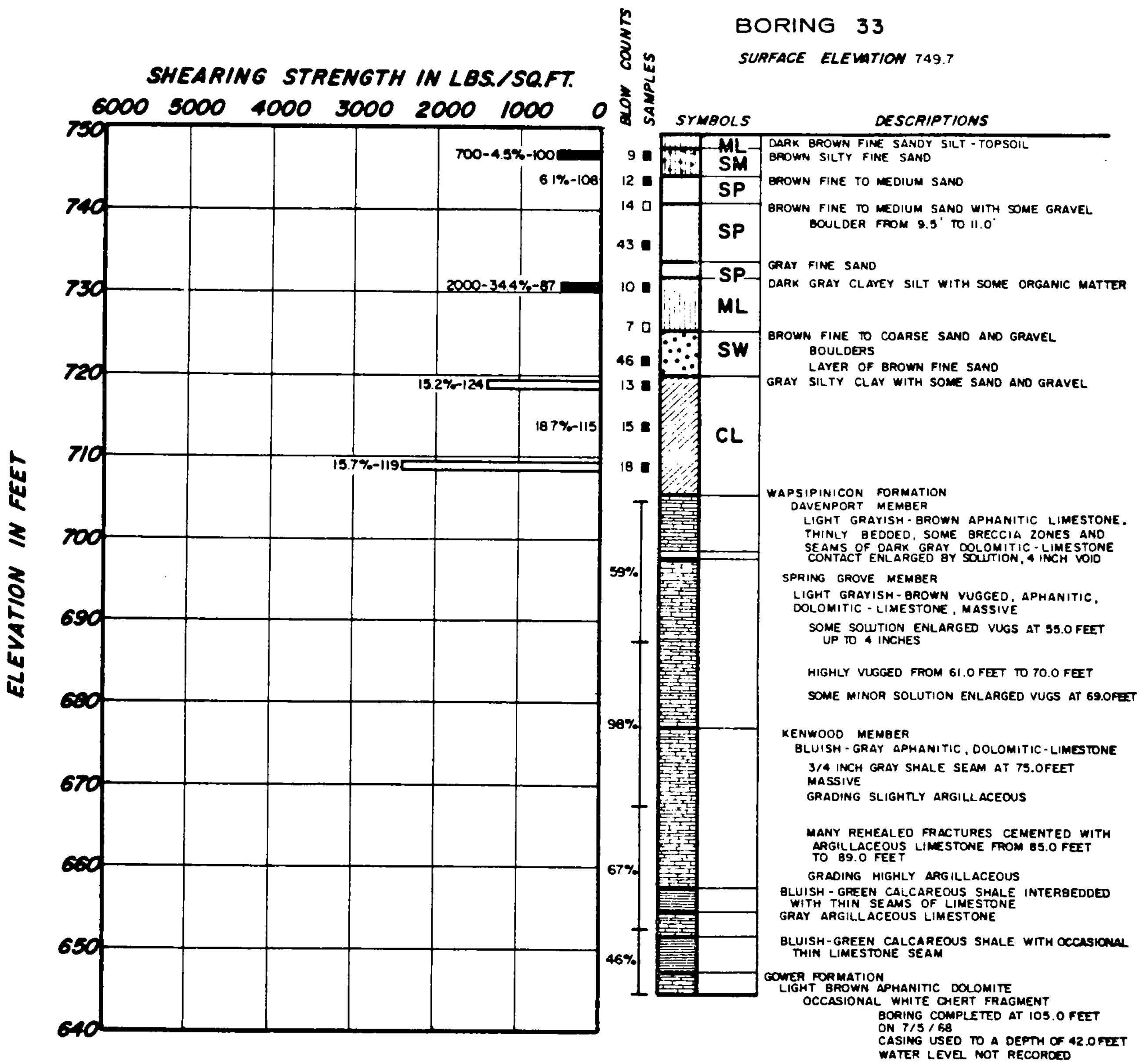

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Figure 2.5-63

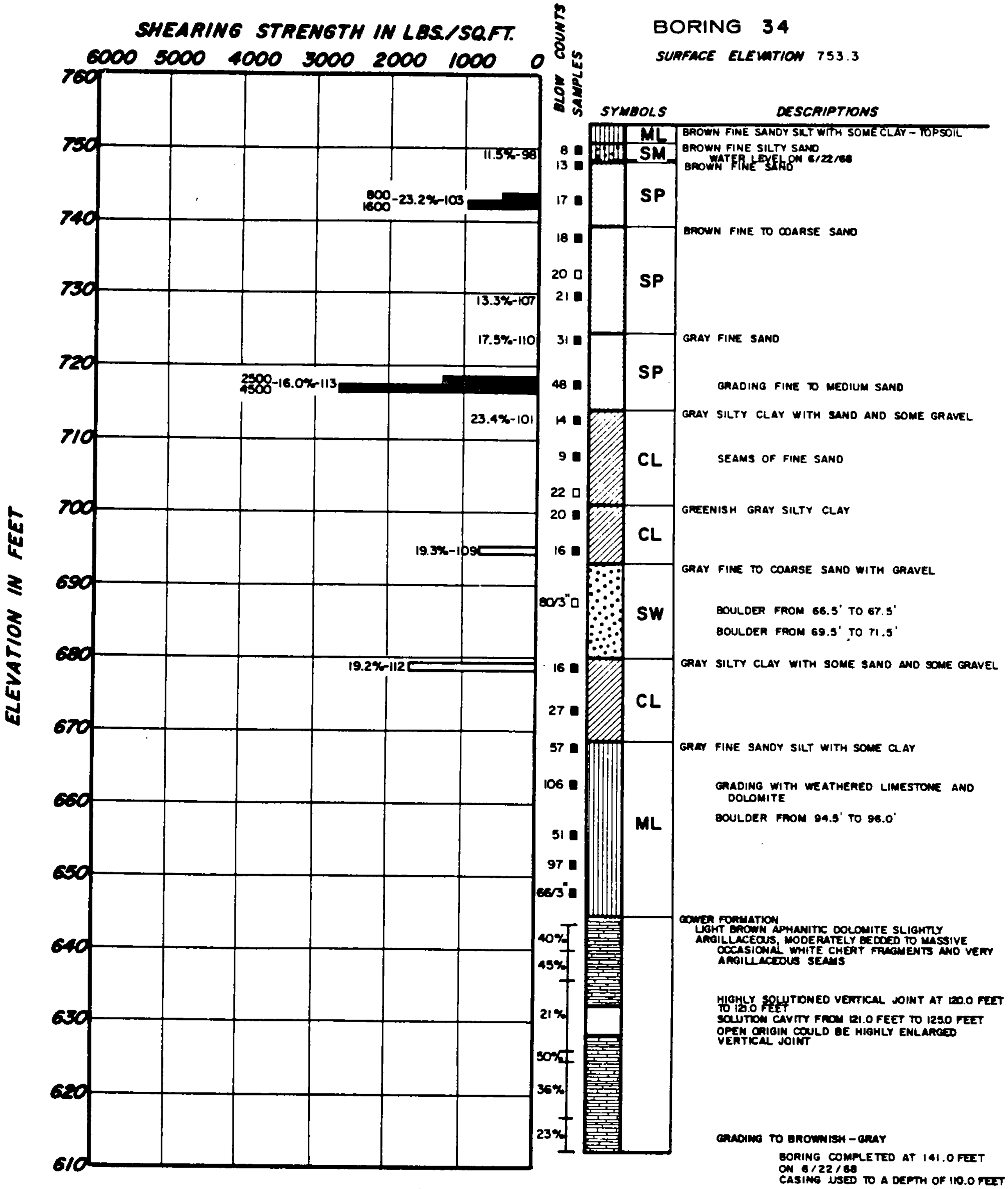

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Figure 2.5-64

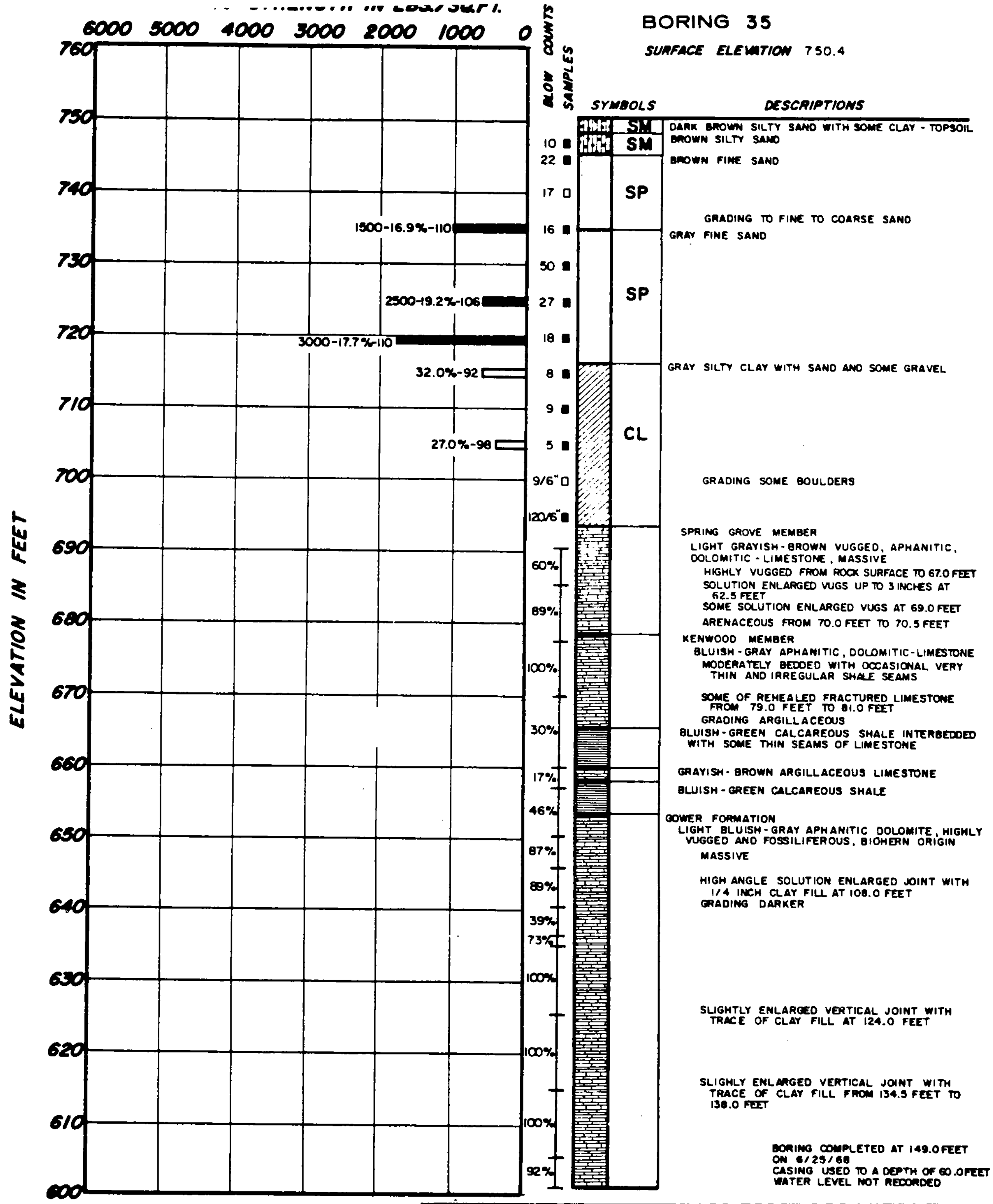

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Figure 2.5-65

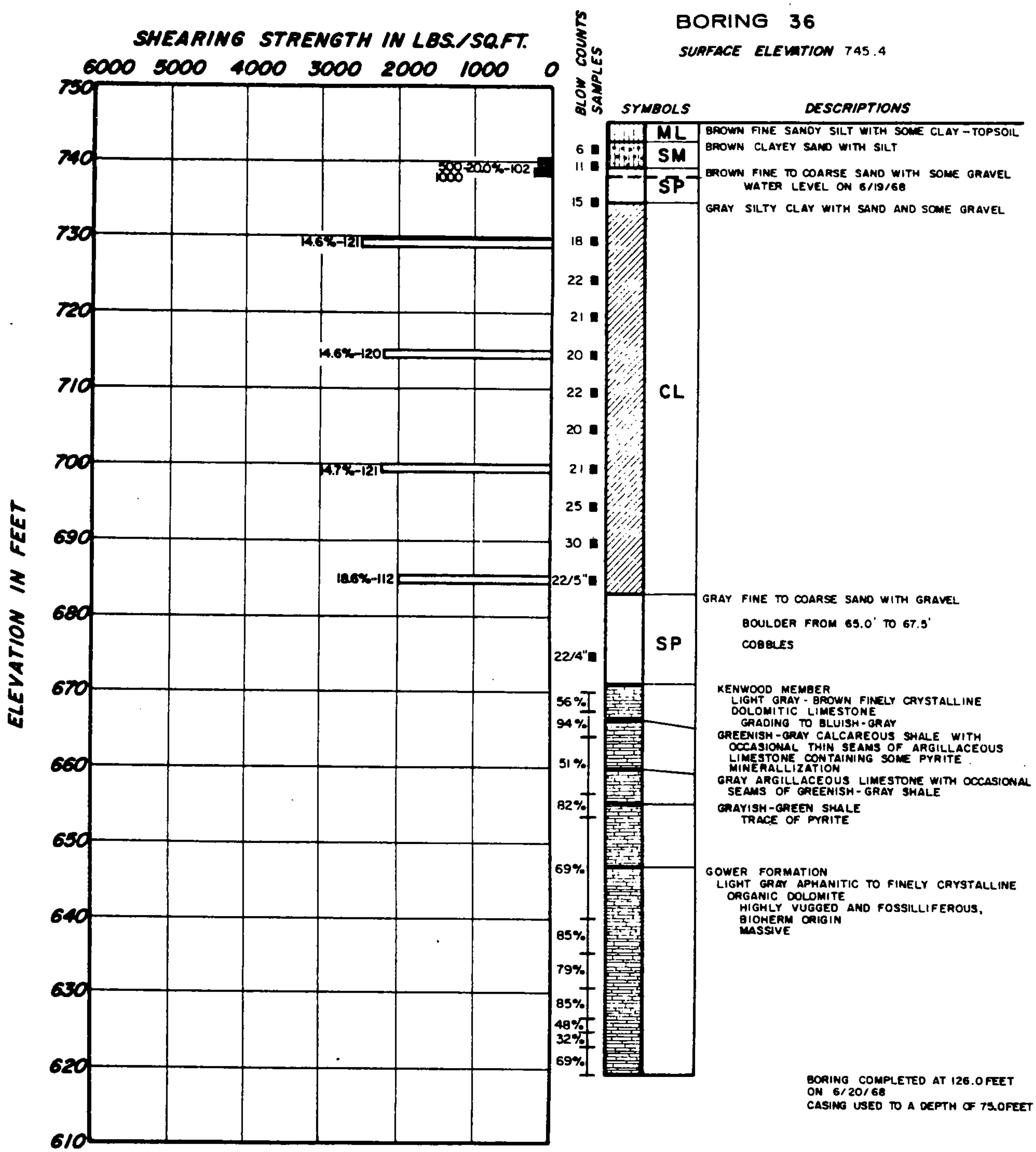

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Figure 2.5-66

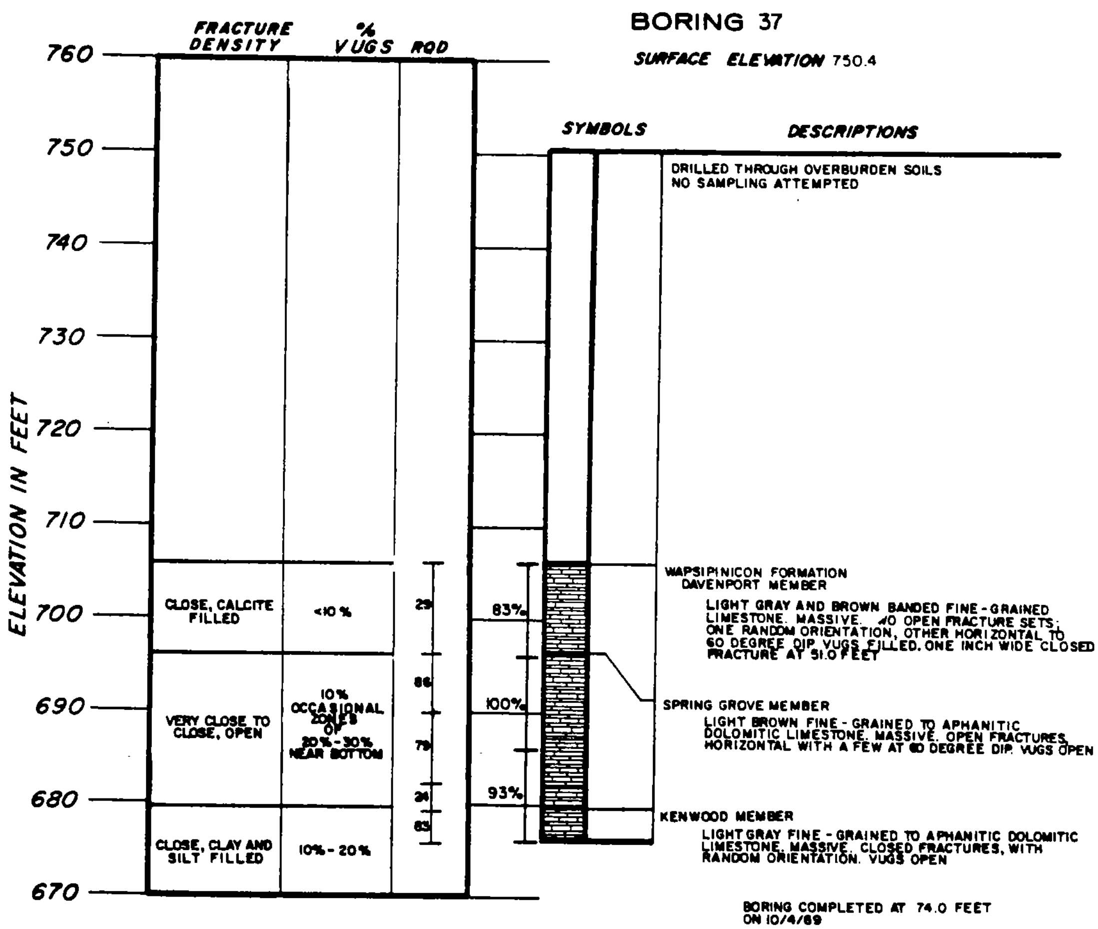

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Figure 2.5-67

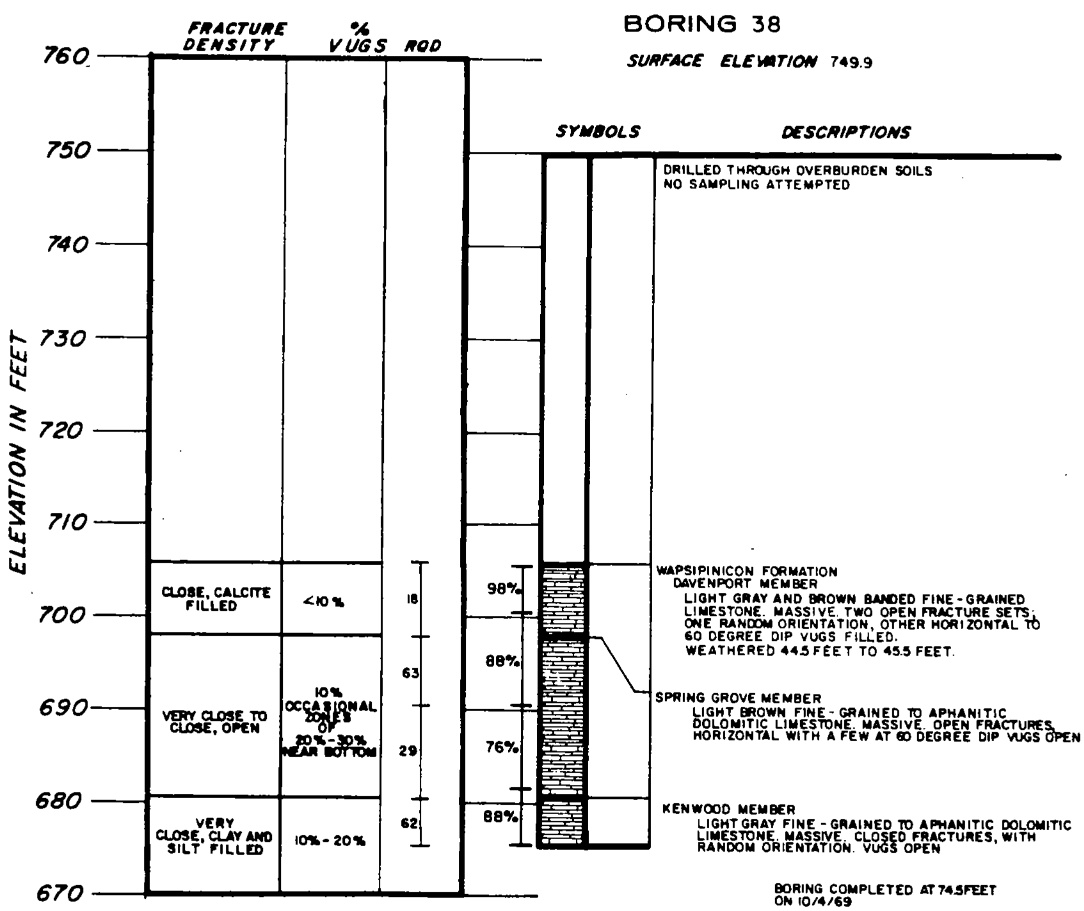

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Figure 2.5-68

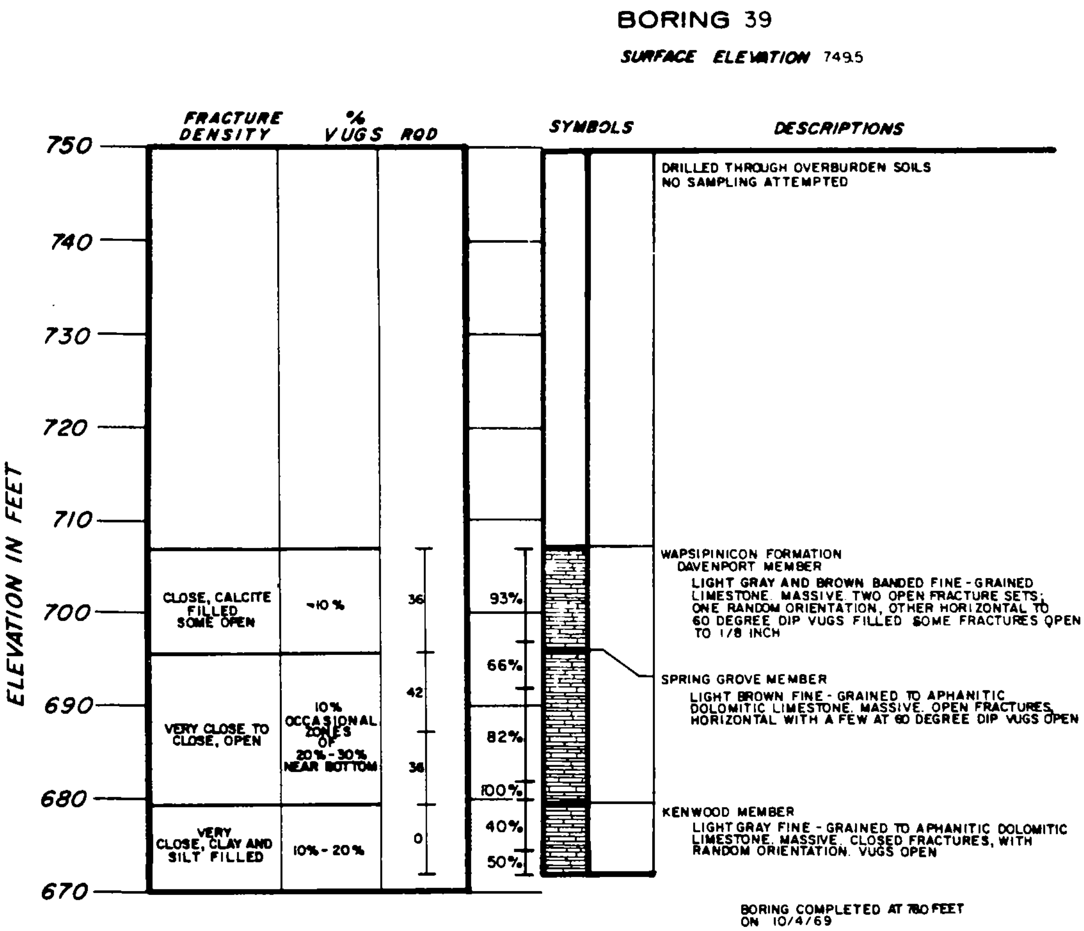

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Figure 2.5-69

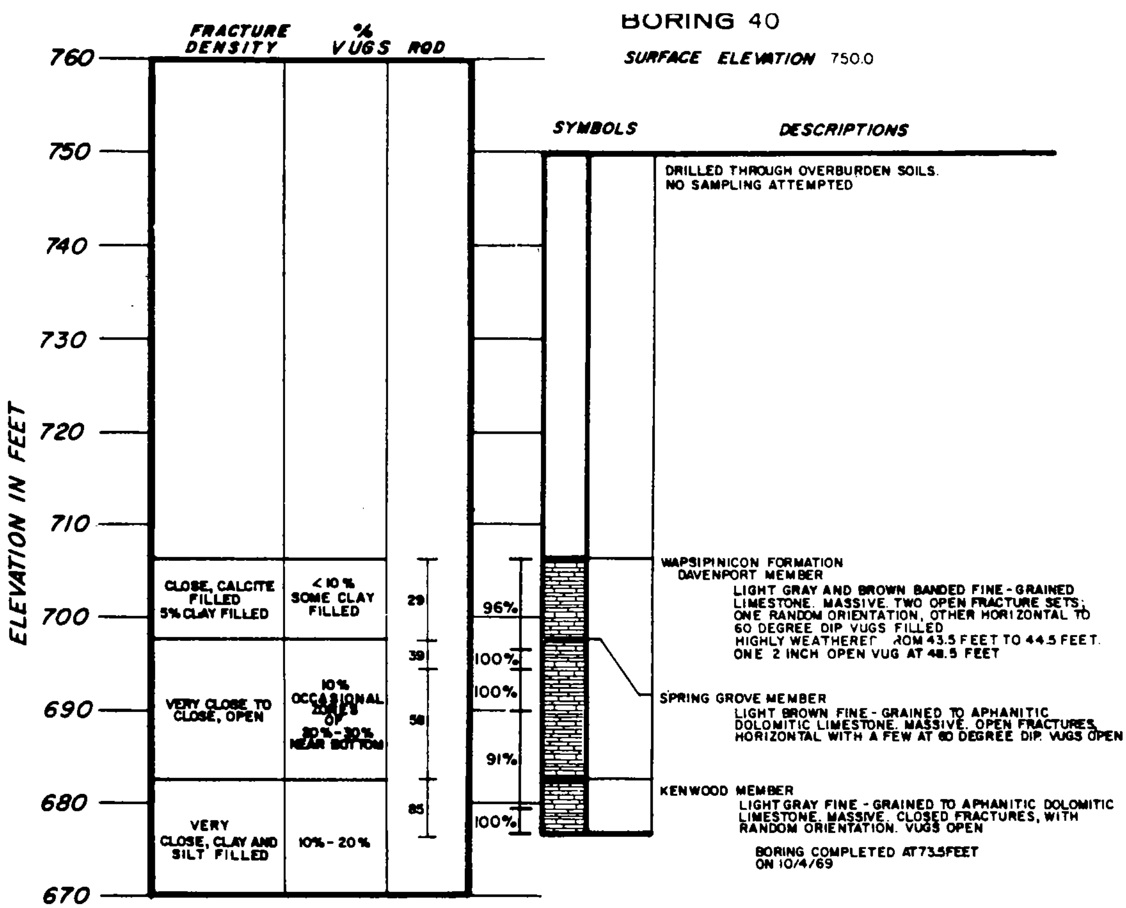

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Figure 2.5-70

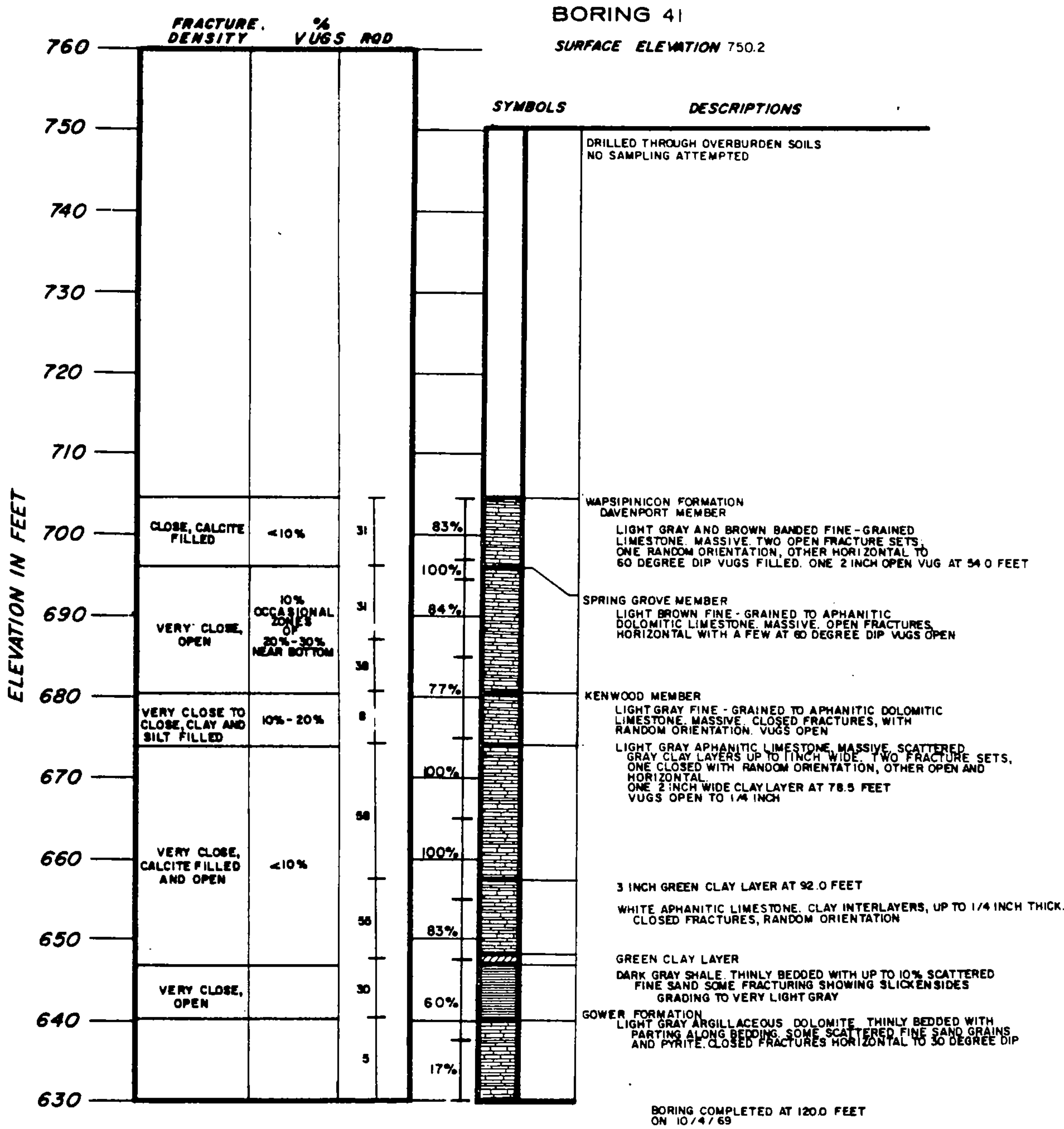

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Figure 2.5-71

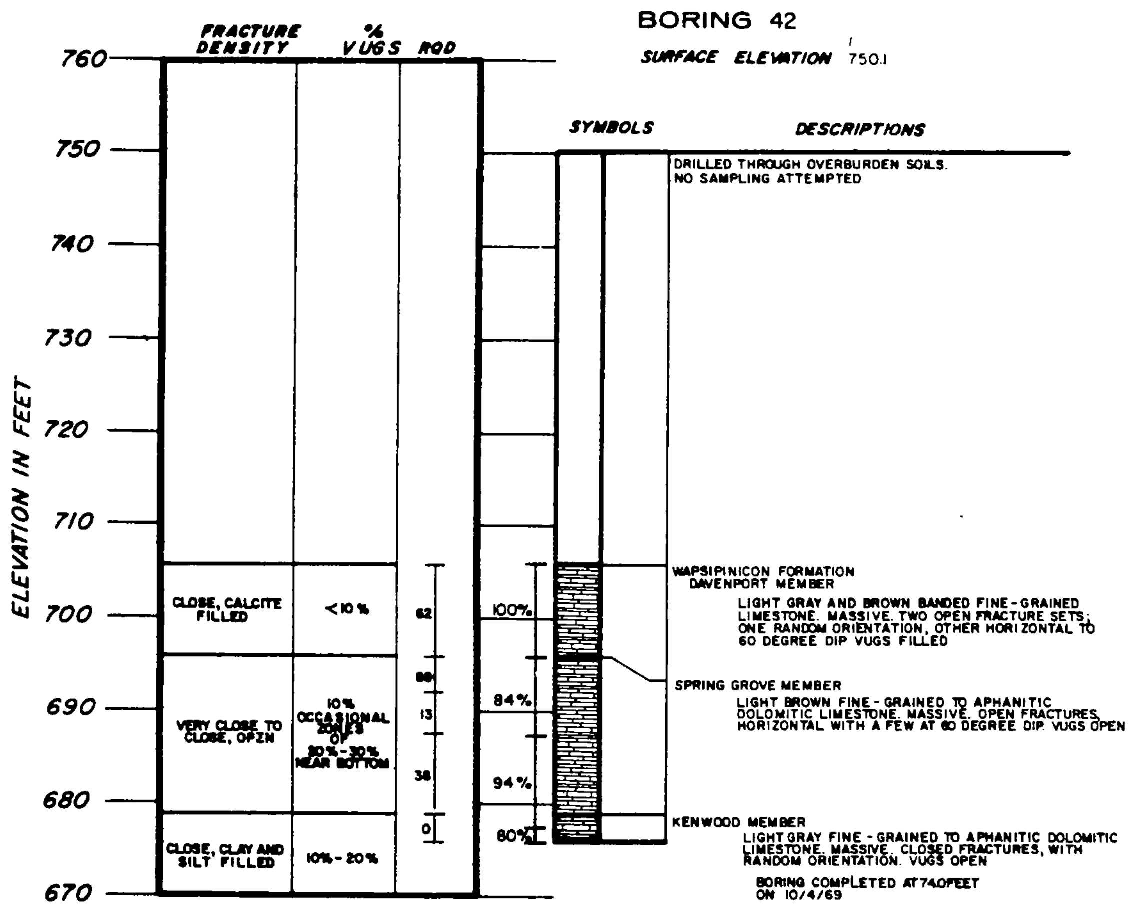

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Figure 2.5-72

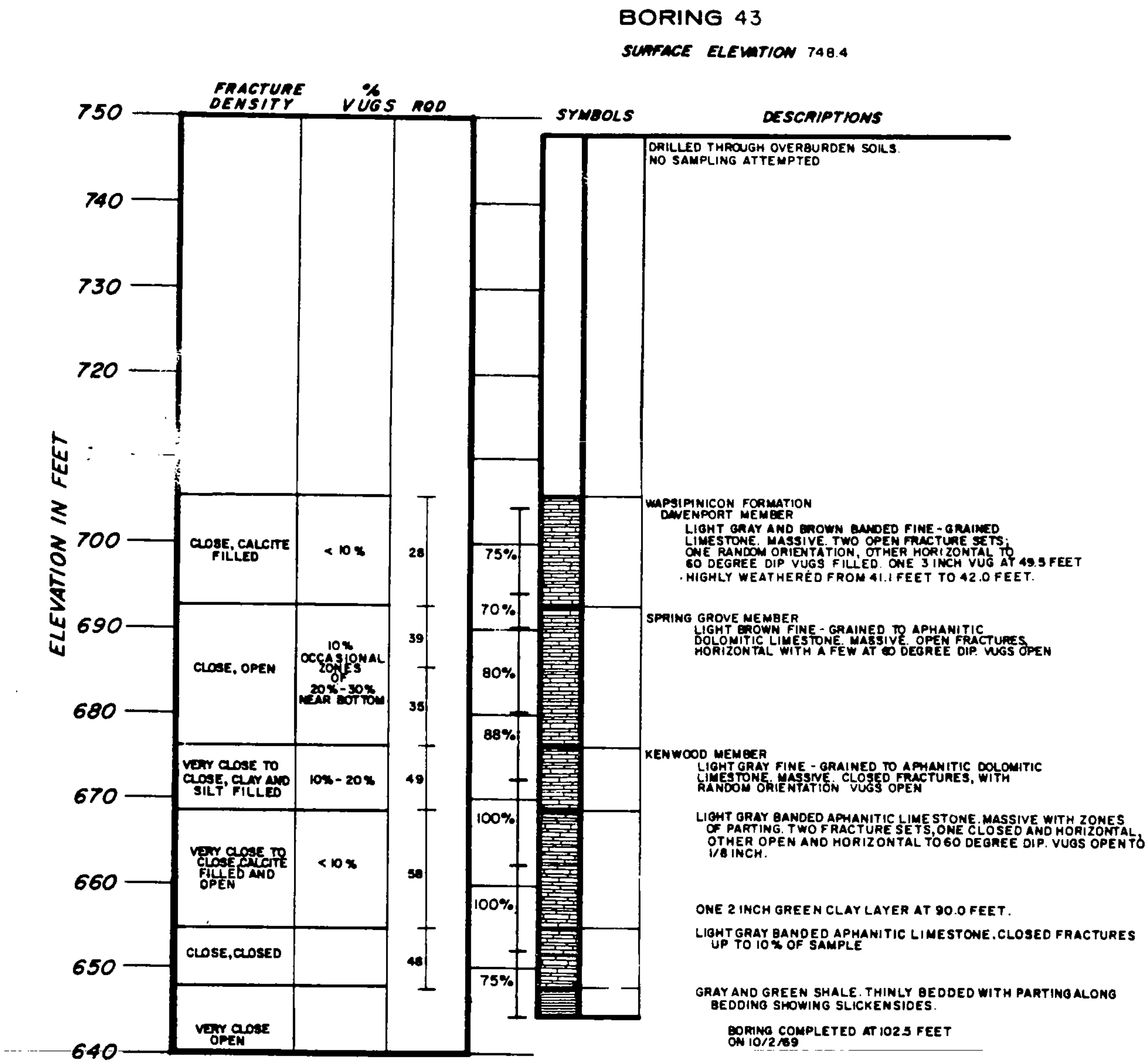

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Figure 2.5-73

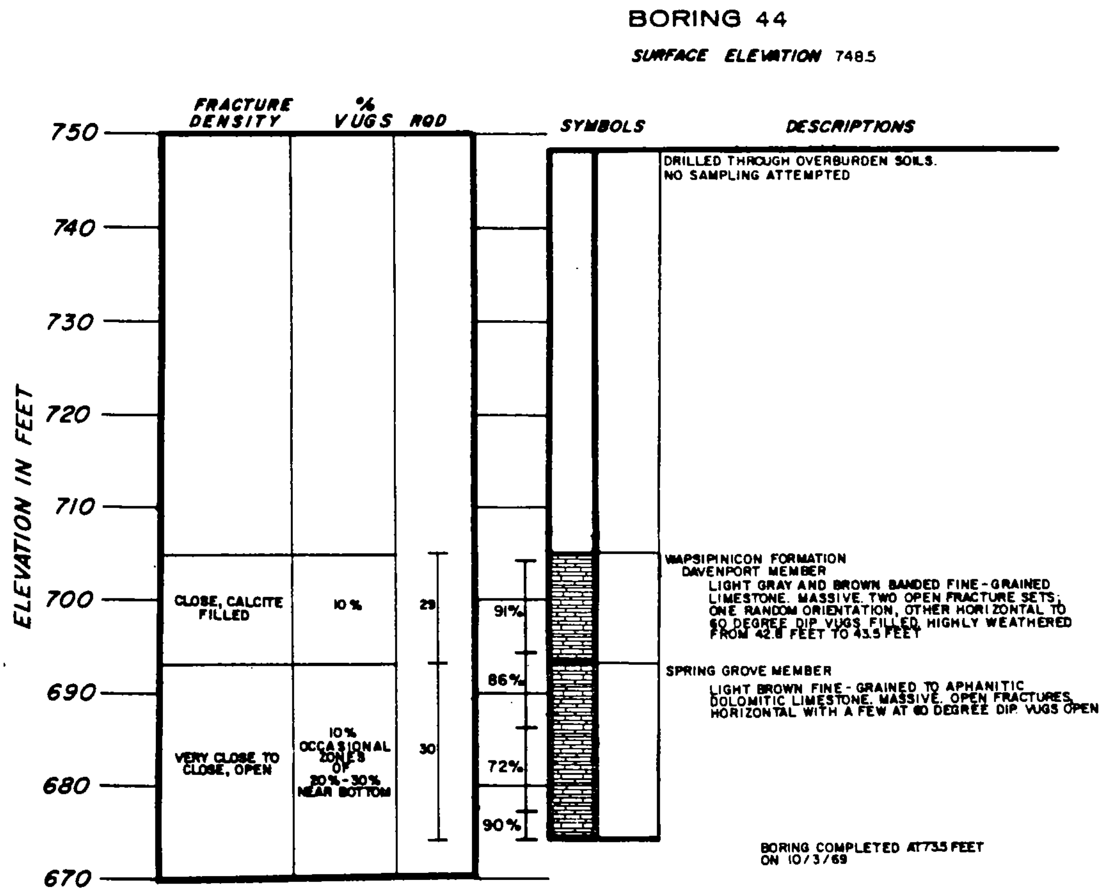

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Figure 2.5-74

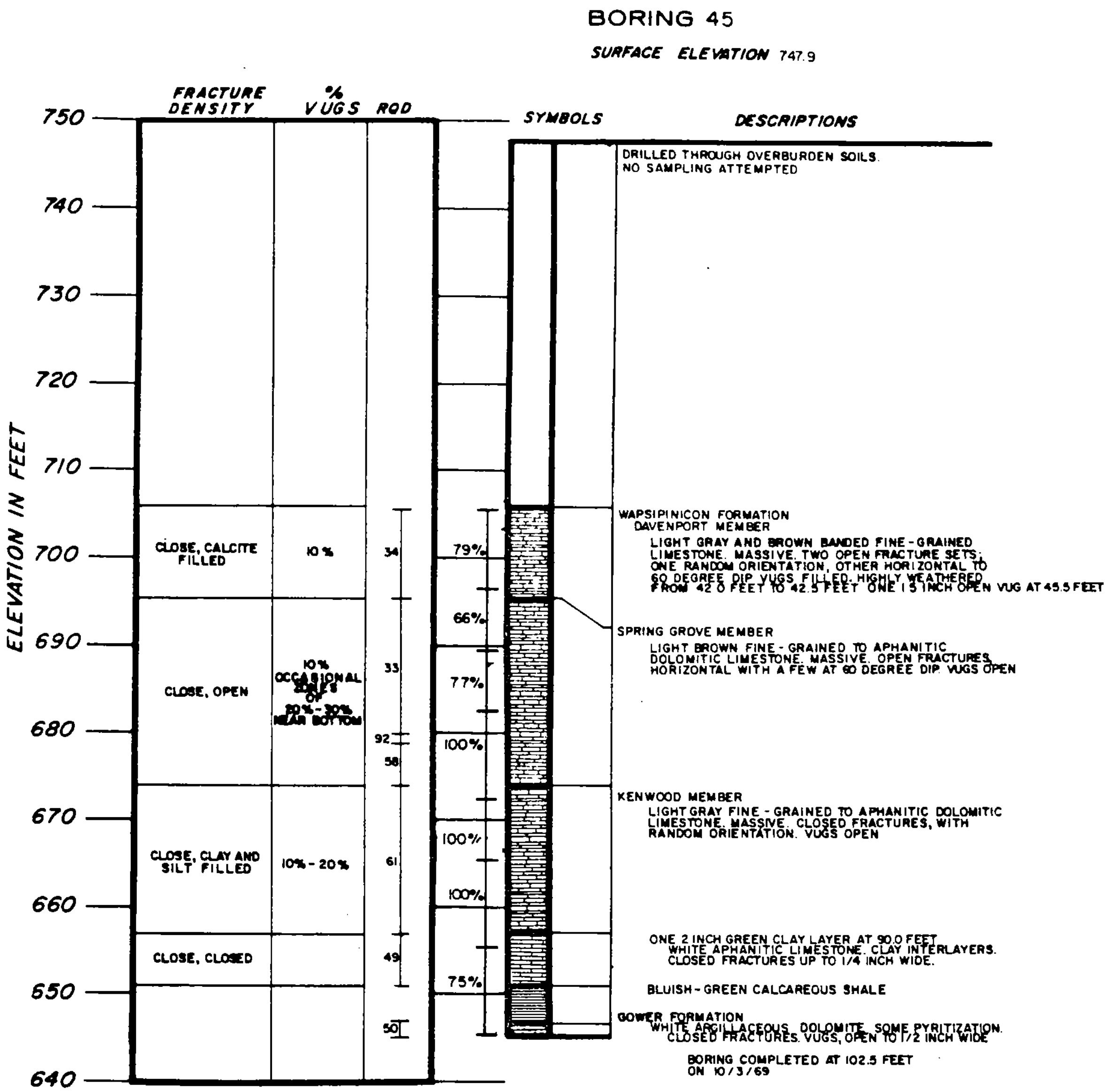

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Figure 2.5-75

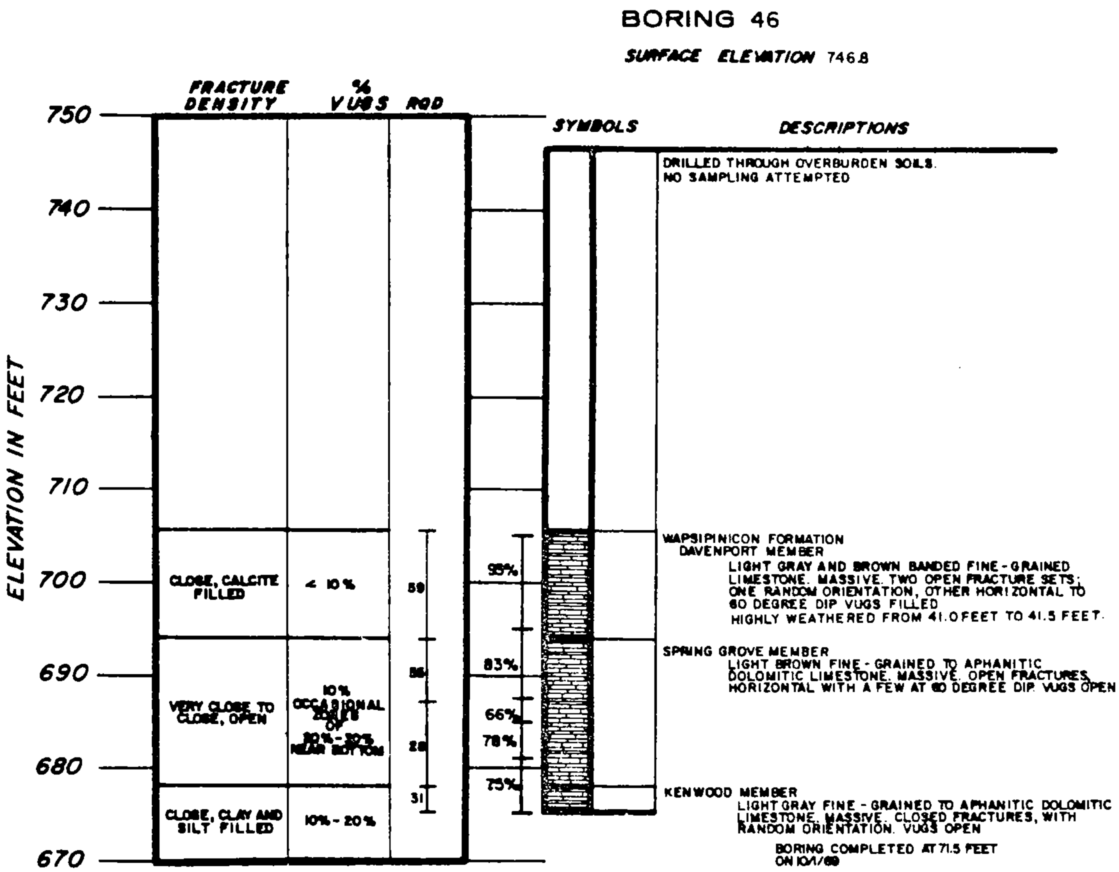

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Figure 2.5-76

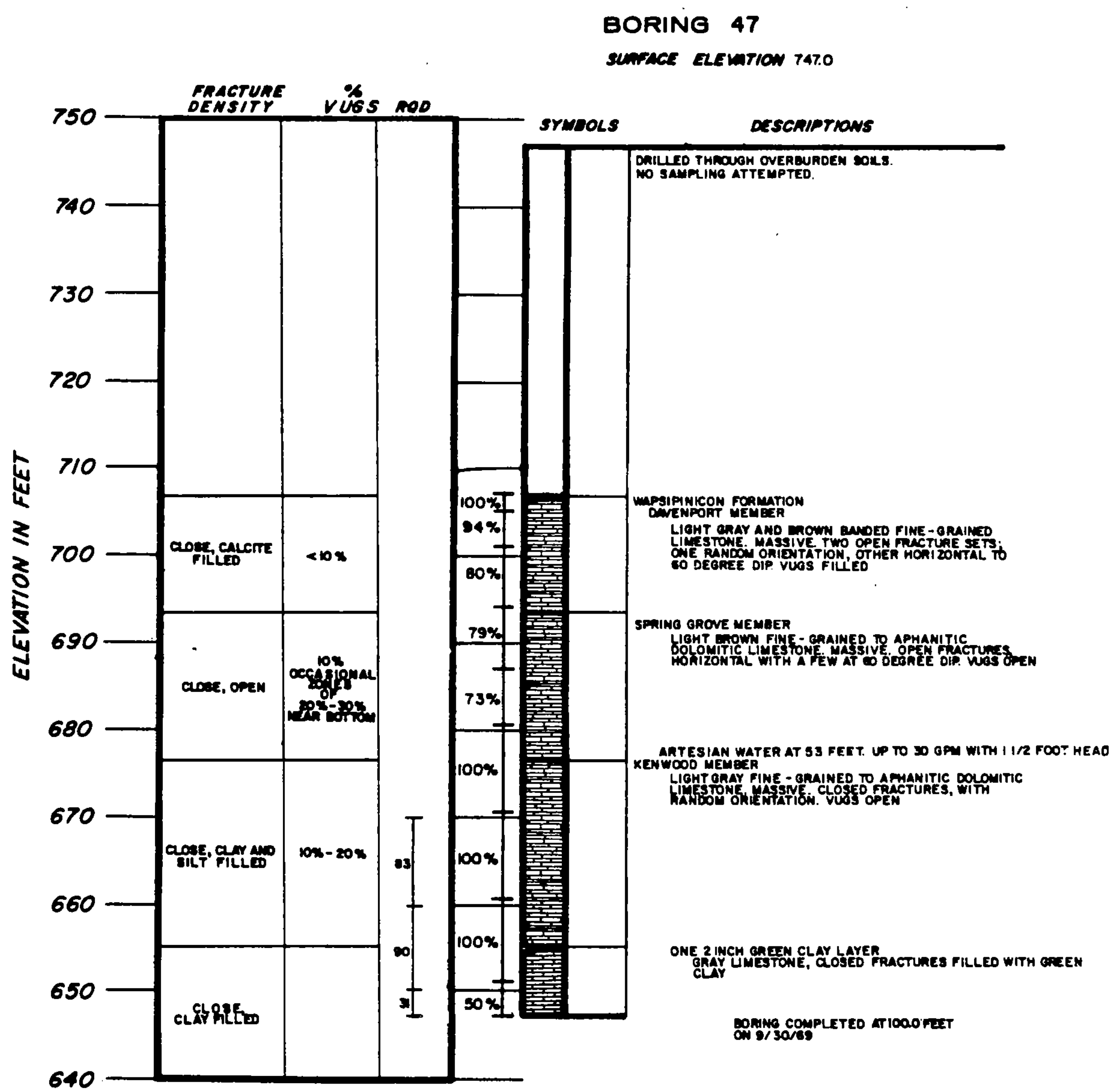

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Figure 2.5-77

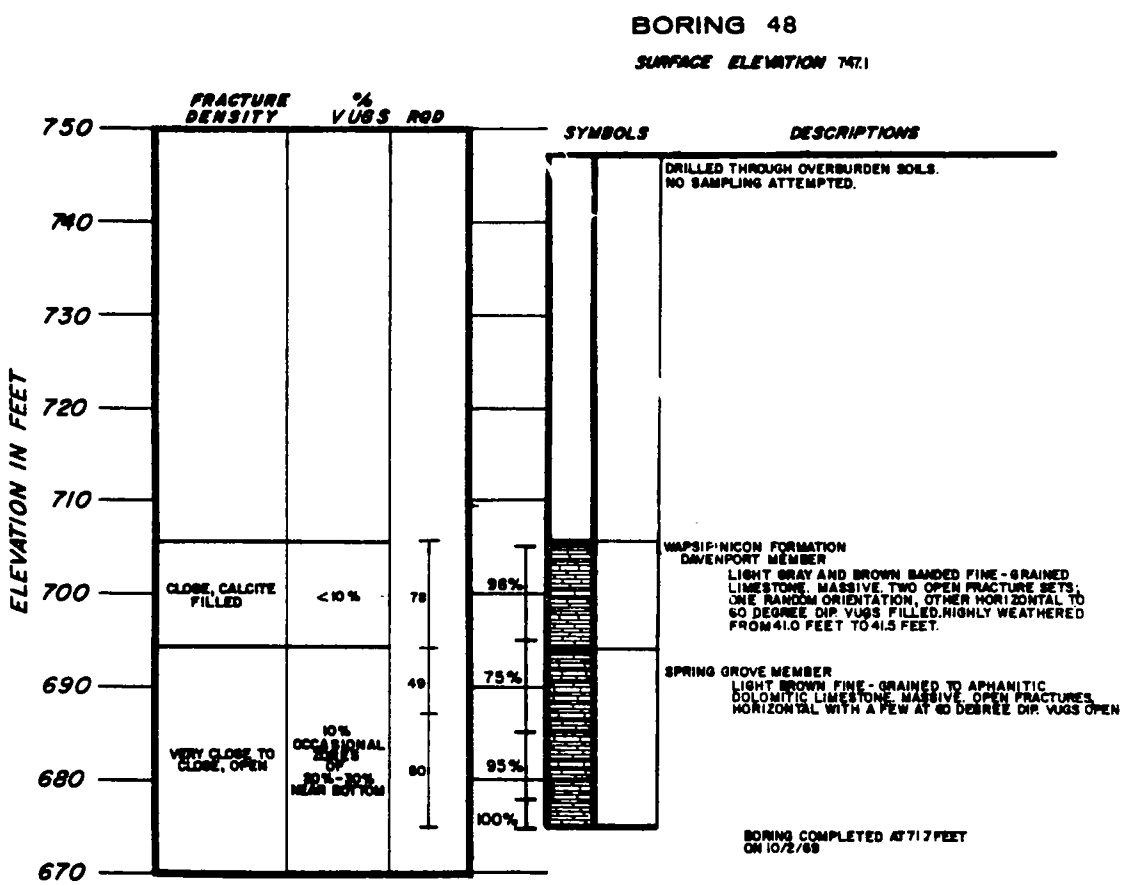

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Figure 2.5-78

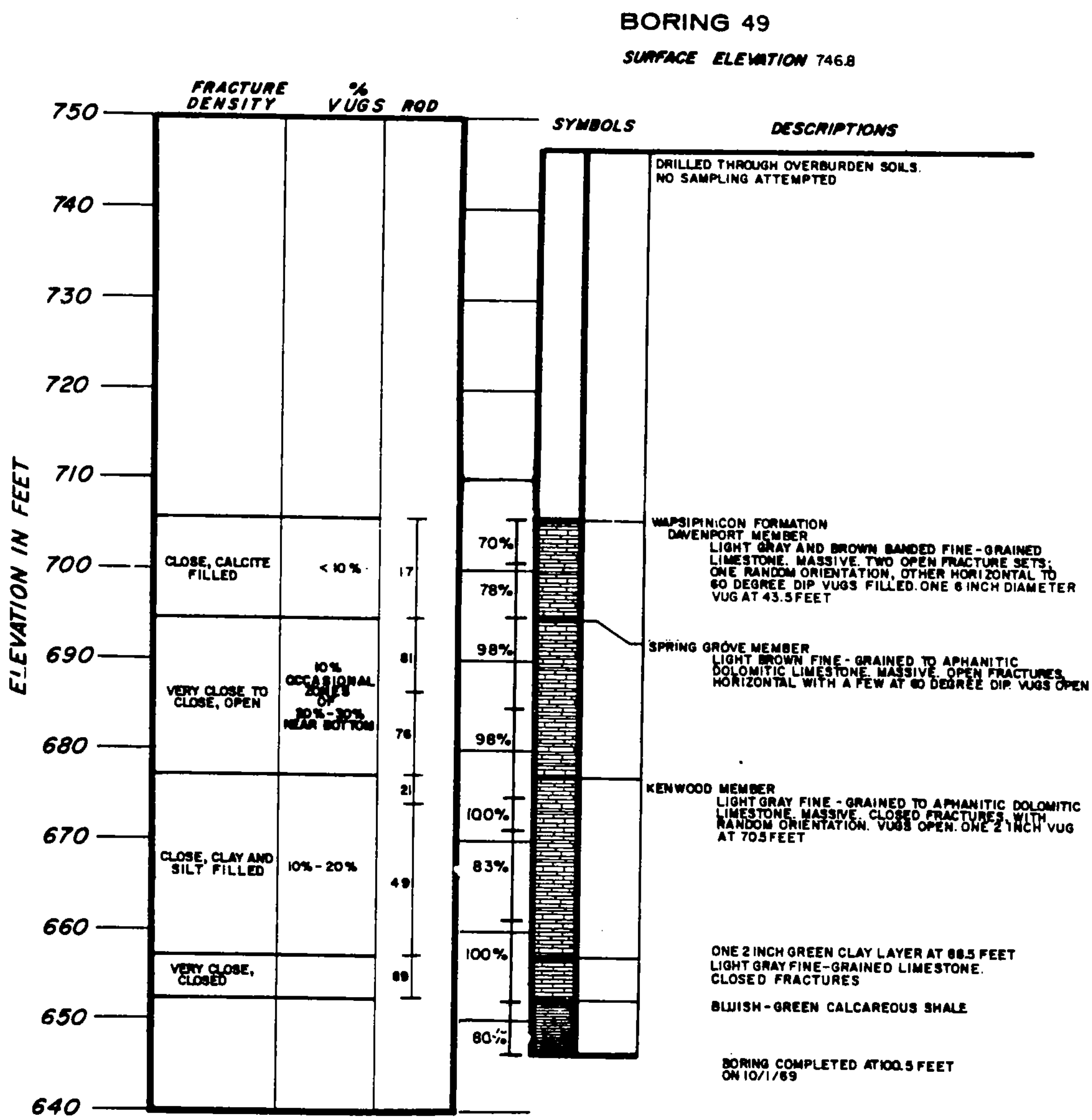

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Figure 2.5-79

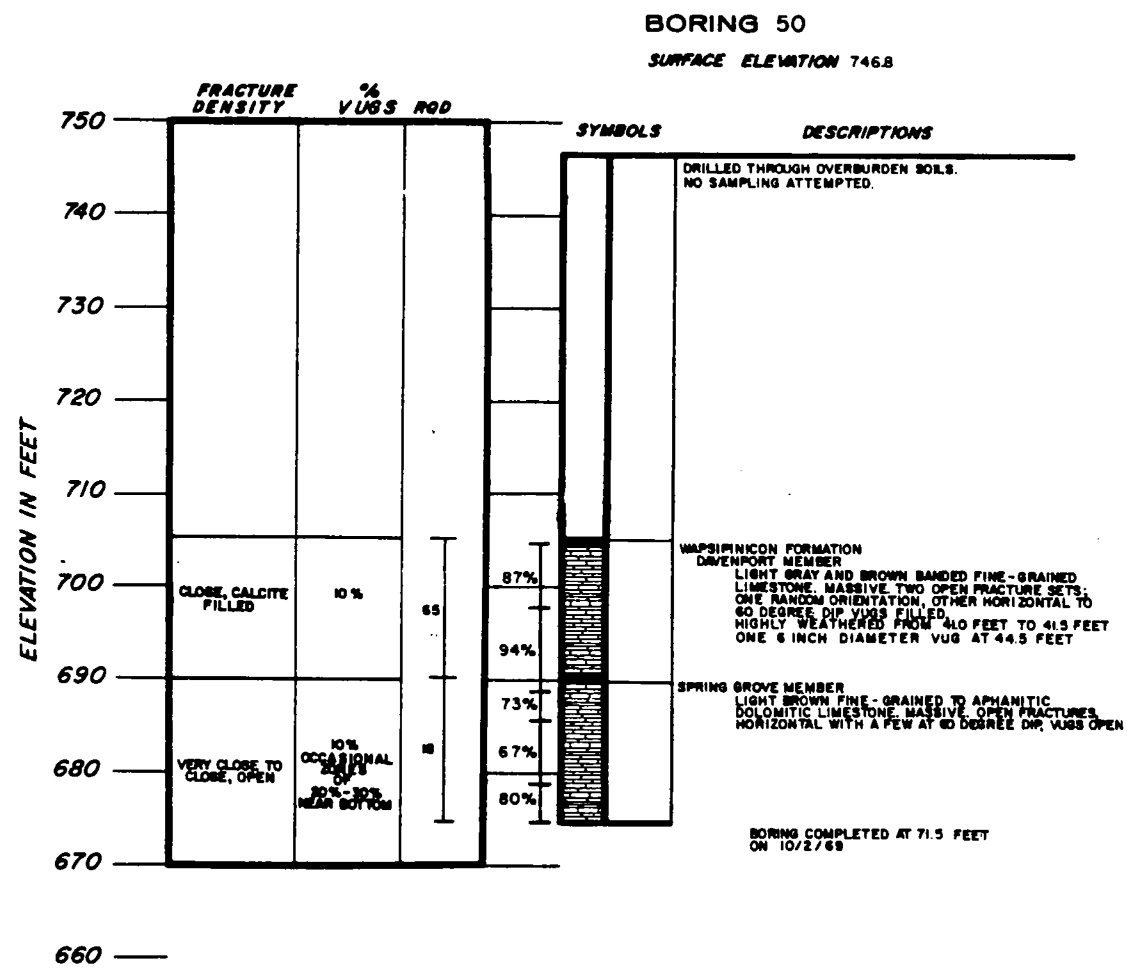

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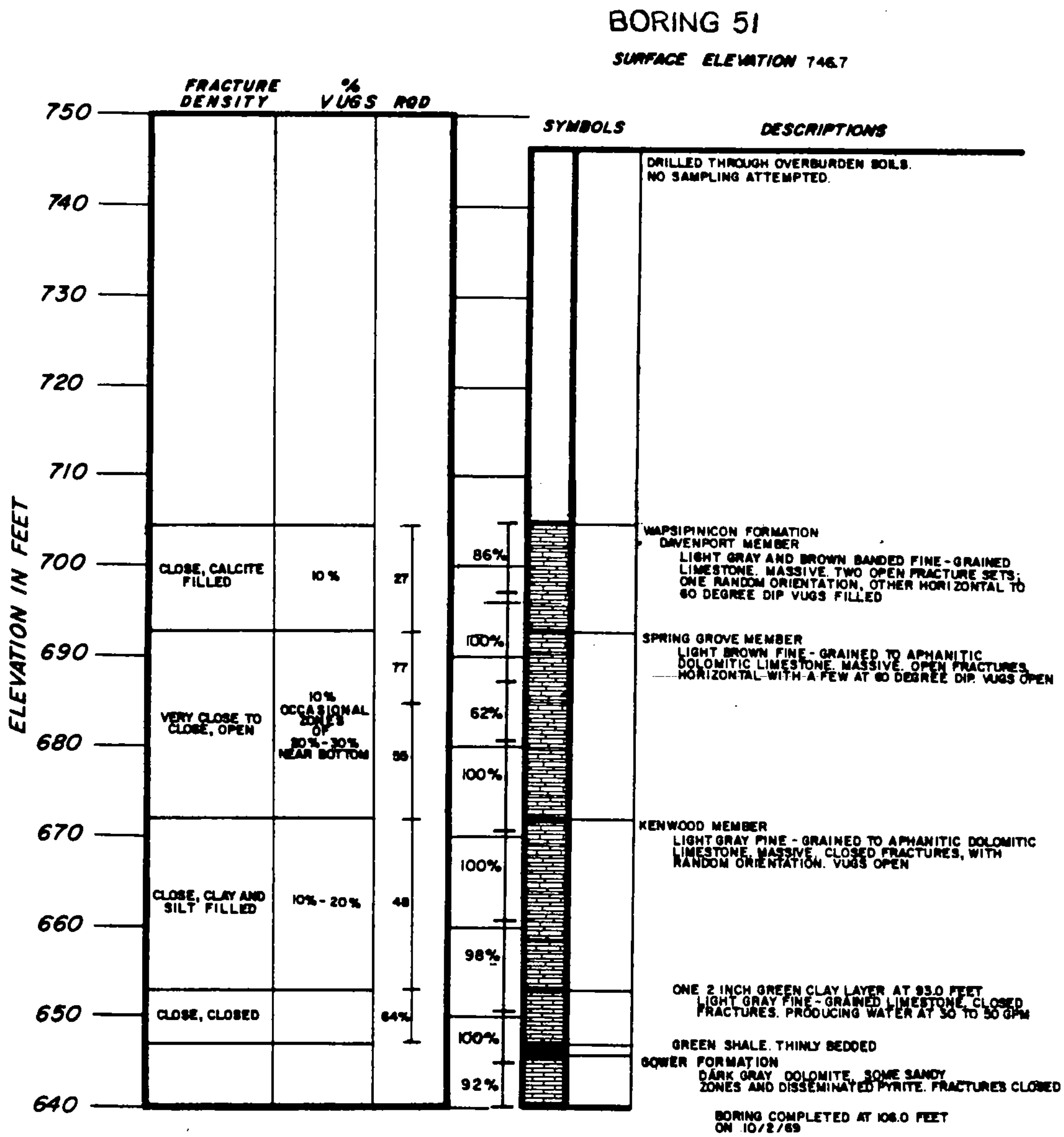
Figure 2.5-80



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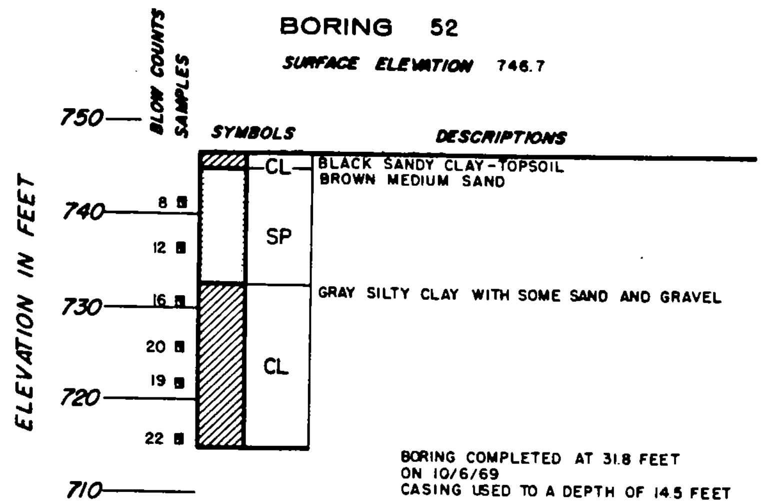
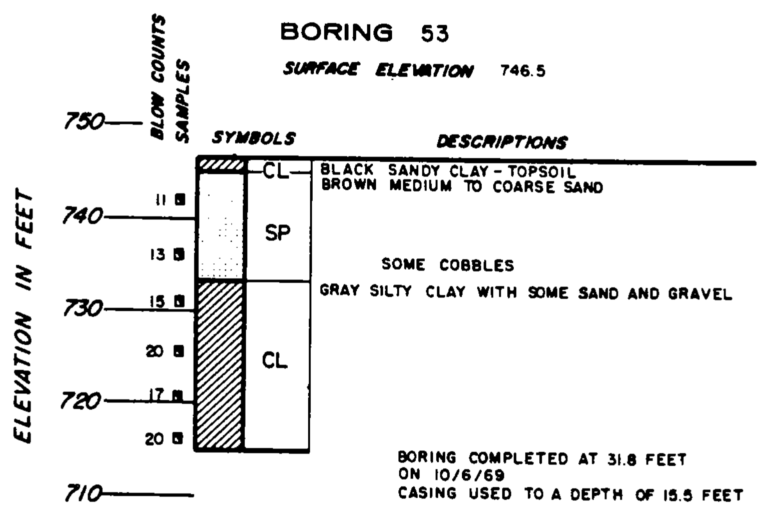
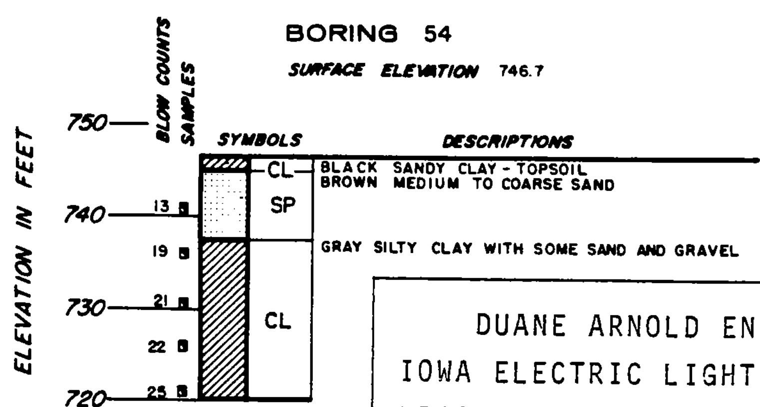
Log of Borings - Boring 51

Figure 2.5-81







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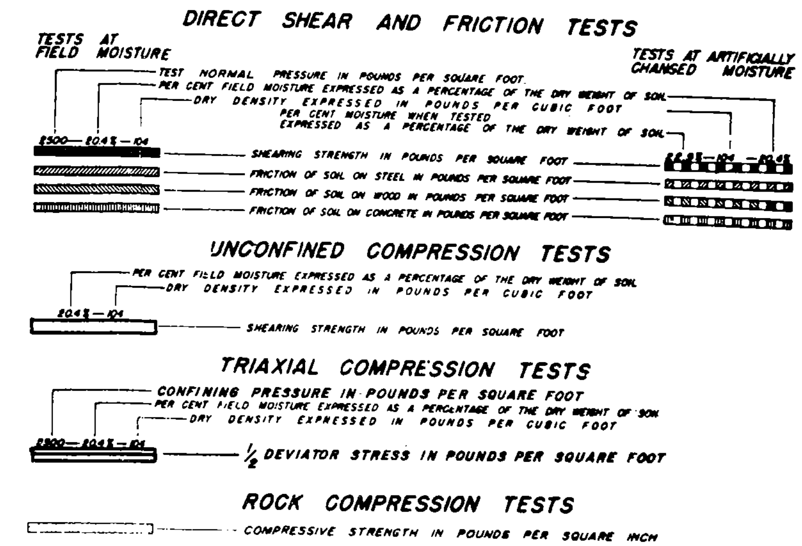
Log of Borings - Boring 52

Figure 2.5-82

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

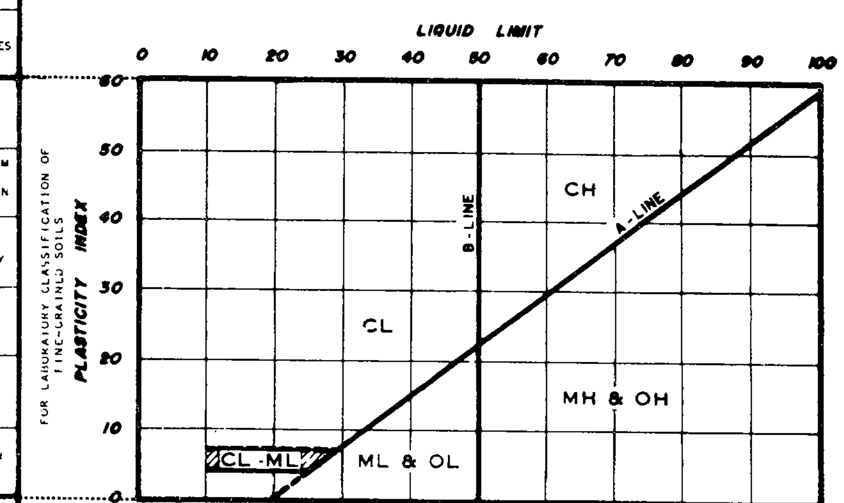
SOIL CLASSIFICATION CHART



KEY TO TEST DATA

- ■ INDICATES DEPTH OF UNDISTURBED SAMPLE
- ⊠ INDICATES DEPTH OF DISTURBED SAMPLE
- □ INDICATES DEPTH OF SAMPLING ATTEMPT WITH NO RECOVERY
- ◪ INDICATES DEPTH OF SPLIT-SPOON SAMPLE
- I INDICATES DEPTH AND LENGTH OF CORING RUN

KEY TO SAMPLES



PLASTICITY CHART

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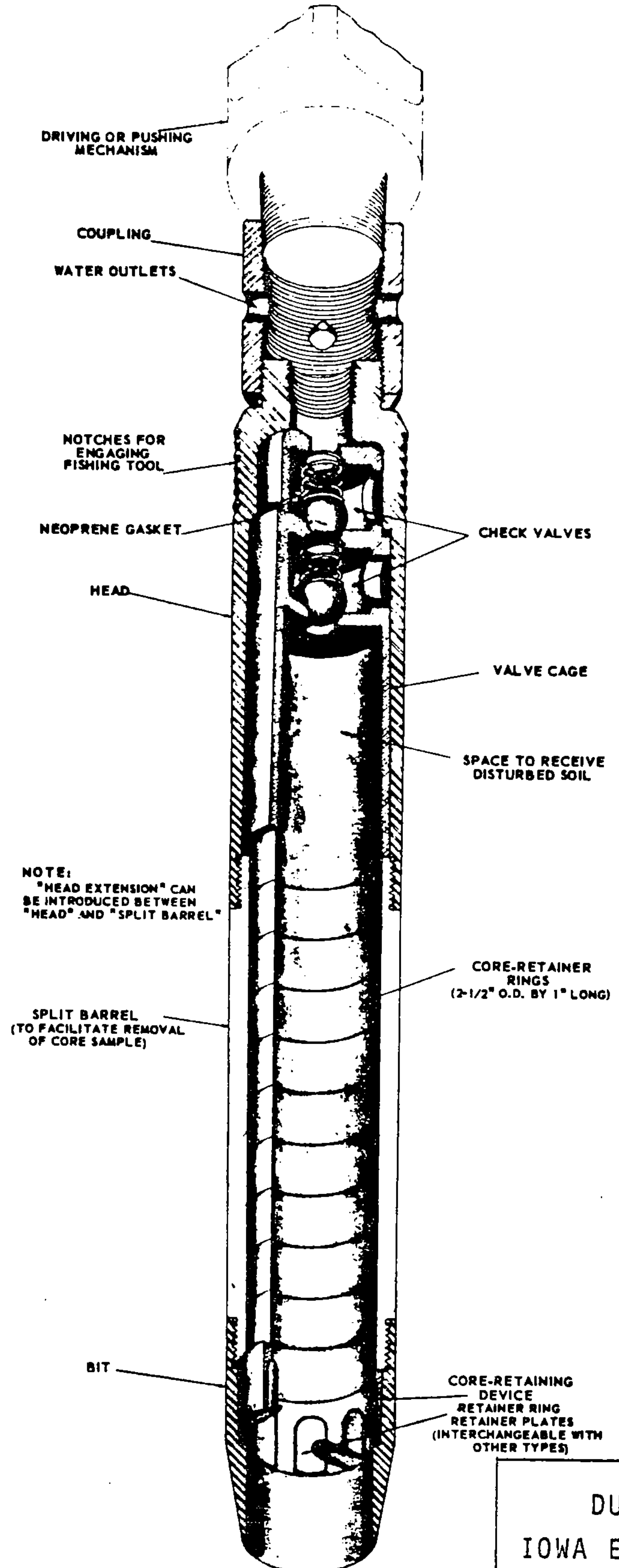
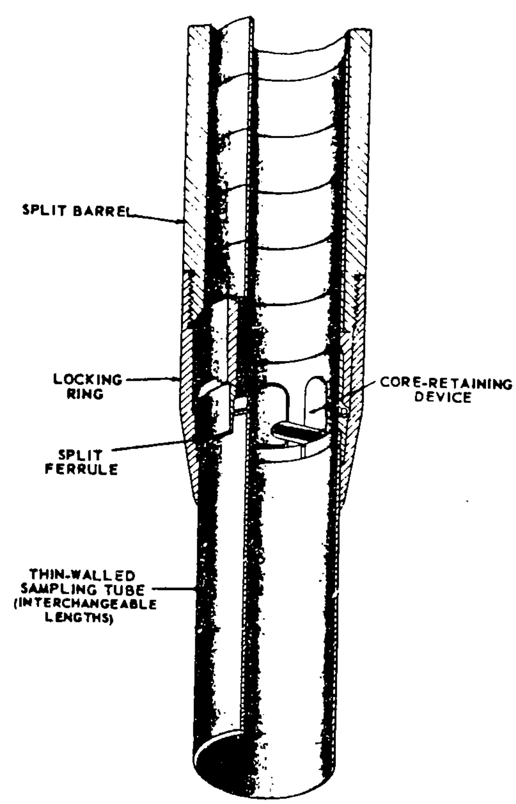
Unified Soil Classification System

Figure 2.5-83



ALTERNATE ATTACHMENTS



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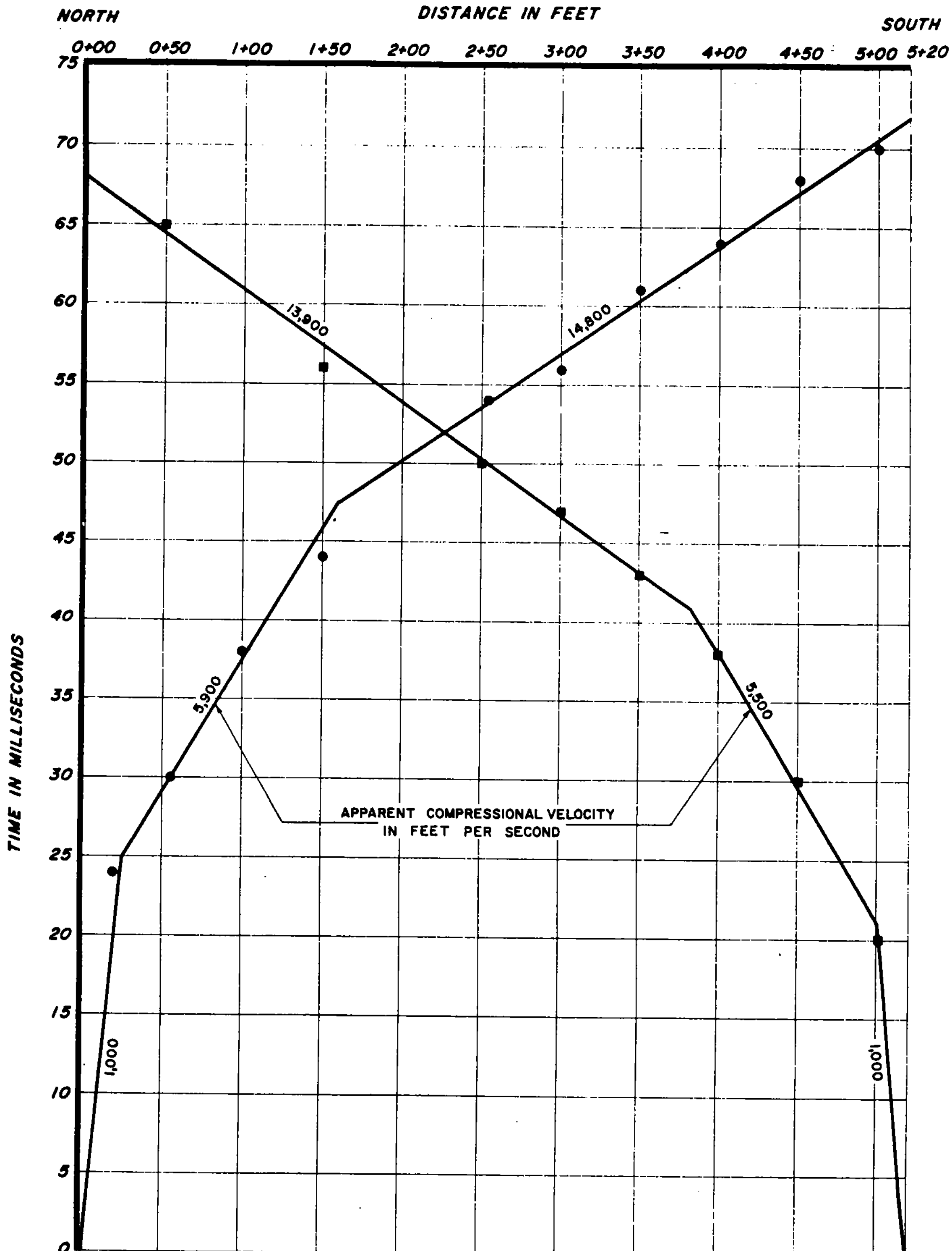
Soil Sampler Type U

Figure 2.5-84



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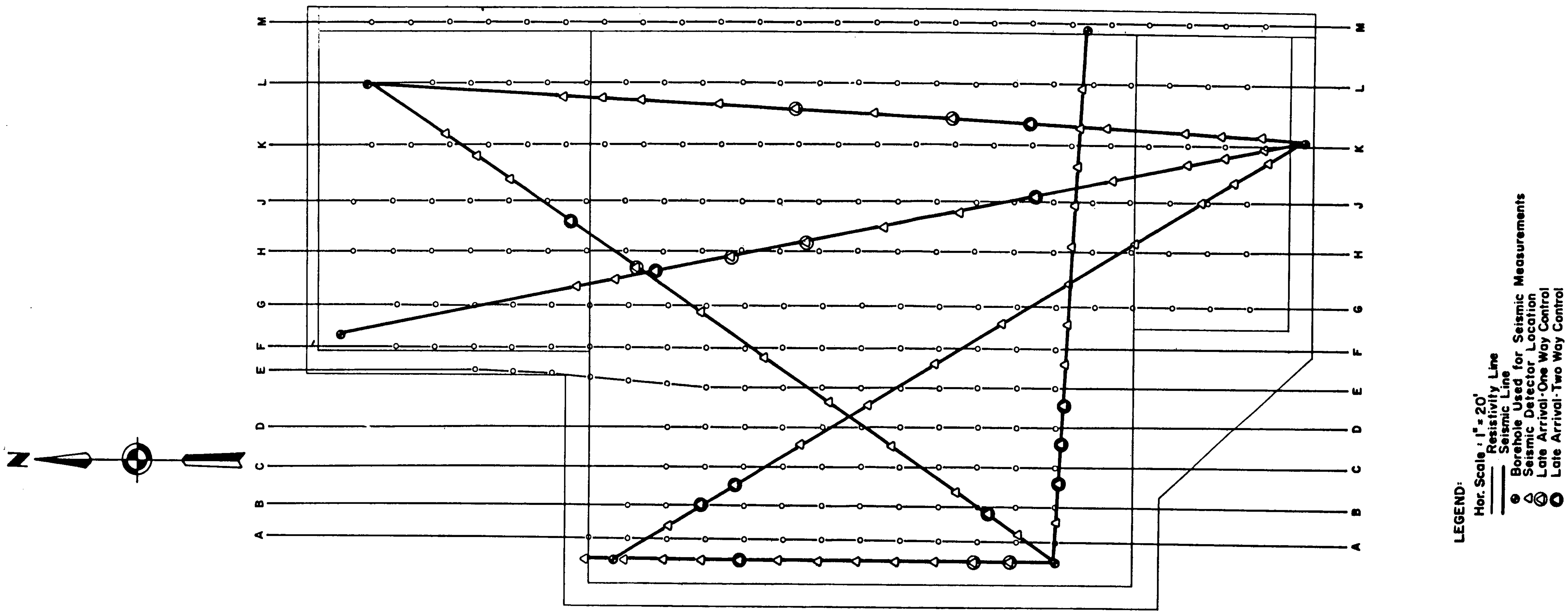
Seismic Refraction Line

Figure 2.5-85



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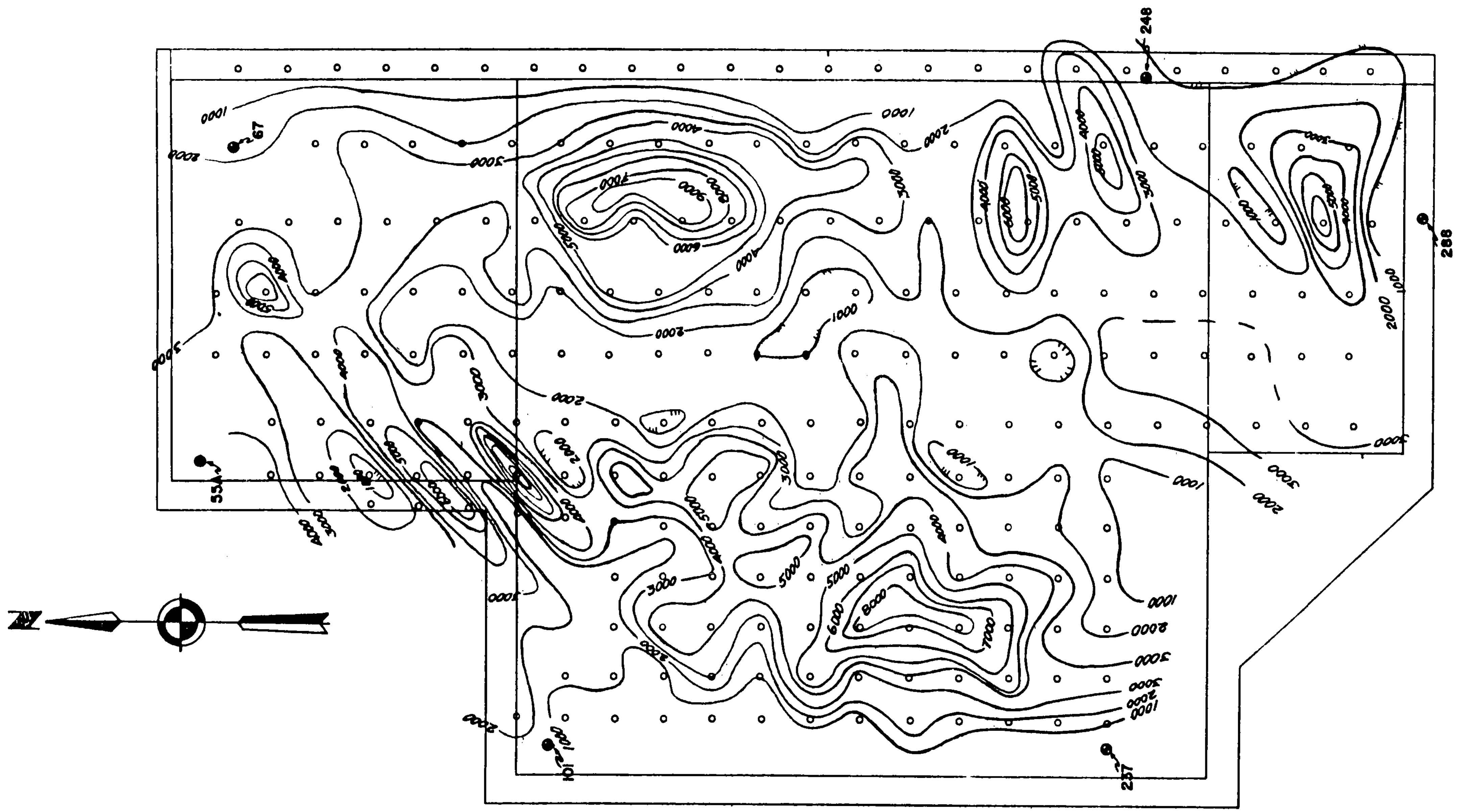
Seismic Lines and Resistivity Survey

Figure 2.5-86



LEGEND:
Contour Interval: 1000 ohm-feet
Hor. Scale: 1" = 20'
● Borehole Used for Seismic Measurements
○ Center of Individual Electrode Configurations

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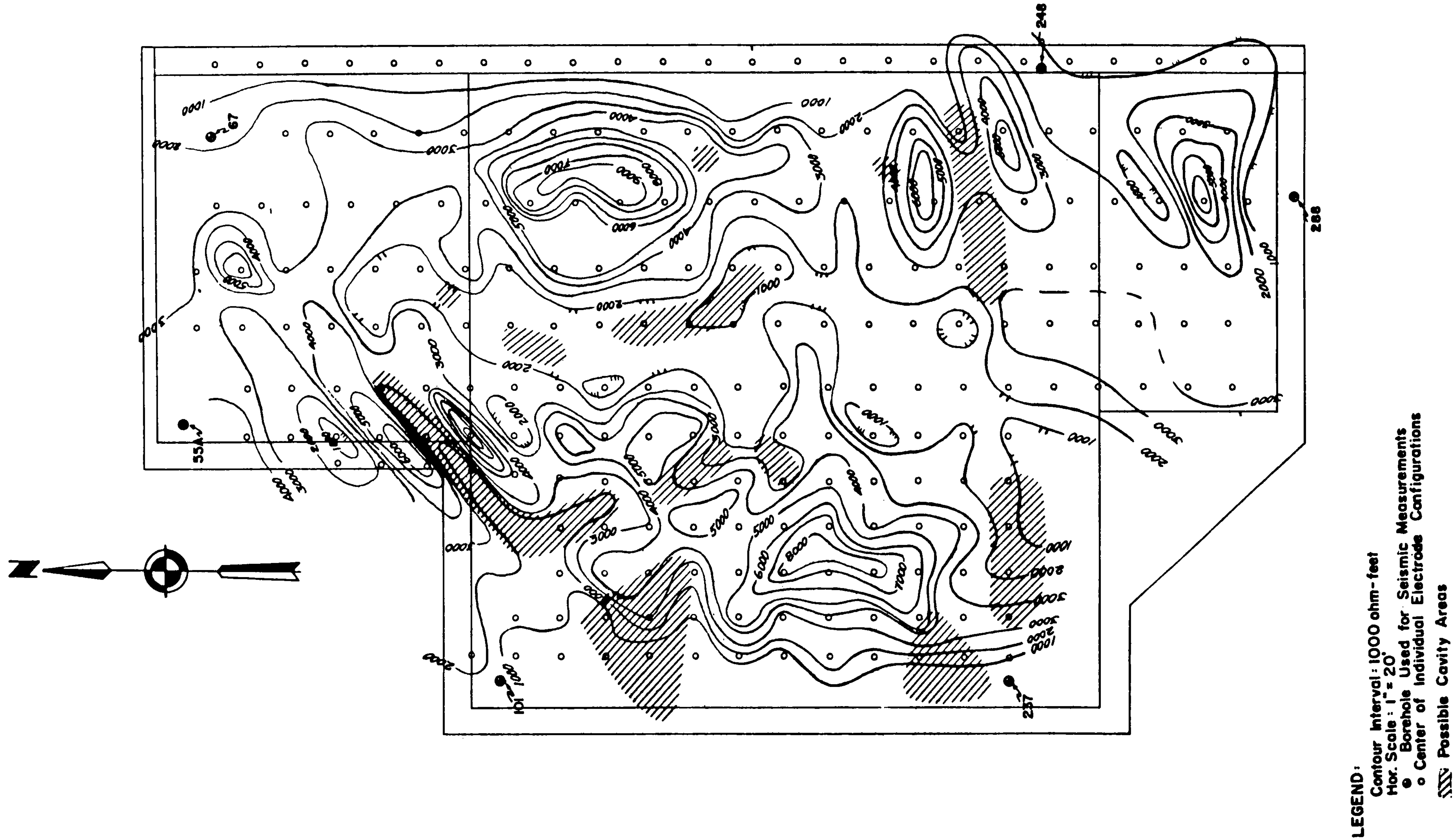
Resistivity Survey

Figure 2.5-87



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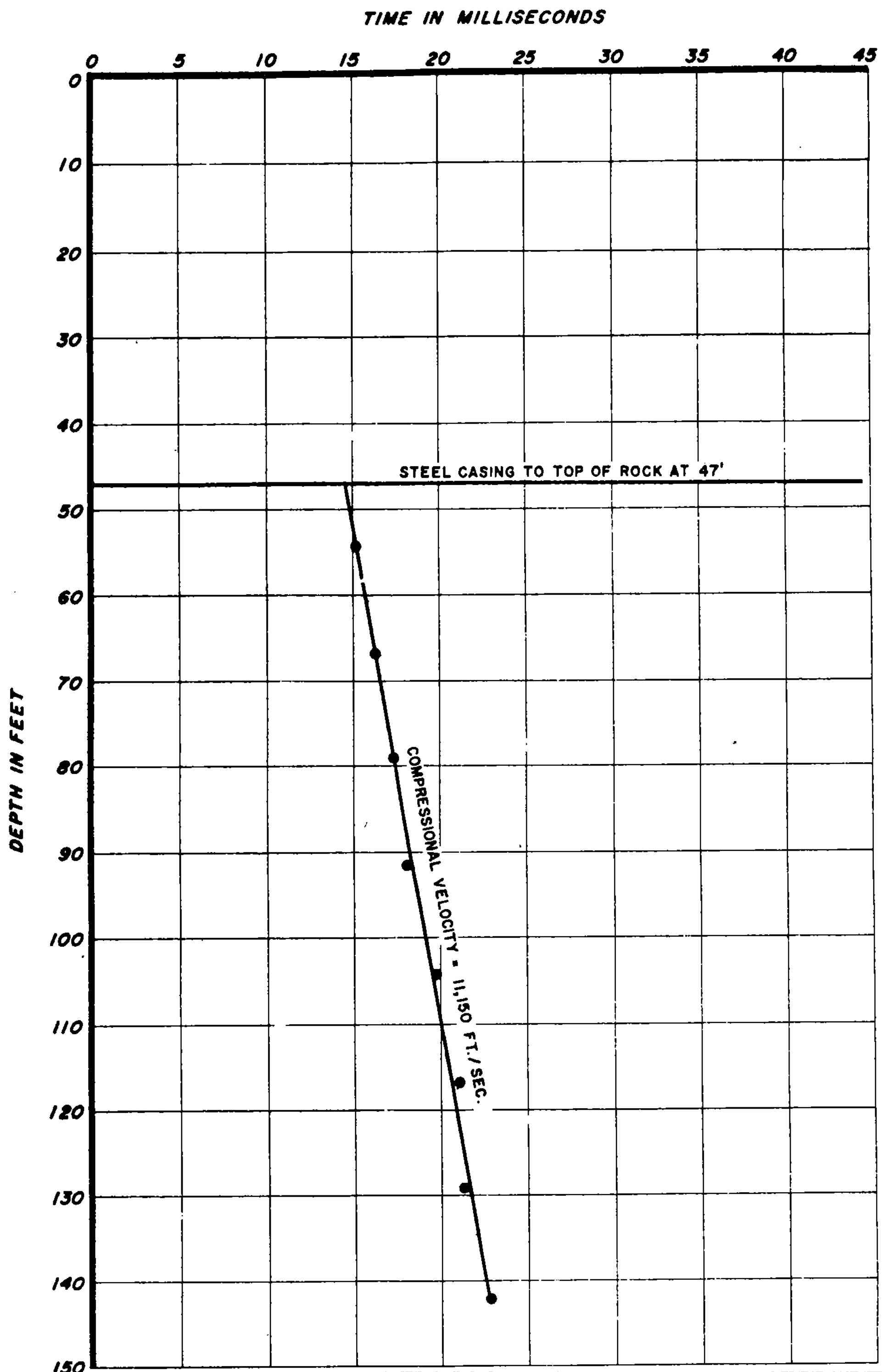
Resistivity Survey

Figure 2.5-88



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Uphole Velocity Survey Boring 12

Figure 2.5-89

DIRECT SHEAR TESTS ARE PERFORMED TO DETERMINE THE SHEARING STRENGTHS OF SOILS. FRICTION TESTS ARE PERFORMED TO DETERMINE THE FRICTIONAL RESISTANCES BETWEEN SOILS AND VARIOUS OTHER MATERIALS SUCH AS WOOD, STEEL, OR CONCRETE. THE TESTS ARE PERFORMED IN THE LABORATORY TO SIMULATE ANTICIPATED FIELD CONDITIONS.

DIRECT SHEAR TESTING
& RECORDING APPARATUS

EACH SAMPLE IS TESTED WITHIN THREE BRASS RINGS, TWO AND ONE-HALF INCHES IN DIAMETER AND ONE INCH IN LENGTH. UNDISTURBED SAMPLES OF IN-PLACE SOILS ARE TESTED IN RINGS TAKEN FROM THE SAMPLING DEVICE IN WHICH THE SAMPLES WERE OBTAINED. LOOSE SAMPLES OF SOILS TO BE USED IN CONSTRUCTING EARTH FILLS ARE COMPACTED IN RINGS TO PREDETERMINED CONDITIONS AND TESTED.

Direct Shear Tests

A THREE-INCH LENGTH OF THE SAMPLE IS TESTED IN DIRECT DOUBLE SHEAR. A CONSTANT PRESSURE, APPROPRIATE TO THE CONDITIONS OF THE PROBLEM FOR WHICH THE TEST IS BEING PERFORMED, IS APPLIED NORMAL TO THE ENDS OF THE SAMPLE THROUGH POROUS STONES. A SHEARING FAILURE OF THE SAMPLE IS CAUSED BY MOVING THE CENTER RING IN A DIRECTION PERPENDICULAR TO THE AXIS OF THE SAMPLE. TRANSVERSE MOVEMENT OF THE OUTER RINGS IS PREVENTED.

THE SHEARING FAILURE MAY BE ACCOMPLISHED BY APPLYING TO THE CENTER RING EITHER A CONSTANT RATE OF LOAD, A CONSTANT RATE OF DEFLECTION, OR INCREMENTS OF LOAD OR DEFLECTION. IN EACH CASE, THE SHEARING LOAD AND THE DEFLECTIONS IN BOTH THE AXIAL AND TRANSVERSE DIRECTIONS ARE RECORDED AND PLOTTED. THE SHEARING STRENGTH OF THE SOIL IS DETERMINED FROM THE RESULTING LOAD-DEFLECTION CURVES.

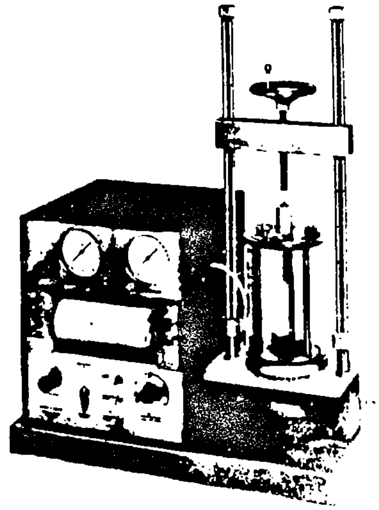
Friction Tests

IN ORDER TO DETERMINE THE FRICTIONAL RESISTANCE BETWEEN SOIL AND THE SURFACES OF VARIOUS MATERIALS, THE CENTER RING OF SOIL IN THE DIRECT SHEAR TEST IS REPLACED BY A DISK OF THE MATERIAL TO BE TESTED. THE TEST IS THEN PERFORMED IN THE SAME MANNER AS THE DIRECT SHEAR TEST BY FORCING THE DISK OF MATERIAL FROM THE SOIL SURFACES.

DUANE ARNOLD ENERGY CENTER IOWA ELECTRIC LIGHT & POWER COMPANY UPDATED FINAL SAFETY ANALYSIS REPORT
Method of Performing Direct Shear and Friction Tests Figure 2.5-90

THE SHEARING STRENGTHS OF SOILS ARE DETERMINED FROM THE RESULTS OF UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS. IN TRIAXIAL COMPRESSION TESTS THE TEST METHOD AND THE MAGNITUDE OF THE CONFINING PRESSURE ARE CHOSEN TO SIMULATE ANTICIPATED FIELD CONDITIONS.

UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS ARE PERFORMED ON UNDISTURBED OR REMOLDED SAMPLES OF SOIL APPROXIMATELY SIX INCHES IN LENGTH AND TWO AND ONE-HALF INCHES IN DIAMETER. THE TESTS ARE RUN EITHER STRAIN-CONTROLLED OR STRESS-CONTROLLED. IN A STRAIN-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO A CONSTANT RATE OF DEFLECTION AND THE RESULTING STRESSES ARE RECORDED. IN A STRESS-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO EQUAL INCREMENTS OF LOAD WITH EACH INCREMENT BEING MAINTAINED UNTIL AN EQUILIBRIUM CONDITION WITH RESPECT TO STRAIN IS ACHIEVED.



TRIAXIAL COMPRESSION TEST UNIT

YIELD, PEAK, OR ULTIMATE STRESSES ARE DETERMINED FROM THE STRESS-STRAIN PLOT FOR EACH SAMPLE AND THE PRINCIPAL STRESSES ARE EVALUATED. THE PRINCIPAL STRESSES ARE PLOTTED ON A MOHR'S CIRCLE DIAGRAM TO DETERMINE THE SHEARING STRENGTH OF THE SOIL TYPE BEING TESTED.

UNCONFINED COMPRESSION TESTS CAN BE PERFORMED ONLY ON SAMPLES WITH SUFFICIENT COHESION SO THAT THE SOIL WILL STAND AS AN UNSUPPORTED CYLINDER. THESE TESTS MAY BE RUN AT NATURAL MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SOILS.

IN A TRIAXIAL COMPRESSION TEST THE SAMPLE IS ENCASED IN A RUBBER MEMBRANE, PLACED IN A TEST CHAMBER, AND SUBJECTED TO A CONFINING PRESSURE THROUGHOUT THE DURATION OF THE TEST. NORMALLY, THIS CONFINING PRESSURE IS MAINTAINED AT A CONSTANT LEVEL, ALTHOUGH FOR SPECIAL TESTS IT MAY BE VARIED IN RELATION TO THE MEASURED STRESSES. TRIAXIAL COMPRESSION TESTS MAY BE RUN ON SOILS AT FIELD MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SAMPLES. THE TESTS ARE PERFORMED IN ONE OF THE FOLLOWING WAYS:

UNCONSOLIDATED-UNDRAINED: THE CONFINING PRESSURE IS IMPOSED ON THE SAMPLE AT THE START OF THE TEST. NO DRAINAGE IS PERMITTED AND THE STRESSES WHICH ARE MEASURED REPRESENT THE SUM OF THE INTERGRANULAR STRESSES AND PORE WATER PRESSURES.

CONSOLIDATED-UNDRAINED: THE SAMPLE IS ALLOWED TO CONSOLIDATE FULLY UNDER THE APPLIED CONFINING PRESSURE PRIOR TO THE START OF THE TEST. THE VOLUME CHANGE IS DETERMINED BY MEASURING THE WATER AND OR AIR EXPELLED DURING CONSOLIDATION. NO DRAINAGE IS PERMITTED DURING THE TEST AND THE STRESSES WHICH ARE MEASURED ARE THE SAME AS FOR THE UNCONSOLIDATED-UNDRAINED TEST.

DRAINED: THE INTERGRANULAR STRESSES IN A SAMPLE MAY BE MEASURED BY PERFORMING A DRAINED, OR SLOW, TEST. IN THIS TEST THE SAMPLE IS FULLY SATURATED AND CONSOLIDATED PRIOR TO THE START OF THE TEST. DURING THE TEST, DRAINAGE IS PERMITTED AND THE TEST IS PERFORMED AT A SLOW ENOUGH RATE TO PREVENT THE BUILDUP OF PORE WATER PRESSURES. THE RESULTING STRESSES WHICH ARE MEASURED REPRESENT ONLY THE INTERGRANULAR STRESSES. THESE TESTS ARE USUALLY PERFORMED ON SAMPLES OF GENERALLY NON-COHESIVE SOILS, ALTHOUGH THE TEST PROCEDURE IS APPLICABLE TO COHESIVE SOILS IF A SUFFICIENTLY SLOW TEST RATE IS USED.

AN ALTERNATE MEANS OF OBTAINING THE DATA RESULTING FROM THE DRAINED TEST IS TO PERFORM AN UNDRAINED TEST IN WHICH SPECIAL EQUIPMENT IS USED TO MEASURE THE PORE WATER PRESSURES. THE DIFFERENCES BETWEEN THE TOTAL STRESSES AND THE PORE WATER PRESSURES MEASURED ARE THE INTERGRANULAR STRESSES.

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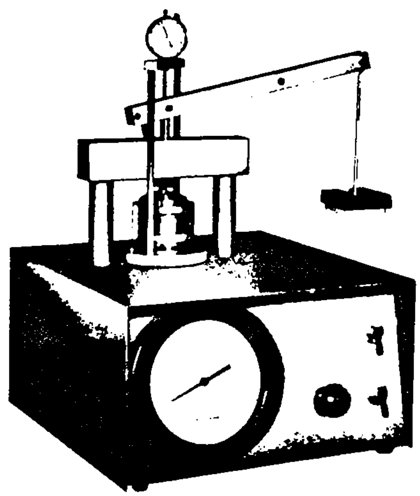
Method of Performing Unconfined Compression and Triaxial Compression Tests

Figure 2.5-91

CONSOLIDATION TESTS ARE PERFORMED TO EVALUATE THE VOLUME CHANGES OF SOILS SUBJECTED TO INCREASED LOADS. TIME-CONSOLIDATION AND PRESSURE-CONSOLIDATION CURVES MAY BE PLOTTED FROM THE DATA OBTAINED IN THE TESTS. ENGINEERING ANALYSES BASED ON THESE CURVES PERMIT ESTIMATES TO BE MADE OF THE PROBABLE MAGNITUDE AND RATE OF SETTLEMENT OF THE TESTED SOILS UNDER APPLIED LOADS.

EACH SAMPLE IS TESTED WITHIN BRASS RINGS TWO AND ONE-HALF INCHES IN DIAMETER AND ONE INCH IN LENGTH. UNDISTURBED SAMPLES OF IN-PLACE SOILS ARE TESTED IN RINGS TAKEN FROM THE SAMPLING DEVICE IN WHICH THE SAMPLES WERE OBTAINED. LOOSE SAMPLES OF SOILS TO BE USED IN CONSTRUCTING EARTH FILLS ARE COMPACTED IN RINGS TO PREDETERMINED CONDITIONS AND TESTED.



DEAD LOAD-PNEUMATIC CONSOLIDOMETER

IN TESTING, THE SAMPLE IS RIGIDLY CONFINED LATERALLY BY THE BRASS RING. AXIAL LOADS ARE TRANSMITTED TO THE ENDS OF THE SAMPLE BY POROUS DISKS. THE DISKS ALLOW DRAINAGE OF THE LOADED SAMPLE. THE AXIAL COMPRESSION OR EXPANSION OF THE SAMPLE IS MEASURED BY A MICROMETER DIAL INDICATOR AT APPROPRIATE TIME INTERVALS AFTER EACH LOAD INCREMENT IS APPLIED. EACH LOAD IS ORDINARILY TWICE THE PRECEDING LOAD. THE INCREMENTS ARE SELECTED TO OBTAIN CONSOLIDATION DATA REPRESENTING THE FIELD LOADING CONDITIONS FOR WHICH THE TEST IS BEING PERFORMED. EACH LOAD INCREMENT IS ALLOWED TO ACT OVER AN INTERVAL OF TIME DEPENDENT ON THE TYPE AND EXTENT OF THE SOIL IN THE FIELD.

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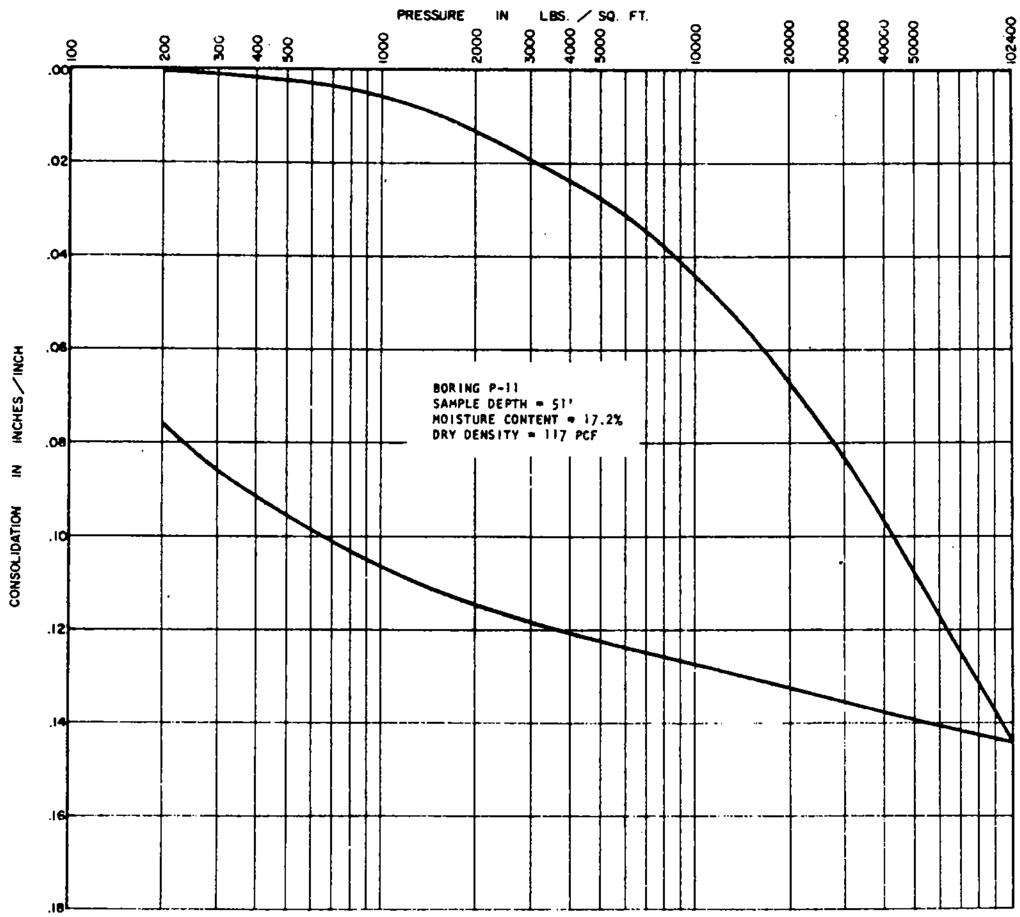
Method of Performing Consolidation Tests

Figure 2.5-92



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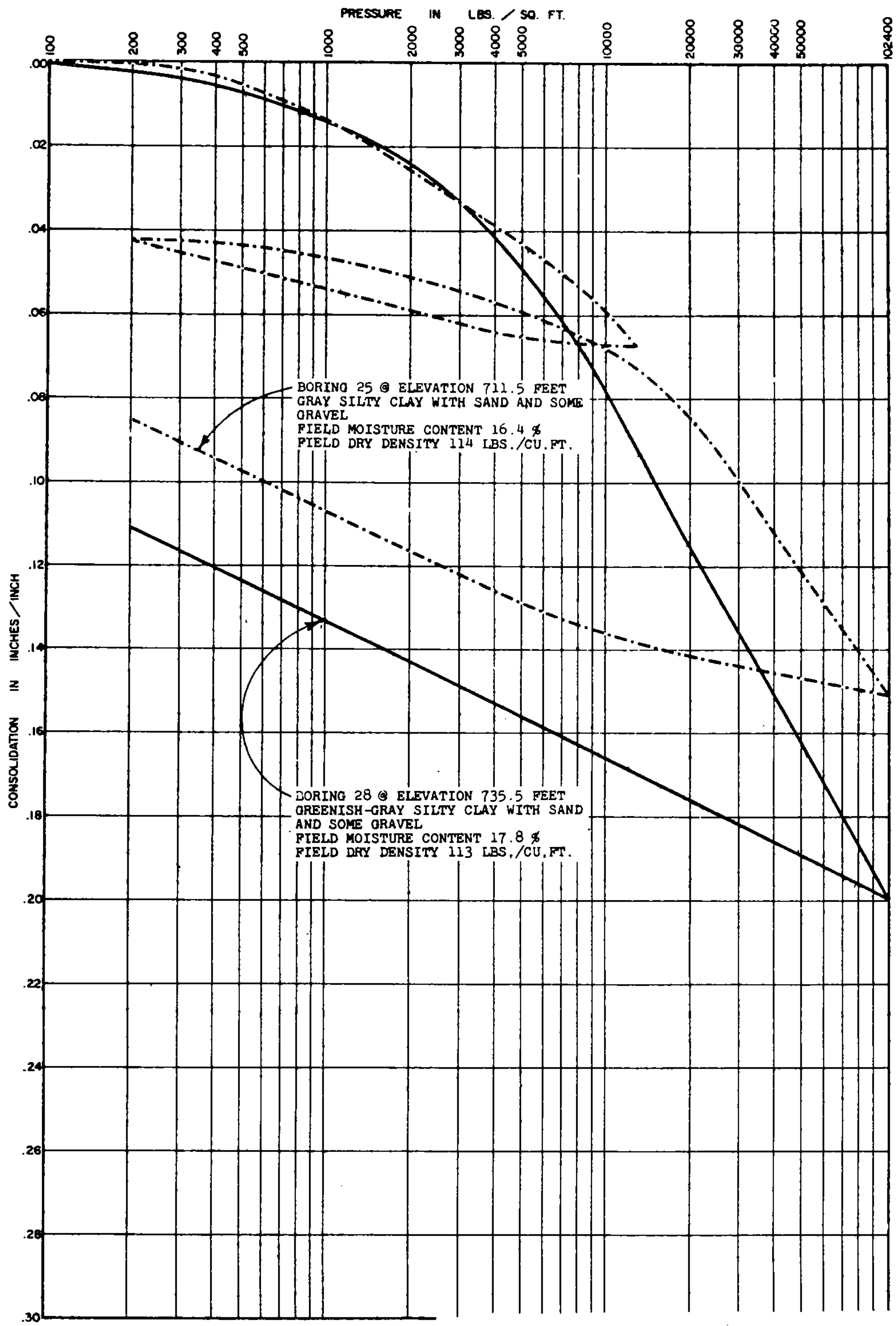
Consolidation Test Data

Figure 2.5-93



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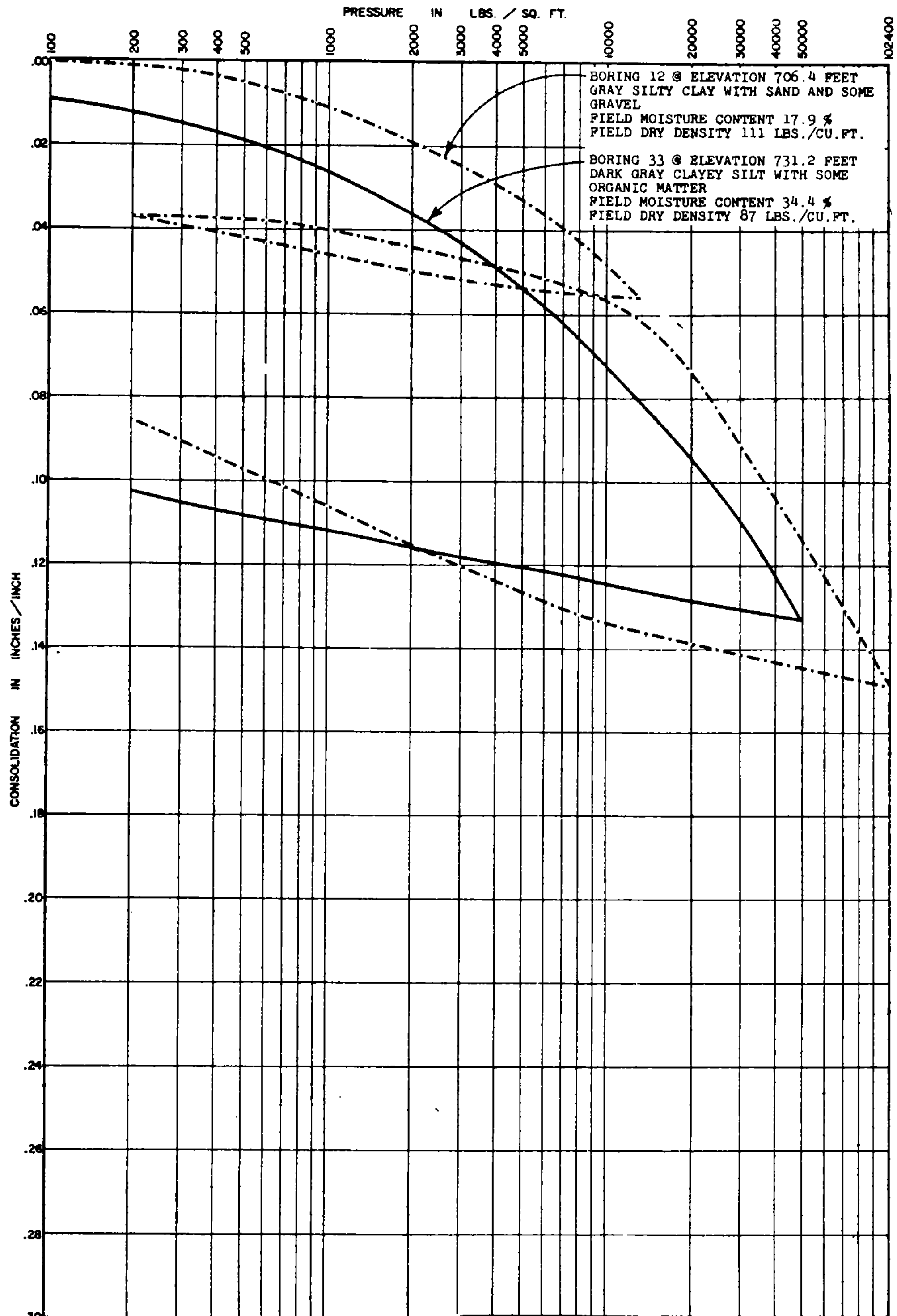
Consolidation Test Data

Figure 2.5-94



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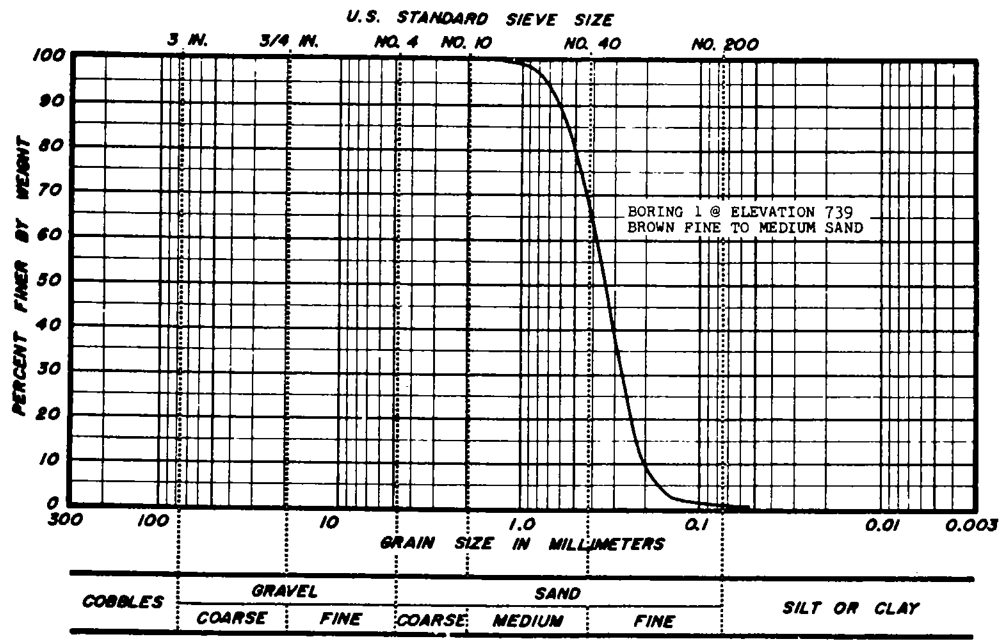
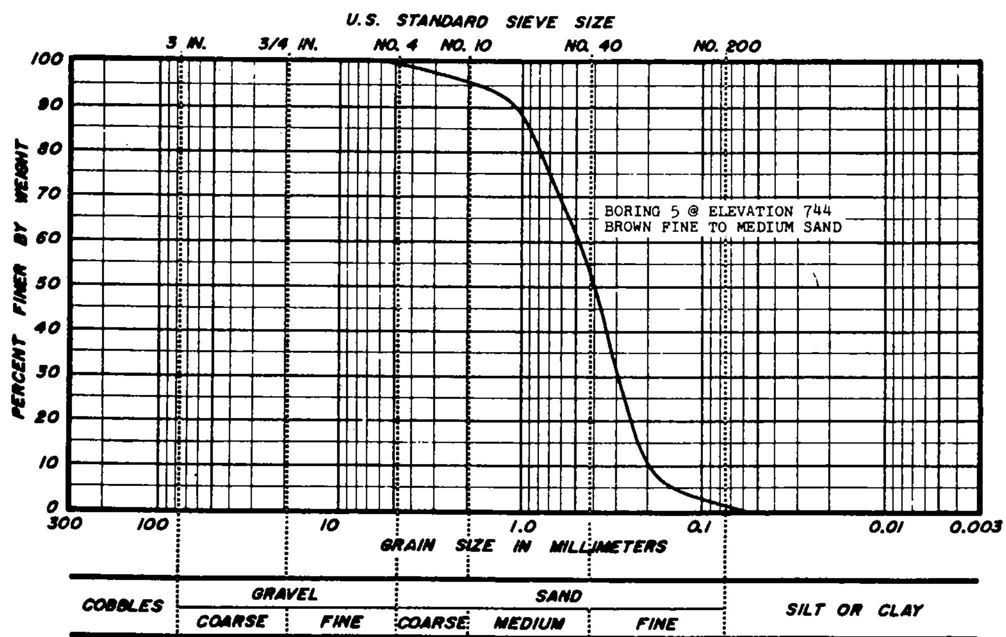
Consolidation Test Data

Figure 2.5-95





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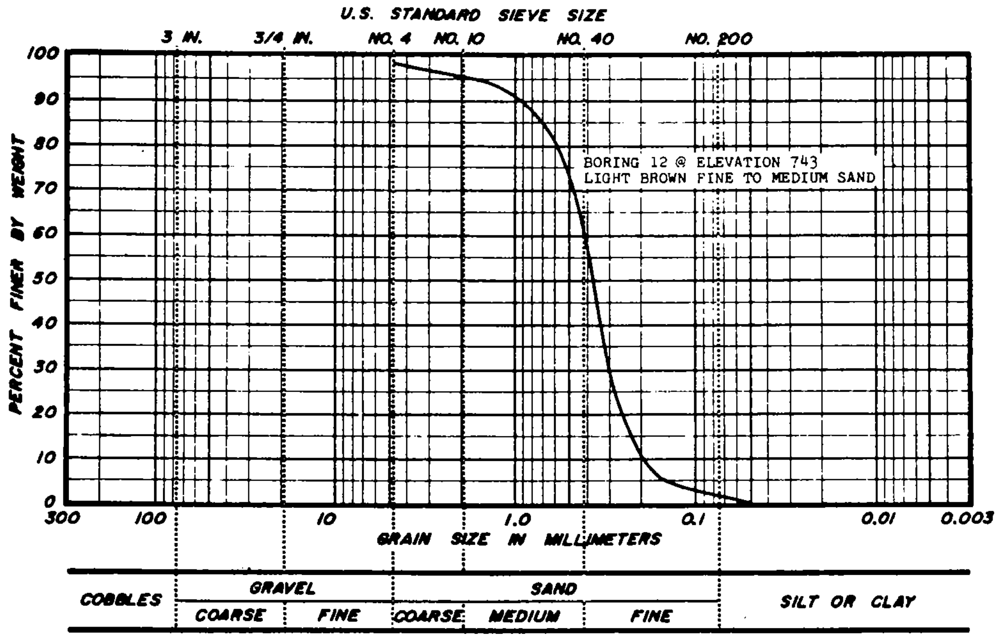
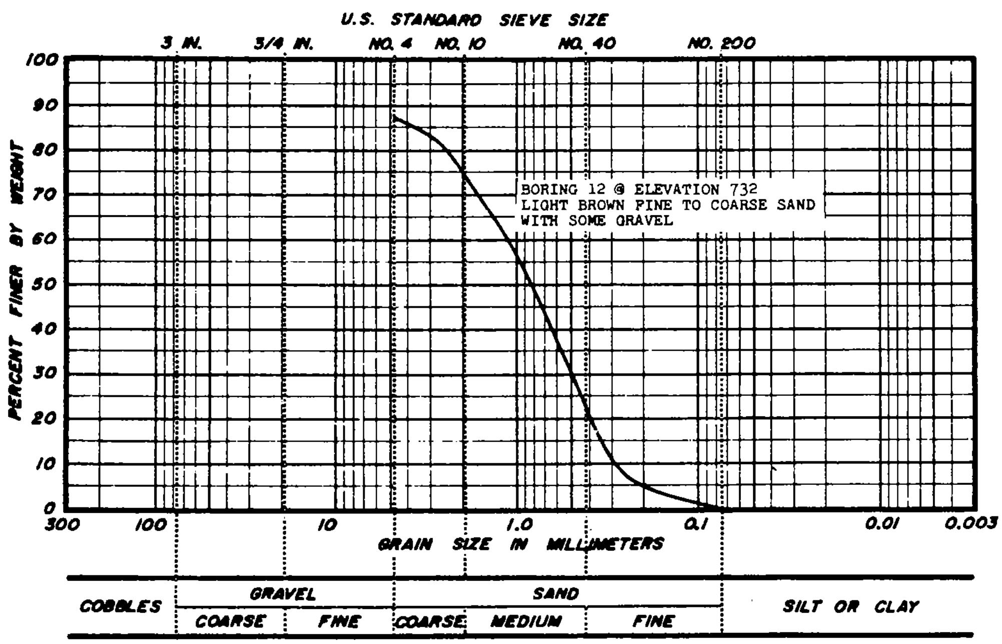
Particle Size Analyses

Figure 2.5-96





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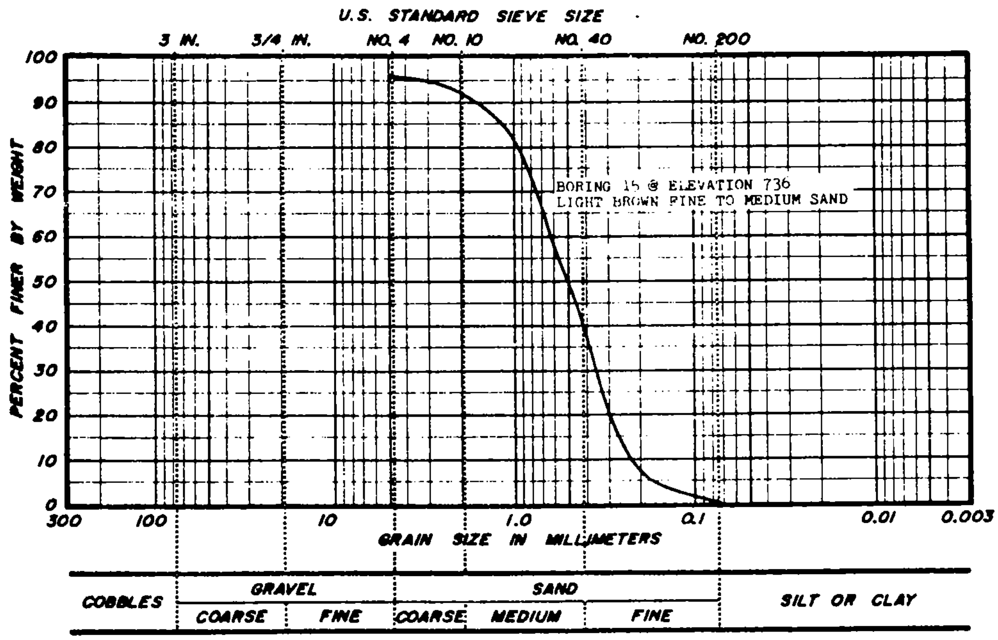
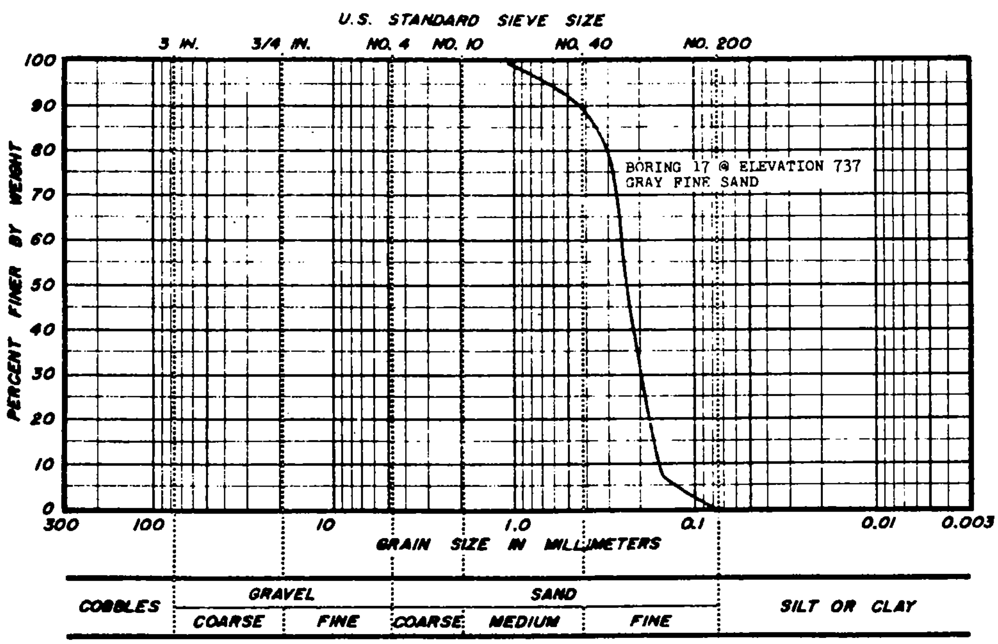
Particle Size Analyses

Figure 2.5-97





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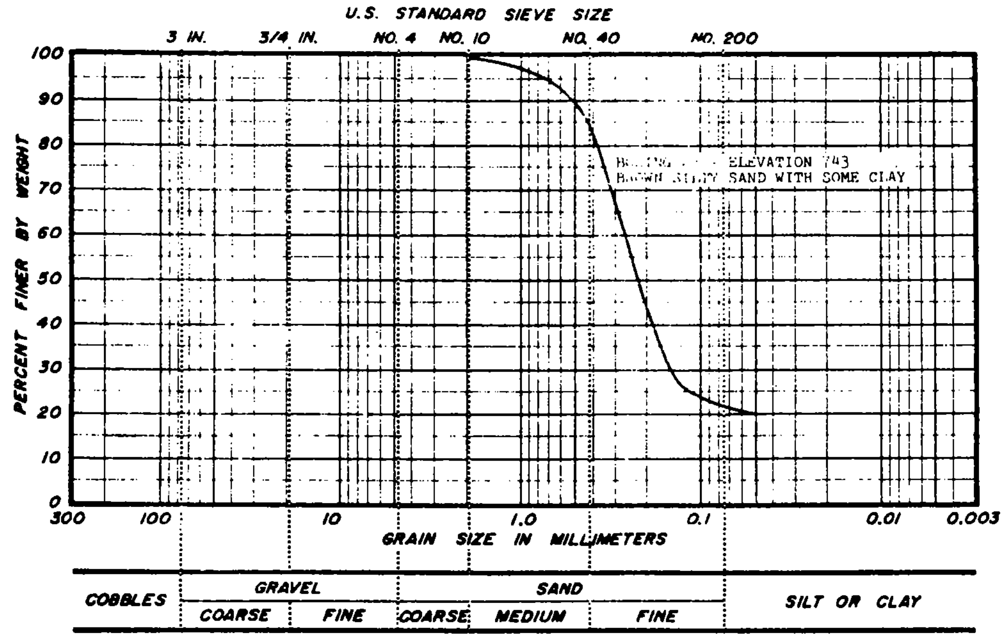
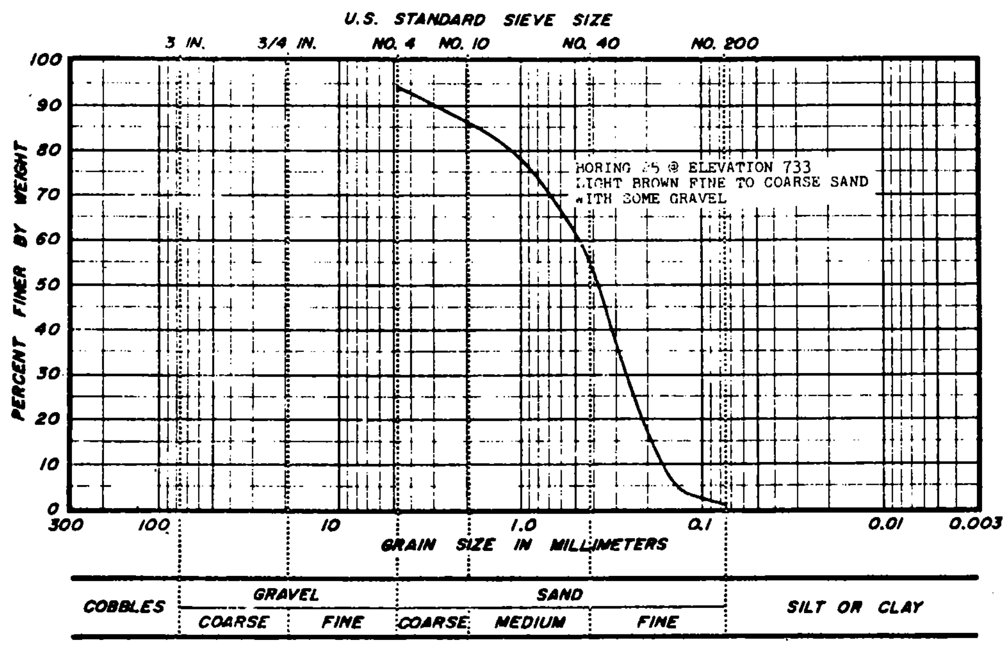
Particle Size Analyses

Figure 2.5-98





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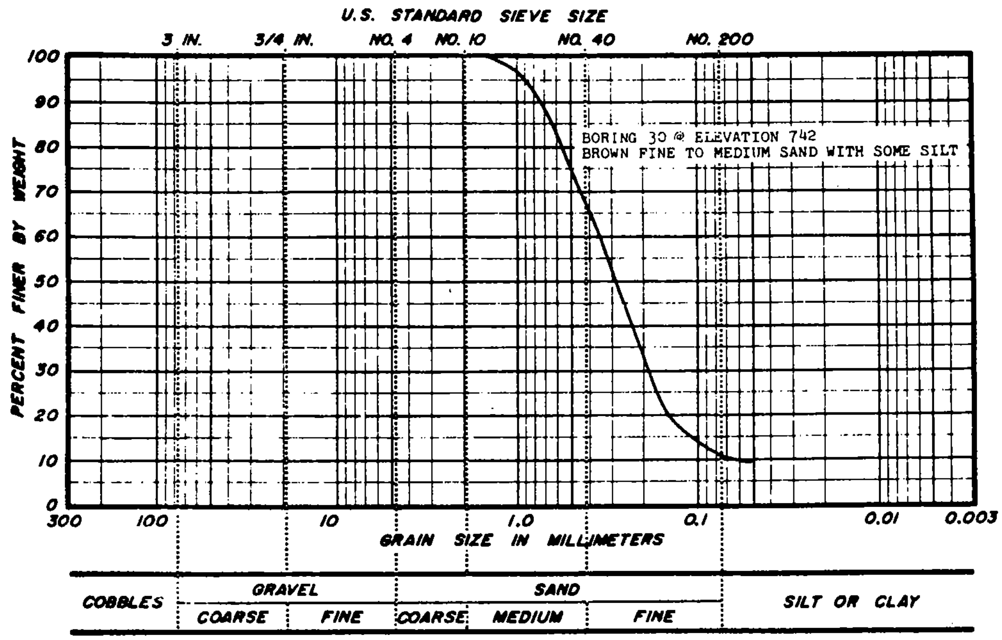
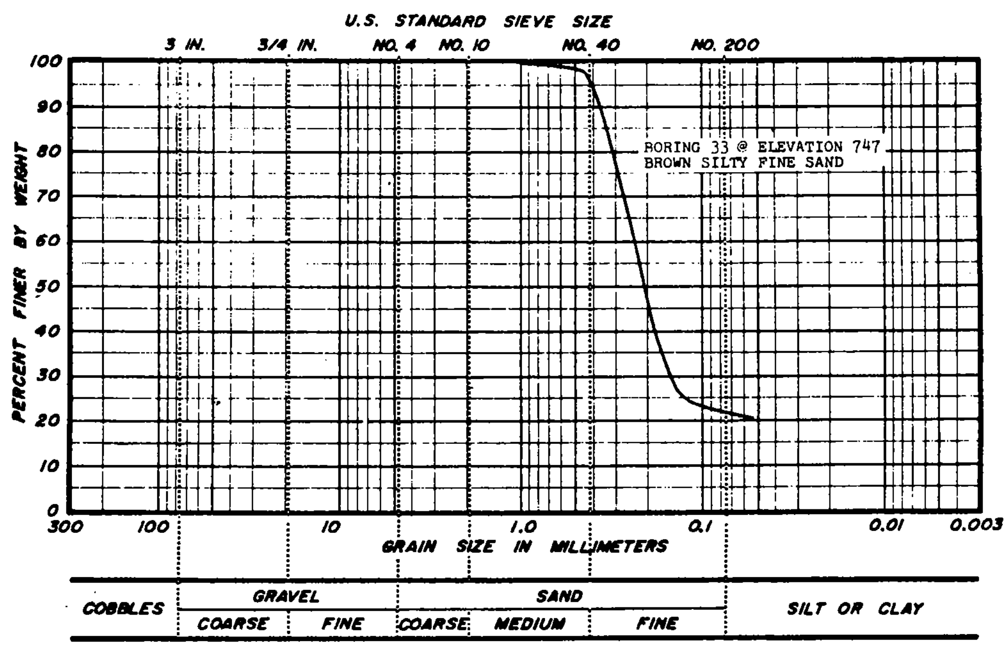
Particle Size Analyses

Figure 2.5-99





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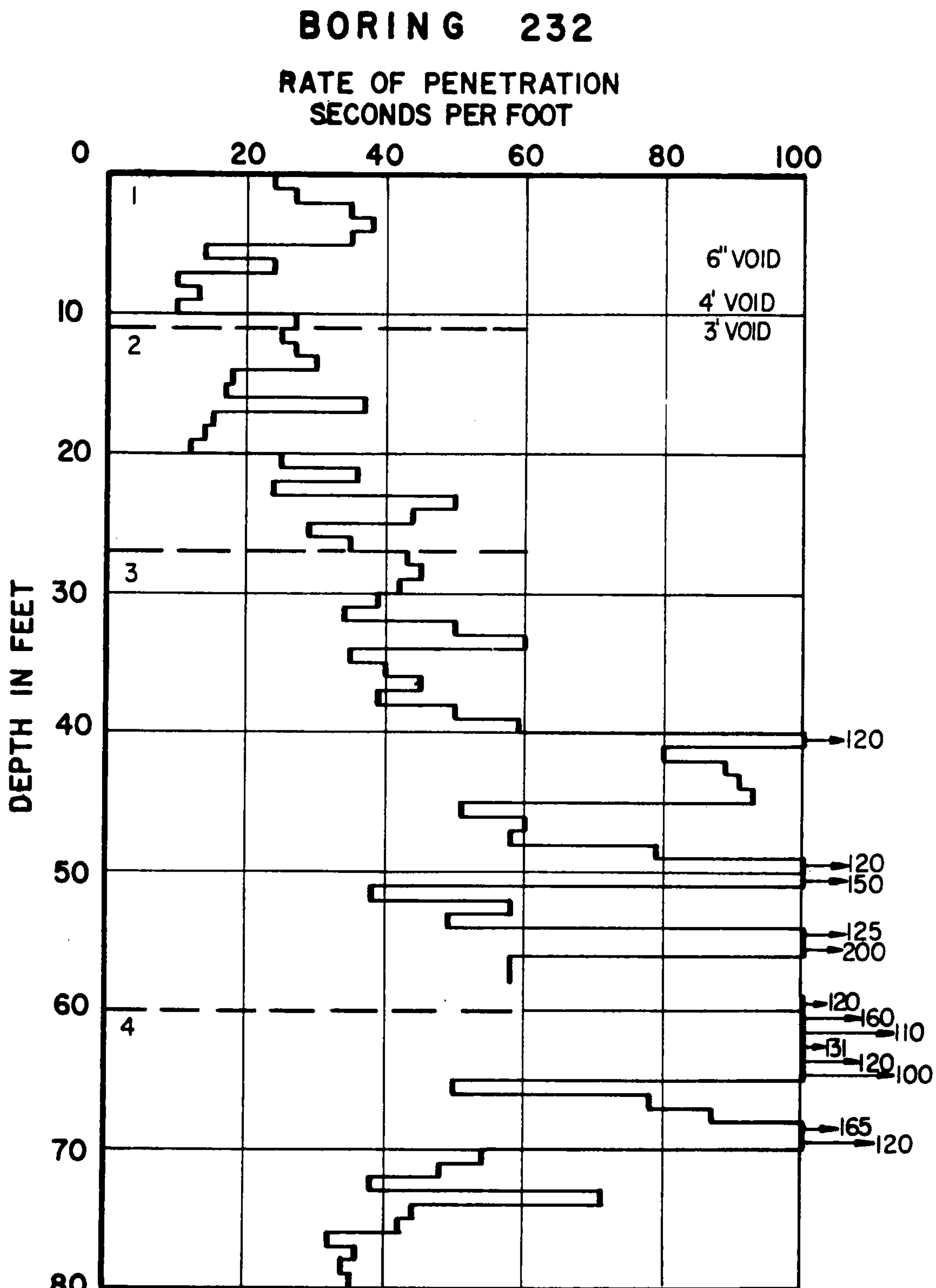
Particle Size Analyses

Figure 2.5-100



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Typical Probing Log

Figure 2.5-101

Project: Iowa Electric Light & Power/DAEC
Cedar Rapids, Iowa

Boring No. RW-1 **Date of Boring** 9/26/84 **Project No.** 184312-0 **Sheet** 1 **of** 2

Surface Elevation = 756.2±

Sample No. and Method	Blows/Foot (N)	Unconfined Compressive Strength (TSF)	Penetrometer Value (TSF)	Moisture Content (%)	Dry Unit Weight (PCF)	Unified System Symbol	Depth (Feet)	Description	Water Table Observations
							0–1	Roadstone	
1-SS	57	-	-	4.3	-	SP	5	Gray to Brown Medium Sand Trace Clay, Trace Gravel Fine from 10.5 to 13.0 Reddish from 13.0 to 17.0,	
2-SS	46	-	-	2.7	-	SP			
3-SS	11	-	1.00	8.2	-	SP	10		
4-SS	12	-	-	13.0	-	SP		Very Dense to Loose, Looser With Depth	WD
5-SS	8	-	-	-	-	SP	15		
6-SS	12	-	-	32.1	-	CL	20	Brown Sandy Silty Clay Trace Gravel Stiff	
7-SS	18	-	-	-	-	SP	25	Gray Medium Sand, Fine Below 27.0, Medium Dense	
8-SS	[illegible]	-	-	-	-	SP	30		
9-SS	9	1.24	0.75	40.1	-	ML-CL	35	Gray Clayey Silt, With Fine Sand Below 37.0, Firm to Stiff With Sand Seams	
10-ST	-	-	0.50	25.3	-	ML-CL	40		

Continued on Sheet 2 of 2

Remarks:

Note: All Shelby tube (ST) unconfined compression tests performed in the laboratory.

SHIVE-HATTERY ENGINEERS
Record of Subsurface Exploration

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UPDATED FINAL SAFETY ANALYSIS REPORT

Log of Borings - Boring RW-1

Figure 2.5-102 (Sheet 1)

Project: Iowa Electric Light & Power/DAEC
Cedar Rapids, Iowa

Boring No. RW-1 **Date of Boring** 9/26/84 **Project No.** 184312-G **Sheet** 2 **of** 2

Sample No. and Method	Blows/Foot (N)	Unconfined Compressive Strength (TSF)	Penetrometer Value (TSF)	Moisture Content (%)	Dry Unit Weight (PCF)	Unified System Symbol	Depth (Feet)	Description (Surface)	Water Table Observations
							40	Continued From Sheet 1 of 2	
11-SS	6	0.39	0.50	25.9	-	CL	45	Gray Silty Clay Trace Fine Sand, Firm to Stiff With Sand Seams	
12-SS	13	1.75	1.50	16.8	-	CL	50		
13-SS	100/3"	-		25.9	-				
							55	Moderately Weathered Limestone	
							60	Bottom of Boring @ 59.7' Boring Backfilled 9/26/84	
							65		
							70		
							75		

Remarks:

Note: All Shelby tube (ST) unconfined compression tests performed in the laboratory.

SHIVE-HATTERY ENGINEERS
Record of Subsurface Exploration

DUANE ARNOLD ENERGY CENTER
IOWA ELECTRIC LIGHT & POWER COMPANY
UPDATED FINAL SAFETY ANALYSIS REPORT

Log of Borings - Boring RW-1

Figure 2.5-102 (Sheet 2)

Project: Iowa Electric Light & Power/DAEC
Cedar Rapids, Iowa

Boring No. RW-2 Date of Boring 9/25/84 Project No. 184312-0 Sheet 1 of 2

Sample No. and Method	Blows/Foot (N)	Unconfined Compressive Strength (TSF)	Penetrometer Value (TSF)	Moisture Content (%)	Dry Unit Weight (PCF)	Unified System Symbol	Depth (Feet)	Description Surface Elevation = 755.2±	Water Table Observations
								Roadstones	
								Brown Sandy Silty Clay Trace Gravel, Hard	
1-SS	33	-	4.50+	7.5	-	CL	5		
2-SS	35	-	-	9.0	-	SW		Brown Sand With Gravel, Dense	
3-SS	18	-	-	8.0	-	SP	10	Brown Fine Sand, Reddish With Clay From 10.5 to 13.0, Medium Dense to Loose	
4-SS	12	-	-	18.0	-	SP			WD.
5-SS	4	-	-	-	-	SP	15		
6-SS	16	-	-	-	-	SP	20		
7-SS	27	-	-	-	-	SP	25	Gray Medium Sand, With Gravel Seams Fine Below 27.0', Medium Dense	
8-SS	10	-	-	-	-	SP	30		
9-SS	7	-	0.50	25.6	-	ML/CL	35	Gray Clayey Silt, With Fine Sand Below 37.0', Firm to Stiff	
10-ST	-					ML/CL	40		

Continued On Sheet 2 of 2

Remarks:

Note: All Shelby tube (ST) unconfined compression tests performed in the laboratory.

SHIVE-HATTERY ENGINEERS
Record of Subsurface Exploration

DUANE ARNOLD ENERGY CENTER
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Log of Borings - Boring RW-2

Figure 2.5-103 (Sheet 1)

Project: Iowa Electric Light & Power/DAEC
Cedar Rapids, Iowa

Boring No. RW-2 **Date of Boring** 9/25/84 **Project No.** 184312-0 **Sheet** 2 **of** 2

Sample No. and Method	Blows/Foot (N)	Unconfined Compressive Strength (TSF)	Penetrometer Value (TSF)	Moisture Content (%)	Dry Unit Weight (PCF)	Unified System Symbol	Depth (Feet)	Description (Surface)	Water Table Observations
							40	Continued From Sheet 1 of 2	
								Gray Sandy Silty Clay, Very Stiff	
11-SS	20	-	1.50	18.5	-	CL	45		
12-SS	21	-	-	-	-	CL	50		
[illegible]	[illegible]	[illegible]		-					
							55	Moderately Weathered Limestone	
							60	Bottom of Boring @ 59.2' Boring Backfilled 9/25/84	
							65		
							70		
							75		

Remarks:

Note: All Shelby tube [ST] unconfined compression tests performed in the laboratory.

SHIVE-HATTERY ENGINEERS
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DUANE ARNOLD ENERGY CENTER
IOWA ELECTRIC LIGHT & POWER COMPANY
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Log of Borings - Boring RW-2

Figure 2.5-103 (Sheet 2)

Project: Iowa Electric Light & Power Company/DAEC
Cedar Rapids, Iowa

Boring No. RW-3 **Date of Boring** 9/26/84 **Project No.** 184312-G **Sheet** 2 **of** 2

Sample No. and Method	Blows/Foot (N)	Unconfined Compressive Strength (TSF)	Penetrometer Value (TSF)	Moisture Content (%)	Dry Unit Weight (PCF)	Unified System Symbol	Depth (Feet)	Description (Surface)	Water Table Observations
								Continued From Sheet 1 of 2	
							40		
11-ST	-	1.09	1.25	19.7	111.8	CL	45	Gray Sandy Silty Clay, Stiff to Very Stiff	
12-SS	15	1.45	1.50	20.9	-	CL	50		
								Moderately Weathered Limestone	
							55		
								Bottom of Boring @ 56.0' Boring Backfilled 9/27/84	
							60		
							65		
							70		
							75		

Remarks:

Note: All Shelby tube (ST) unconfined compression tests performed in the laboratory.

SHIVE-HATTERY ENGINEERS
Record of Subsurface Exploration

DUANE ARNOLD ENERGY CENTER
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Log of Borings - Boring RW-3

Figure 2.5-104 (Sheet 2)

Project: Iowa Electric Light & Power/DAEC
Cedar Rapids, Iowa

Boring No. RW-3 **Date of Boring** 9/27/84 **Project No.** 184312-0 **Sheet** 1 **of** 2

Sample No. and Method	Blows/Foot (N)	Unconfined Compressive Strength (TSF)	Penetrometer Value (TSF)	Moisture Content (%)	Dry Unit Weight (PCF)	Unified System Symbol	Depth (Feet)	Description — Surface Elevation = 752.0±	Water Table Observations
								Brown Sand With Gravel and Clay	
1-SS	12	0.89	4.50	22.1	-	CL	5	Dark Gray Silty Clay, Stiff	
2-ST	-	1.58	1.25	33.9	88.1	CL			
3-SS	5	1.16	0.75	22.7	-	CL	10	Light Gray Sandy Silty Clay Firm to Stiff	
4-SS	7	-	0.25	27.8	-	CL			WD ▼
5-SS	12	-	-	-	-	SC	15	Light Gray Silty Clayey Medium Sand, Medium Dense	
6-SS	34	-	-	-	-	SP	20	Brown Medium Sand Trace Gravel, Dense	
7-SS	39	-	-	-	-	SW	25	Light Gray Sand Trace Gravel, Dense	
8-SS	47	-	-	-	-	SM	30	Dark Gray Clayey Silty Fine Sand, Dense	
9-SS	26	-	-	-	-	SW	35	Gray Sand With Silt, Medium Dense	
10-SS	7	0.38	0.50	24.9	-	ML/CL	40	Gray Clayey Silt, Firm to Stiff	

Continued on Sheet 2 of 2.

Remarks:

Note: All Shelby tube (ST) unconfined compression tests performed in the laboratory.

SHIVE-HATTERY ENGINEERS
Record of Subsurface Exploration

DUANE ARNOLD ENERGY CENTER
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Log of Borings - Boring RW-3

Figure 2.5-104 (Sheet 1)

Project: Iowa Electric Light & Power/DAEC
Cedar Rapids, Iowa

Boring No. RW-4 **Date of Boring** 9/28/84 **Project No.** 184312-0 **Sheet** 1 **of** 2

Surface Elevation = 752.5±

Sample No. and Method	Blows/Foot (N)	Unconfined Compressive Strength (TSF)	Penetrometer Value (TSF)	Moisture Content (%)	Dry Unit Weight (PCF)	Unified System Symbol	Depth (Feet)	Description	Water Table Observations
								Dark Gray Silty Clayey Sand Trace Gravel, Medium Dense	
1-SS	13	-	2.50	12.2	-	SW	5		
2-ST	-	0.87	1.00	30.6	91.0	CL		Gray Silty Clay, With Fine Sand Below 11.0, Firm to Stiff	0Hr
3-ST	-	1.48	0.50	30.3	95.1	CL	10		
4-SS	10	0.70	0.25	29.6	-	CL			WD
5-SS	14	-	-	15.2	-	CL	15		
6-SS	35	-	-	16.3	-	SP	20	Brown to Gray, Medium to Fine Sand With Silt, Medium Dense to Dense	
7-SS	21	-	-	20.3	-	SM	25		
8-SS	33	-	-	22.7	-	SP	30		
9-ST	-	-	-	-	-	-	35	Gray Sandy Silty Clay, Very Stiff	
10-ST	-	2.00	1.75	18.0	113.9	CL	40		

Continued on Sheet 2 of 2

Remarks:

Note: All Shelby tube (ST) unconfined compression tests performed in the laboratory.

SHIVE-HATTERY ENGINEERS
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DUANE ARNOLD ENERGY CENTER
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Log of Borings - Boring RW-4

Figure 2.5-105 (Sheet 1)

Project: Iowa Electric Light & Power/DAEC
Cedar Rapids, Iowa

Boring No. RW-4 **Date of Boring** 9/28/84 **Project No.** 134312-0 **Sheet** 2 **of** 2

Sample No. and Method	Blows/Foot (N)	Unconfined Compressive Strength (TSF)	Penetrometer Value (TSF)	Moisture Content (%)	Dry Unit Weight (PCF)	Unified System Symbol	Depth (Feet)	Description	Water Table Observations
							Surface		
								Continued From Sheet 1 of 2	
							40		
11-SS	26	2.70	4.00	20.0	-	CL	45		
							50	Moderately Weathered Limestone	
							55	Bottom of Boring @ 54.7' Boring Backfilled 9/28/84	
							60		
							65		
							70		
							75		

Remarks:

Note: All Shelby tube (ST) unconfined compression tests performed in the laboratory.

SHIVE-HATTERY ENGINEERS
Record of Subsurface Exploration

DUANE ARNOLD ENERGY CENTER
IOWA ELECTRIC LIGHT & POWER COMPANY
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Log of Borings - Boring RW-4

Figure 2.5-105 (Sheet 2)

BORING LOG

DUANE ARNOLD ADDITIONS	11186-001	1 of 1	B-1
West of Radwaste Building	20' West of radwaste; 5' South of fence	90°	
6/21/78	6/21/78	STS IOWA J. Buckhahn	CME 75
3"	50.5		50.5'
		13	
756.7	13.0'/743.7	50.5'/706.2	
140 lb/30 in.	None	J. B. Givens	

Sample type				Penetration blows			Elevation	Depth	Sample	Description and classification	Remarks
							756.7	0			
										0-2.5' Fine to coarse poorly graded Sand, brown, dense, trace silt, low moisture (SP)	Began drilling with 4" auger to 14.5'; set casing; continued drilling with 3" drag bit and bentonite water bit at 13.0 ft.
SS	18	12	41	14	20	21	754.2		1		
SS	18	12	82	21	34	48			2	2.5-21.0' Fine to coarse Sand, tan, dense to very dense, trace silt, trace fine gravel, low moisture (SP)	
SS	18	12	53	31	30	23			3		
SS	18	14	42	16	22	20		10	4		
SS	18	6	80	28	40	40			5	13.0-21.0' Wet	
SS	12	7	54/6	26	54				6		
							735.7	20 21		21.0-31.0' Sandy Clay, light brown, stiff, low to medium plasticity, moist (CL)	
SS	18	9	11	4	5	6			7		
SS	18	6	21	6	6	15			8		Qp #8 = 1.5 tsf
SS	18	11	23	6	10	13			9		Qp #9 = 2.25 tsf
							725.7	30 31		31.0-50.5' Silty to sandy Clay, dark grey, very stiff, medium plasticity, organic odor (CL)	
SS	18	14	26	7	11	15			10	36.0' Gravel layer	Qp #10 = 1.5 tsf
SS	18	16	27	5	12	15		40	11		Qp #11 = 2.0 tsf
SS	18	14	25	6	12	13			12		Qp #12 = 2.5 tsf
SS	18	16	32	8	13	19			13		Qp #13 = 2.75 tsf 40' artesian pressure noted by driller
							706.2	50.5		Bedrock at 50.5' Total Depth 50.5'	Hole grouted to top 6/21/78.

West of Radwaste Building | B-1

DUANE ARNOLD ENERGY CENTER
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Log of Borings - Boring B-1

Figure 2.5-106

BECHTEL

BORING LOG

DUANE ARNOLD ADDITIONS	11186-003	1 of 1	B-2	
West of Radwaste Building	50' West of radwaste; 5' South of fence	90°		
6/22/78	6/22/78	STS IOWA J. Buckhahn	CME 75	3"
51.5		51.5'		
12	756.2	13.5'/742.7	51.5'/704.7	
140 lb/30 in.	None	J. B. Givens		

				PENETRATION BLOWS		
SS	18	4	46	16	27	15
SS	18	8	60	19	28	32
SS	18	12	64	19	32	32
SS	18	10	25	10	12	13
SS	18	8	14	7	7	7
SS	18	9	45	20	20	25
SS	18	4	11	7	5	6
SS	18	3	27	6	12	15
SS	18	1	27	7	12	15
SS	18	14	30	12	16	14
SS	18	18	35	10	15	20
SS	18	4	38	16	18	20

Elevation	Depth	Samples	Description and Classification	Remarks
756.2	0	1, 2, 3, 4, 5, 6	2" gravel layer 0-24.0' Fine to coarse poorly graded Sand, brown, dense to very dense, trace gravel, trace silt, low moisture (SP) 3.5-13.5' Light brown 8.0' Very thin seam of gray silt 9.5' Medium dense 13.5-24.0' Light grayish brown, lenses of silty sand with trace clay, medium dense 19.5-24.0' Seams of gravel, dense	Auger with 4" auger to 14.5'; set casing; drill with 3" drag bit and bentonite; water hit at 13.5 ft.
732.2		7	24.0-27.0' Silty Clay, dark gray, stiff, medium plasticity, moist, trace organics and organic odor (CL)	
729.2	27	8, 9	27.0-37.0' Sandy Clay, light brown, very stiff, low to medium plasticity, moist (CL)	
719.2	37	10, 11, 12	37.0-51.5' Silty to sandy Clay, dark gray, very stiff, medium plasticity, moist, trace gravel, organic odor (CL)	Qp #10= 2.0 tsf Qp #11= 1.75 tsf
704.7	51.5		Bedrock at 51.5' Total Depth 51.5'	Hole grouted to top 6/22/78.

West of Radwaste Building | B-2

DUANE ARNOLD ENERGY CENTER
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Log of Borings - Boring B-2

Figure 2.5-107

BORING LOG

DUANE ARNOLD ADDITIONS	1186-001	1 of 1	B-3	
West of Off Gas Building	40' north of B-4 (B-4 is 125' West of SW corner off gas)	90°		
6/20/78	6/21/78	STS IOWA J. Burkhahn	CME 75	3"
51.5'		51.5'		
		12		751.7
8.5'/743.2	51.5'/700.2			
140 lb/30 in.	None	J. B. Givens		

				PENETRATION BLOWS		
SS	18	12	20	11	10	10
SS	18	1	17	8	9	8
SS	18	14	13	3	5	8
SS	18	16	8	3	3	5
SS	18	12	22	6	9	13
SS	18	9	39	15	20	19
SS	18	14	43	23	21	22
SS	18	15	87	23	42	45
SS	18	12	36	16	24	12
SS	18	14	24	5	9	15
SS	18	16	20	11	9	11
SS	18	18	21	5	9	12

Elevation	Depth
751.7	0
747.7	4
745.2	6.5
743.7	8
715.7	36
708.2	43.5
705.7	46
700.2	51.5

0-4.0' Fine to coarse Sand, brown, medium dense, trace to little silt, trace gravel to 1", moist (SP)

4.0-6.5' Sandy to silty Clay, brown, very stiff, low plasticity, gravel to 1-1/2", moist (CL)

6.5-8.0' Clayey Silt, gray, stiff, low plasticity, 1"-2" seams of brown silty sand throughout layer (ML)

8.0-36.0' Fine to medium Sand, grayish brown, loose to medium dense, trace coarse sand, moist (SP)
13.5-14.0' Rust color
14.0-17.0' Light brown, medium dense
17.0-21.0' Trace to little coarse sand and trace to little gravel, trace silt, dense
21.0-26.0' Trace coarse sand and no gravel, trace organics, silt seam at 24.3', trace silt
26.0-36.0' Occasional seams of darker sand containing trace organics, very dense, trace silt
31.0-36.0' Dense

36.0-43.5' Silty Sand, gray, medium dense, moist, trace clay, fine to medium grained with trace coarse (SM)

43.5-46.0' Sandy Clay, gray, very stiff, low to medium plasticity, moist (CL)

46.0-51.5' Clayey Silt to silty Clay, gray, stiff, low plasticity, moist (CL-ML)

Bedrock at 51.5'
Bottom of hole.

Auger (4") to 14.5'; set casing; drill with 3" drag bit and bentonite. Noted water at 8.5'; water at 7.5' before casing set.

Qp #12= 1.0 tsf

Pumped 100 gal. of grout into hole 6/20/78; let cave over night to 30.0' from ground surface; complete grouting 6/21/78 (total about 200 gal. both days)

West of Off Gas Building | B-3

DUANE ARNOLD ENERGY CENTER
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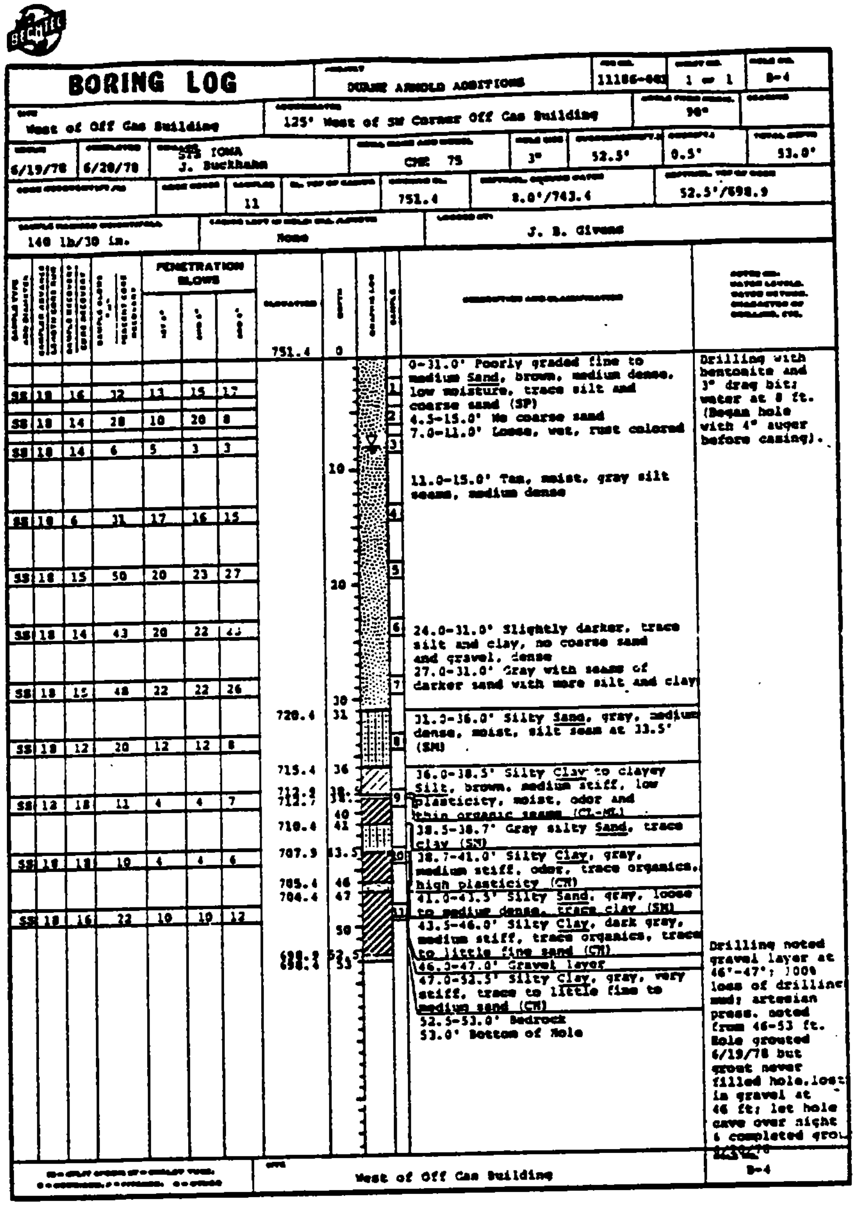
Log of Borings - Boring B-3

Figure 2.5-108



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Log of Borings - Boring B-4

Figure 2.5-109