2.5 GEOLOGY, SEISMOLOGY, AND GEOTECHNICAL ENGINEERING

This section provides information regarding the seismic and geologic characteristics of the site and region surrounding the site. Extensive geologic and related explorations were conducted at Unit 2, and can be categorized as: 1) general exploration; 2) bedrock investigation and mapping; 3) supplementary geologic exploration; and 4) rock mechanics investigation. Extensive geophysical surveys were performed, including seismic refraction profiling and downhole logging. The principal consultant for performing these studies was Dames & Moore, as identified in Section 1.4.5.

Unit 2 is located on the southeast shore of Lake Ontario near Scriba, within a portion of the Central Lowlands Physiographic Province and the northern part of the Appalachian Basin Geologic Province. Unit 2 is situated within the Eastern Stable Platform Tectonic Province.

The results of the investigations indicate that the site is relatively tectonically stable and is free of major active tectonic structures. There are no known capable faults within 8 km of the site, and there is no potential for surface faulting within the site area. The subsurface materials at the Unit 2 site are not adversely affected by collapse, subsidence, or uplift. The ground motion level for the SSE was calculated using conservative correlations sufficiently representative of seismic wave transmission characteristics of the site.

2.5.1 Basic Geologic and Seismic Information

Section 2.5.1 provides the basic geologic and seismic information required for evaluation of the content in other sections of Section 2.5. Section 2.5.1 is subdivided on the basis of information pertaining to the region (Section 2.5.1.1) and information pertaining specifically to the site (Section 2.5.1.2). The topics discussed for both the region and site are similar, consisting of physiography, stratigraphy, geologic structure, geologic history, regional geologic hazards, and site engineering geology.

2.5.1.1 Regional Geology

This section provides basic regional geologic information required for evaluation of the content in other sections of Section 2.5. Section 2.5.1.1 is subdivided on the basis of general topics pertaining to the regional geology. Regional physiography and geologic setting are described in Section 2.5.1.1.1. The stratigraphy and structural geology of the site region are discussed in Sections 2.5.1.1.2 and 2.5.1.1.3, respectively. A synopsis of the geologic history of the region is provided in Section 2.5.1.1.4. Natural geologic hazards and hazards that might be induced by man's activities are discussed in Section 2.5.1.1.5.

2.5.1.1.1 Physiography and Geologic Setting

Unit 2 is located on the southeast shore of Lake Ontario, in the town of Scriba, Oswego County, NY. It is situated about 10 km (6.2 mi) northeast of the city of Oswego and about 53 km (32.8 mi) northwest of Syracuse, NY (Figure 2.5-1).

Physiography

The Nine Mile Point site is situated within the Erie-Ontario Lowlands, a subdivision of the Central Lowlands Physiographic Province (Figure 2.5-2)[1]. The Erie-Ontario Lowlands subprovince is bounded on the south by the Appalachian Uplands Province, on the north by the Canadian Shield, and on the east by the Tug Hill Upland and the Adirondack Highlands. To the northeast, the Erie-Ontario Lowlands is separated from the St. Lawrence Lowlands by the Frontenac Axis.

The Erie-Ontario Lowlands extend southward from the site about 56 km (35 mi) to the Portage Escarpment which forms the boundary between the lowlands and the Appalachian Uplands Province, and westward into Canada near Niagara Falls. Differential erosion of southerly dipping, relatively more resistant carbonate rock of the Lockport and Onondaga Formations has resulted in east-west trending cuestas within the lowlands.

The generally flat to gently undulating topography of the Erie-Ontario Lowlands is superimposed upon an erosional bedrock surface of irregular, low relief. The land surface rises gradually to the south and southeast from an elevation of 75 m (246 ft) above msl at the southern shore of Lake Ontario to an elevation of approximately 152 m (500 ft) above msl at the Allegheny Escarpment. A veneer of glacial deposits such as tills, glaciofluvial sediment, and proglacial lake sediments covers most of the area. The deposits occur as drumlin fields and recessional moraines. Postglacial weathering and stream erosion have modified the terrain only slightly.

Geologic Setting

The region around the Unit 2 site has been subdivided by geologists into various geologic provinces, that is, large areas that are unified with respect to lithofacies development and gross structural style. The boundaries of geologic provinces typically (although not always) coincide with those of the physiographic provinces identified above. Thus, in the literature, the names are often used interchangeably in descriptions of geologic setting. A regional geologic map (Figure 2.5-3) identifies the geological provinces used in this subsection to characterize the geologic setting.

The Unit 2 site is situated within the northern part of the Appalachian Basin geologic province in New York State (Figure

2.5-3)[2,3]. The bedrock at the site consists of early Paleozoic marine sediment underlain by Precambrian crystalline rock at a depth of approximately 457 m (1,500 ft). The Appalachian Basin is elongate, trending northeast-southwest, and is bounded by the Precambrian rocks of the Canadian Shield to the north, and the Adirondack Massif to the northeast. The latter two are linked by the Frontenac Axis across the St. Lawrence River Valley. To the west, the Appalachian Basin is bounded by the Findlay Arch and its northward extension, the Algonquin Axis[2]. The Findlay Arch represents a structural rise in the crystalline basement rock, over which the Paleozoic sediment thins substantially. Westward, beyond the Findlay Arch, lies the Michigan Basin, another geologic province. The eastern boundary of the Appalachian Basin geologic province is termed the South Mountain Front[4] because rocks to the southeast of the front are metamorphosed, whereas those of the Appalachian Basin are not[5]. In Canada this boundary is approximately equivalent to Logan's Line (Figure 2.5-3). The Appalachian Basin extends as far south as Alabama (Figure 2.5-2).

The Unit 2 site is located in a portion of the Appalachian Basin geologic province that is characterized by few deformation features despite the long history of the site bedrock (Sections 2.5.1.1.4 and 2.5.1.2.6). The regional structural geology is presented in Section 2.5.1.1.3, and the site structural geology is discussed in Sections 2.5.1.2.3 through 2.5.1.2.5. Knowledge of the geologic setting is important to the discussion of regional tectonic provinces, which are presented and discussed in Sections 2.5.2.2 and 2.5.2.3 (Figure 2.5-4).

2.5.1.1.2 Stratigraphy

The regional stratigraphic succession in New York and adjacent Canada is illustrated by the map (Figure 2.5-3) and cross sections provided (Figures 2.5-5 and 2.5-6). Figure 2.5-7 further depicts the relationship between stratigraphic units investigated at the site and the regional stratigraphy.

<u>Precambrian Basement</u>

Crystalline basement rocks of Precambrian age are exposed north and northeast of the Unit 2 site in three areas:

1. The Adirondack Massif 80 km (50 mi) northeast.

2. The Frontenac Axis of the Thousand Islands region 121 km (75 mi) north.

3. The Canadian Shield of Ontario and Quebec Provinces, Canada 145 km (90 mi) north.

In general, the Precambrian basement rocks of these three areas are divided into two broad geologic provinces and several subprovinces and belts[6-8] which are illustrated on the regional

geologic map (Figure 2.5-3). In western Ontario, the Superior subprovince of the Canadian Shield is separated from the neighboring Grenville subprovince of central and eastern Ontario and Quebec by the Grenville Front (Figure 2.5-3). The crystalline rocks of the Superior subprovince (metamorphosed igneous and sedimentary rock) are of Archean age, that is, more than approximately 2.6 billion years old[9]. Southeast of the Grenville Front, the Grenville subprovince consists of largely high-grade metamorphic rocks, whose ages range from Archean in the northwest to Early and Medial Proterozoic (2.6 to 1.1 billion yr ago) near the St. Lawrence valley[7,9]. The Grenville subprovince consists of both orthogneiss and paragneiss surrounding a central belt of paragneiss. This belt (Figure 2.5-3) is termed the Central Metasedimentary Belt[6]. The terrain of mixed gneisses of Canada and the Adirondacks is understood to represent the pre-Grenville Precambrian "basement" shallow platform upon which the protolithic sediments and volcanics were deposited in Proterozoic time (1.1 to 1.4 billion yr ago)[9]. These particular paragneisses are termed the Grenville Series or the Grenville Supergroup[6,7]. The Late Precambrian sediments underwent progressive, high-grade metamorphism culminating approximately 1.1 billion yr ago, during an event known as the Grenville Orogeny. The sediments were altered to marble, quartzite, quartzofeldspathic gneiss, granulite, leucogranitic gneiss, and amphibolite. The gneisses bordering the Central Metasedimentary Belt in western Ontario consist largely of quartzofeldspathic paragneisses, whereas those in the Adirondack Massif consist of intermediate igneous rocks with granitic rocks and marbles[10]. Local igneous intrusives ranging from felsic to mafic occur throughout the Grenville subprovince.

The erosional surface developed on the Precambrian rocks of the Canadian Shield slopes gradually southward, beneath the Paleozoic cover rocks of the Appalachian Basin at a gradient of 8.5 to 9.5 m/km (45 to 50 ft/mi). This slope gradually steepens to 17 to 19 m/km (90 to 100 ft/mi) in south central New York[2,11] (Figures 2.5-5 and 2.5-6). As the erosional surface dips southward, the Paleozoic cover becomes thicker.

Direct knowledge of the composition of the Precambrian basement beneath the cover rocks is very limited, consisting of core samples from widely-spaced boreholes that penetrated the basement complex[11,12]. The long geological history of the rock, which includes intensive metamorphism, makes interpretation of the distribution and structure of the covered basement rocks very difficult. More information is gained from studies of regional Bouguer gravity and aeromagnetic anomalies (Figures 2.5-8 and 2.5-9).

Deep well data from central New York State[11,12] indicate that the basement rocks are typically of gneissic composition (biotite-quartz-feldspar gneiss). Marble, amphibolite, and quartz-feldspar granulite have also been reported. Granite was reported from a well near Oswego[12]. A geothermal test well

drilled in Spring 1982 penetrated the basement near Auburn, NY. Core retrieved from the Auburn Geothermal Test well at 1560-m depth consisted of a coarse-grained, light gray massively-bedded dolomite marble with occasional trace amounts of pyrite, chalcopyrite with bornite and chalcocyte. This basement rock type is consistent with rock types previously extracted from deep wells in southern Ontario and Central and Western New York State.

Deep well data from western New York State[11,12] indicate extensive areas of granite and marble. Locally, gabbroic intrusives surrounded by areas of metavolcanics are known[12].

The Central Metasedimentary Belt of the Canadian Shield in Ontario consists of rock types similar to those extracted from deep wells in central and western New York State. In the belt, local mafic intrusives and extensive felsic intrusive bodies are surrounded by highly-deformed gneisses, marbles, and schists[6]. This information suggests that prominent local density and magnetic contrasts may exist in the basement rocks. Moreover, the pattern of regional Bouguer gravity anomalies (Figure 2.5-8)[13,14] and regional aeromagnetic anomalies (Figure 2.5-9)[15] confirms such contrasts. From these observations, it appears reasonable to suggest that the Central Metasedimentary Belt extends southward from the Canadian Shield into central and western New York. It is uncertain, however, how far south this belt continues.

Paleozoic Sedimentary Rocks

The Paleozoic formations in northern central New York State form a wedge that thickens to the south, away from the Canadian Shield. The strata are relatively flat-lying but have been rotated slightly, exhibiting a gentle, regional gradient to the south (approximately 9.5 m/km, 50 ft/mi). In the vicinity of the Nine Mile Point site, Late Ordovician formations have been exposed by erosion that removed younger units (Figure 2.5-3). Farther south, younger Silurian, Devonian, and Carboniferous formations are still preserved in the central part of the Appalachian Basin.

The basal units of this sedimentary wedge consist of an Early Cambrian clastic sequence (the Potsdam Sandstone) and an Ordovician carbonate sequence (the Beekmantown, Black River, and Trenton Groups), both of which were deposited in a relatively stable shelf environment (Figures 2.5-7 and 2.5-10). These strata are overlain by a Late Ordovician-Early Silurian clastic sequence, which constitutes the Utica Shale and the Lorraine Group (Whetstone Gulf and Pulaski Formations), the Oswego Sandstone, the Queenston Formation, and the Grimsby Formation[16]. This sedimentary sequence represents a transition from a shelf to a terrestrial environment of deposition, reflecting a westward regression of the Ordovician sea, which was synchronous with development of the Taconic highlands from the time of development

of the Middle Ordovician unconformity (on the top of the Beekmantown Group, Figure 2.5-7).

Following the deposition of the Late Ordovician-Early Silurian clastic sequence, a marine transgression occurred and produced a second carbonate sequence. This Silurian-Devonian sequence includes the Clinton, Lockport, Salina, and Helderberg Groups, which are presently exposed in central New York (Figures 2.5-3, 2.5-7, and 2.5-10). Above the Helderberg Group, a sequence of clastic deposits was deposited unconformably above the Onondaga Group (carbonates), namely the Hamilton through Conewango sequence (Figure 2.5-7). These are syn-orogenic deposits related to the Acadian Orogeny and formation of the Catskill delta.

Farther south a clastic sequence of younger Paleozoic formations (Mississippian-Pennsylvanian) is exposed (Figures 2.5-3, 2.5-7, and 2.5-10). These formations are also syn-orogenic sedimentary rocks, deposited synchronously with the Alleghanian deformation.

The youngest rocks of the Appalachian Basin are of Early Permian age and are exposed in southwestern Pennsylvania and West Virginia[2].

Mesozoic Sedimentary Rocks and Intrusive Igneous Rocks

A regional, erosional unconformity occurs in New York State that was developed on the Paleozoic sedimentary strata (Figure 2.5-7). There is no record of Mesozoic sedimentation preserved within 320 km (200 mi) of Nine Mile Point. Mesozoic (Triassic and Jurassic) sediment and basic igneous intrusive and extrusive rocks are known to occur, however, in narrow, extensive fault-bounded basins in southeastern Pennsylvania, New Jersey, New York, and the Connecticut River Valley of New England (Figure 2.5-3). Moreover, Cretaceous marine strata are exposed in the Atlantic Coastal Plain of New Jersey.

As shown on Figure 2.5-11, igneous rocks are rare in the Western Appalachian Mountains. In New York State, small kimberlite-alnoite dikes occur near Ithaca striking roughly north-south. These dikes yielded radiometric ages of 136 to 145 million yr[17]. Lamprophyre dikes are reported along the borders of the Adirondack Massif that are generally oriented east-west[18] and have yielded isotopic ages of 118 to 150 million yr[17,19]. Larger alkaline intrusions of the Monteregian Hills of Canada occur in an east-west trending belt east of Montreal. These intrusions yielded isotopic ages of 90 to 150 million yr[20].

Alkalic intrusive bodies have been classically interpreted as characteristic of the rift tectonic environment[21]. Their parent magmas are believed to have been generated from very deep levels in the crust or from within the upper mantle.

Cenozoic Sedimentary Rocks

Tertiary Deposits There are no Tertiary strata known within 320 km (200 mi) of the Nine Mile site because the region was uplifted and undergoing widespread erosion during that time. However, Tertiary strata do comprise the major portion of the Atlantic Coastal Plain formations exposed in New Jersey and Delaware (Figures 2.5-2 and 2.5-3). Early Tertiary igneous activity occurred in Virginia and West Virginia[22,23]. This is the youngest igneous event in the eastern United States.

Quaternary Deposits It is generally believed that there were several advances of continental ice sheets in New York State during the Pleistocene Epoch. The ice advances were separated by warm interglacial stages[24].

Glacial scour and periglacial slope processes were such that virtually all New York State unlithified sediments are of late Wisconsinan (the latest glacial advance) or more recent derivation. Only in narrow transverse valleys cut deeply enough to escape intense glacial scour might preglacial paleosol be preserved[25]. In southwestern New York State a reentrant in the Wisconsinan terminal moraine near Salamanca may contain materials related to pre-Wisconsinan glaciation, perhaps Illinoian[25]. These materials include glacial drift, possible end moraine deposits, plus outwash, kame terrace, and deltaic deposits. However, these deposits may also be earliest Wisconsinan[25].

During the advance and retreat of the Wisconsinan ice sheet, the bedrock surface was scoured and till was deposited unconformably on the older Paleozoic strata of New York State. Minor readvances resulted in additional till being laid down, in some areas, covering glaciofluvial or glaciolacustrine deposits above the lowermost till. During stages of retreat of the Wisconsinan ice sheet, glaciolacustrine sediment from proglacial lakes such as Lake Iroquois, together with glaciofluvial sediment, were also deposited[26].

2.5.1.1.3 Structural Geology of the North-Central Appalachian Basin

To facilitate understanding the structural evolution of the north-central Appalachian Orogen, geologists have separated the deformed rocks from those virtually undeformed, and have subdivided the deformed areas. One principal type of subdivision is based on a structural front[5,27] which delineates sharp changes in surface geology in belts of deformed rocks in the Appalachian region. Three structural fronts are mentioned below (Figure 2.5-4). They coincide with the boundaries of the tectonic provinces discussed in Section 2.5.2 (Figures 2.5-4 and 2.5-12) in most instances, but not all of them.

The intensity of deformation of sedimentary strata of the Appalachian Basin progressively increases toward the east and southeast. As mentioned previously, the rocks underlying the Nine Mile Point site area are situated within the northwestern

region away from the major effects of lateral crustal shortening associated with orogenesis. The line that delineates the change from nonfolded to folded sedimentary strata marks the boundary between two tectonic provinces defined by Hadley and Devine[28], namely the Central Stable Region and the Appalachian Foldbelt (Figure 2.5-4). The former is divided into two subprovinces in this report (Section 2.5.2.2) termed the Eastern Stable Platform and the Ottawa Basin. The Appalachian Foldbelt tectonic province is divided into two tectonic subprovinces[28] and also has several structural partitions as discussed below. The Northern Appalachian Foldbelt subprovince is separated from the Faulted Foldbelt tectonic subprovince by approximate coincidence with the Blue Mountain structural front (Figure 2.5-4), where a transition occurs from broad, concentric folds in Silurian to Carboniferous rocks to short, tight, similar and isoclinal folds in Cambro-Ordovician rocks, that are commonly bounded by thrust faults. The eastern border of the Faulted Foldbelt approximately coincides with the South Mountain Front, the Taconic Front, and Logan's Line (Figure 2.5-4). These fronts largely separate the metamorphosed terrain to the southeast from the nonaltered basin rocks on the northwest. The tectonic province east of this line is called herein the Appalachian Mobile Belt (Section 2.5.2.2).

Since Late Precambrian time, the earth's crust in the region surrounding the Unit 2 site has been deformed both on large and small scales. The large-scale deformation has occurred in two lithic domains of importance to the site, namely the Precambrian basement gneisses and the Paleozoic sedimentary strata of the north-central Appalachian Basin covering much of the basement. The mode of large-scale deformation reflects both major crustal shortening and differential vertical displacement, and is represented by major faults and folds. The small-scale deformation consists of regional fractures (joints), faults with lengths less than 8 to 16 km (5 to 10 mi), and minor folds. Mineralization of both major and minor structures occurred. Moreover, localized igneous intrusions are known. The principal structural features pertinent to this report are depicted on Figure 2.5-4.

This section evaluates the tectonic framework of the north-central Appalachian Basin region which is necessary 1) to provide a basis for understanding the evolution of the geologic conditions at the site, and 2) to provide baseline information for evaluation of the potential geologic and seismic hazards that might adversely affect the site (Section 2.5.1.1.5).

<u>Late Precambrian Faults</u> In Section 2.5.1.1.2, the composition and distribution of the Precambrian basement are described (Figure 2.5-3). The Precambrian rocks are metamorphosed and have undergone extensive ductile deformation. The first major episode of brittle deformation that followed the last ductile deformation in the site region was associated with the development of extensive northeast-striking transcurrent faults during the formation of the Proto-Atlantic Ocean in Late Precambrian time[29].

Faults of this origin represent fundamental features in the evolution of the basement crust and these features are characteristic of the development of later brittle deformation. Two major zones of Late Precambrian faulting in the region are known: the Ramapo fault system in southeastern New York and New Jersey and the Carthage-Colton Mylonite Zone in the St. Lawrence lowlands of New York (Figure 2.5-4).

Ramapo Fault System The Ramapo fault system (Figure 2.5-4) is a deeply rooted, northeast striking zone of shearing that dates back to Late Precambrian time as evidenced by the association of basic intrusives of this age within the fault zone[30]. This feature was also active during Paleozoic and Mesozoic time. Minor contemporary seismicity occurs along the trend of the zone[31,32] but this feature is more than 320 km (200 mi) from the Unit 2 site and is therefore not of concern regarding the seismic design of the site (Section 2.5.2).

Carthage-Colton Mylonite Zone This structural feature, approximately 97 km (60 mi) northeast of Nine Mile Point, forms a 113-km (70-mi) long, northeast-trending border between the Central Adirondack highlands (igneous and metaigneous rocks) from the northwest lowlands (metasedimentary and volcanic rocks) of the Precambrian basement (Figures 2.5-4 and 2.5-13). It varies in thickness, and at its southern exposed extremity, it is folded over the Lowville dome[33]. This feature is interpreted to be a fold-thrust nappe. Subsequent brittle deformation along this structure might also have occurred. A very steep gravity gradient parallels the mylonite zone (Figure 2.5-8) with a prominent gravity high that corresponds[34] with the northwest lowland terrain. Simmons[35] attributed this anomaly to density contrasts between the Grenville metasediments and the central Adirondack Massif, and suggested it is rooted at a depth of 5 km (3 mi). Bouguer residual (100 km, 62 mi) gravity maps indicate that the probable source of this anomaly is less than 33 km (21 mi) deep[36]. From this information, the origin of the mylonite zone must be considered complex. The mylonite was developed during the Grenville Orogeny (1,100 million yr ago). Moreover, the occurrence of a swarm of Hadrynian basic dikes in Precambrian rocks (Figure 2.5-4) along the St. Lawrence River indicates a probable Late Precambrian reactivation[37].

Paleozoic Orogenic Folds and Decollements In the Central Appalachians, the folds and thrusts in the belts of deformed strata can be attributed to orogenic pulses that began in the Medial Ordovician Period and climaxed with the major deformation in Late Carboniferous to early Permian time[38]. This span of time encompasses the three phases of Appalachian orogenesis known as the Taconic, Acadian, and Alleghanian Orogenies. Only a portion of the Appalachian Basin in New York, Pennsylvania, and Canada was deformed by major lateral shortening associated with the foregoing orogenies. The deformed portion corresponds to the Northern Appalachian Foldbelt tectonic province (Figure 2.5-4).

The extent of major Paleozoic shortening is to some degree evident from regional gravity studies. Figure 2.5-8 shows the configuration of Bouguer gravity anomalies of New York State and adjacent Canada[34,39]. At the east edge of this map, a pronounced and steep gravity gradient separates the broad area of negative gravity anomalies to the west from higher magnitude anomalies to the east. This gradient represents a fundamental change from the sedimentary rock of the basin to the highly-metamorphosed, plutonized, and faulted rocks of the core of the crystalline Appalachians. Because of the thrusting of large rock masses to the west of this gradient during orogenesis, the resultant increased loading depressed the light crust into the asthenosphere, perhaps accounting for the negative gravity anomalies in the Appalachian Basin region[40].

<u>Northern Appalachian Foldbelt</u> Fold deformation within this tectonic province is characterized by changes in geometry and finite strain from northwest to southeast. These changes occur in three belts separated by structural fronts. The first belt occurs between the northern and western boundary of this tectonic province and the Allegheny Front (corresponding to a portion of the Appalachian Plateau physiographic province).

In this belt, flexures are so broad and gentle that they are best analyzed on structural contour maps. These maps clearly indicate a geometry that is part of the fold system east of the Allegheny Front (corresponding to the Valley and Ridge physiographic province)[41]. Broad folds west of the Allegheny Front are shown on the regional tectonic map (Figure 2.5-4), where it is seen that the folding appears to diminish progressively to the north and northwest. In New York State, indications of permanent shortening strain along the tectonic transport direction have been detected as deformed fossils in Upper Devonian strata 145 km (90 mi) northwest of the last major fold south of the Appalachian Structural Front[42]. Moreover, finite strain of calcite and the occurrence of solution cleavage[43] indicate that minor penetrative strain dies out progressively northward from the location where the outermost deformed fossil has been recognized in the plateau rocks of New York. The strata of this belt are of Carboniferous age.

Rodgers[27] interpreted that folds of the Appalachian Plateau (in Pennsylvania) actually resulted from sliding at depth within the basin strata along layer-parallel detachment structures (decollements) which locally climbed upward through the sedimentary section as overthrusts. This has been extended to the Appalachian Plateau of New York by Prucha[44] in the Finger Lakes district, where folding was found to be disharmonic, affecting only the strata above the Salina Group salt beds. Prucha postulated that a major decollement occurs along the base of the salt, and passes southward into a thrust fault of Alleghanian age. Geologic studies of the Salina basin[45] have suggested the presence of two right-lateral strike slip faults

and a normal fault near Keuka Lake (Figure 2.5-4) in central New York. These are interpreted as tear faults above slip-planes in the salt during Alleghanian deformation. Numerous other smaller-scale thrusts have been found in surface studies in the Northern Appalachian Foldbelt tectonic province in Pennsylvania[46-48]. Engelder and Geiser[49] postulate that the transported rocks above the Salina Group behaved as a lithotectonic "ductile unit" (which includes the Onondaga Limestone, and Oriskany Sandstone of the Helderberg Group) overlying a "brittle unit" (which includes all of the Late Devonian Group of western New York) that drapes over the thrust faulted rocks at depth. The brittle unit developed tear faults that did not propagate upward through the ductile unit. Differential displacement on the tear faults produced an arcuate shape in the overlying ductile unit by solution along arcuate surfaces.

The second belt in the Foldbelt tectonic province is bounded on the northwest by the Allegheny Front and on the southeast by the Blue Mountain Front (which also corresponds to the western tectonic subprovince boundary of the faulted part of the Appalachian Foldbelt of Hadley and Devine[28]. The prominent folds of this belt (corresponding to the west half of the Valley and Ridge physiographic province) have fairly uniform wavelengths of 11 to 18 km (7 to 11 mi). This spacing is also present in the Appalachian plateau west of the Allegheny Front[41]. The fold lengths range from generally 55 km (35 mi) to as much as 320 km (200 mi). Fold geometries are basically of two types: concentric with faults at their cores[50], and large kink-style folds[51]. The rocks of this belt are of Silurian-Devonian age, in contrast to those in the first belt.

The faults of this belt that are expressed at the surface are typically moderate to steep thrusts dipping southeastward. They range from a length of 11 km (7 mi) and stratigraphic displacement of a few hundred feet to a length of 320 km (200 mi) and a displacement of 8 km (5 mi)[41]. They are intimately related to the development of anticlines as splays off of major decollements. Thus, folding and faulting likely proceeded synchronously as major horizontal shortening occurred[38,41,50].

A third, narrow belt extending northeastward into New York is characterized by thrust faults. This belt occurs between the Blue Mountain Front on the northwest and the South Mountain Front on the southeast, which becomes (approximately) the Taconic Front in eastern New York State. This structural front is the leading edge of a major detachment feature. The episode of orogenesis during which this feature evolved, namely the Taconic Orogeny, takes its name from this feature.

The configuration and role of the basement during the formation of the structures in all three belts of the Foldbelt tectonic province has long been considered problematical[5,27,38]. Basement involvement in the deformation is possible near the South

Mountain anticlinorium[5] and near the Neelytown Anticline and Marlboro Syncline (Figure 2.5-13)[52]. However, the majority of evidence[5,50,53] suggests major detachments at depth. The basement surface now dips to the southeast[2] and appears to have dipped that way during Paleozoic deformation. Some, for example, Gwinn[53], have suggested that these features have been caused by gravitational sliding; whereas others (for example, Root[41]) have proposed that the basement slope opposed the tectonic transport of the strata leading, in part, to the type of deformation observed.

Appalachian Mobile Belt Southeast of the South Mountain Front, where metamorphism of the basin rocks begins, major nappes reflect the large amount of lateral shortening (folding and thrusting) that occurred in response to intense deformation in the collision zones between continents during the Paleozoic orogenies. The majority of structures in this tectonic province lie beyond 320 km (200 mi) from the Unit 2 site and, hence, detailed discussion is not warranted. However, recent reflection surveys from northeastern New York and central Vermont indicate that the Precambrian basement of western Vermont is probably detached[54]. Similar findings in the Southern Appalachians have been reported[55,56]. The detachment in Vermont may extend beneath the eastern Adirondack Massif[54]. This major decollement is a remnant of the Paleozoic continental convergence, and the collision zone occurred within the Appalachian Mobile Belt Tectonic Province (Section 2.5.2.2).

Paleozoic Epeirogenic Folds and Faults A number of deformation structures occurring in the Central Stable Region and Appalachian Foldbelt tectonic provinces were not directly induced by major horizontal shortening due to orogeny, but instead by mainly vertical tectonic movements over a very large region (epeirogeny). This process has been fundamental to the evolution of the northern and western portion of the Appalachian Basin in which the Nine Mile Point site is located.

Folds The Findlay Arch and its extension, the Algonquin Axis (Figure 2.5-4), is a structural rise that separates the Michigan Basin to the northwest from the Appalachian Basin to the southeast. The Precambrian basement surface dips symmetrically from the Arch into each basin, except in the region of the Chatham Sag, where the dip is steeper toward the Michigan Basin than the Appalachian. However, the dips away from the Chatham Sag are gentler than those away from the arches[57]. There are no faults known to occur parallel to the arches.

The Adirondack Massif is a dome (Figure 2.5-4) that formed sometime later than Upper Devonian time[58,59]. The core of the dome consists of Precambrian rocks (Section 2.5.1.1.2), and the edges are mantled by eroded Early Paleozoic strata of the Appalachian Basin. The Frontenac Axis is a narrow structural rise, consisting of Precambrian basement rocks, that connects the west side of the Adirondack Massif with the southeastern part of

the Canadian Shield, also composed of Precambrian rocks. The St. Lawrence Valley transects the Frontenac Axis. Within the basin strata adjoining the southern boundary of the Frontenac Axis in New York State, many northeast-trending gentle, upright to steeply-inclined folds and domes occur (Figure 2.5-4)[60,61]. These features have been attributed to tectonic deformation during Paleozoic time, probably related to basement faulting such as the Indian River faults[60] (Figure 2.5-13).

<u>Faults</u> The Eastern Stable Platform and the Ottawa Basin subprovinces of the Central Stable Region, and the Appalachian Plateau portion of the Foldbelt tectonic province, are characterized by persistent small-scale structural features and limited areas where larger structures occur. The majority of these features originated in Paleozoic time. Reactivation of faults also occurred in Mesozoic time, as discussed below.

<u>Northeast-Striking Faults</u> The Ramapo fault system was reactivated during the Paleozoic Era as a major tectonic element during the Taconic and Acadian Orogenies[30]. Major strike-slip displacements occurred, and deep-seated Paleozoic plutons used the system as pathways for crustal intrusion.

In Pennsylvania, extending from the extreme southwest corner to south-central New York State (Figure 2.5-4), is a broad zone of anomalously distributed Paleozoic sedimentary rock units. These distribution patterns are attributed to apparent down-to-the-basin normal faults within this zone, which functioned as growth faults during the evolution of the north-central Appalachian Basin[62-65]. This structural zone in the Precambrian basement is an extension of the intracratonic graben known as the Rome Trough, which stretches from eastern Kentucky to northern Pennsylvania[63]. Root[62] termed the Pennsylvania portion the Greene-Potter fault zone, which he suggests is "possibly the most apparent part of an extensive fault system rather than a discrete zone" caused by crustal extension that penetrates the continental plate. This major crustal feature evidently may extend toward the Adirondack Massif in New York, based upon both geological and geophysical data.

In Pennsylvania, the southeast boundary of the Greene-Potter zone nearly coincides with the major aeromagnetic lineament of King and Zietz[66], the New York-Alabama lineament, which seems to indicate a profound crustal break. Farther north in New York State, the lineament (Figure 2.5-9) diverges from the Greene-Potter zone to the east toward the Champlain Valley. Each feature is parallel to the grain of younger Appalachian structures, a fact that Root[62] and King and Zietz[66] believe implies a causal relationship.

The Clarendon-Linden fault[67-69] is about 145 km (90 mi) west of Nine Mile Point and is approximately 161 km (100 mi) long (Figure 2.5-4). Hutchinson, et al[70], interpret that this feature extends northward across Lake Ontario to the Bancroft area of southern

Ontario based upon geological and geophysical data. They also interpret that the extension of the Clarendon-Linden fault does not connect with the Picton fault zone of Liberty[71] that occurs several miles farther east (Figure 2.5-13). The Picton zone has not been demonstrated to extend southward from Lake Ontario into New York, although Rickard[52] interpreted minor faults along strike of the zone southeast of Rochester, NY (Ontario County faults on Figure 2.5-13). The southern extremity of the Clarendon-Linden fault seemingly terminates in New York State[69].

Gravity and magnetic investigations of this fault[72] suggest that some control of the Paleozoic structure by the Precambrian basement is evidenced by the location of the structure on the western flank of a series of gravity and magnetic highs (Figures 2.5-8 and 2.5-9)[73-75] that are probably caused by mafic intrusives in the basement. The distribution of these geophysical anomalies suggests that the fault may indeed extend to Pennsylvania. This possibility is further strengthened by some Devonian depositional patterns in Pennsylvania (Figure 2.5-10)[62].

The total relative vertical displacement across the fault zone is in excess of 91 m (300 ft)[76]. Van Tyne[69] interpreted this fault as originating in Cambro-Ordovician time as a growth fault with down-to-the-east displacement and with at least 30 m (100 ft) of additional strata on the down-faulted side. Growth fault displacements perhaps continued into Devonian time[62]. During the Late Paleozoic (Carboniferous-Permian) Alleghanian deformation, the Clarendon-Linden was reactivated and became a high-angle reverse fault with the east-side upthrown relative to the west side[69]. Displacements of this nature diminish upwards and the structure passes into a complex drape-fold. This structure is important because historic and contemporary seismicity are associated with the feature, but only along a limited segment in the Attica-Dale region of New York. Section 2.5.2.3 reviews the seismological correlation of earthquakes with the fault zone.

Another northeast-striking structure, the Demster-Mexico Structural Zone, occurs about 8 km (5 mi) east of Nine Mile Point near New Haven and Mexico, NY (Figures 2.5-4 and 2.5-13). It might represent a discontinuity at depth with drape-folding of the cover strata. This feature differs from the Clarendon-Linden significantly in that there are no recognized historic or contemporary concentrations of seismic activity along the feature. Deep-seated Kimberlite dikes have locally intruded the cover strata in proximity to the Demster Zone (Figures 2.5-4 and 2.5-13).

The geometry and extent of this structure have been extensively investigated and are discussed in Section 2.5.1.2.4 under Site Geology. The seismologic considerations pertinent to this feature are discussed in Section 2.5.2.3.

Northeast faults with major vertical displacements occur between Utica and Saratoga Springs, NY, affecting both the Precambrian

basement and the on-lapping Early to Medial Paleozoic strata. Locally, kimberlithic and lamprophyric dikes have intruded in proximity to the zones of major displacement (Figures 2.5-4 and 2.5-13). Fisher[77] describes these structural features of the Mohawk and Champlain Valleys, which apparently die out upsection in Silurian strata and appear to extend southward beneath the Devonian strata[78], as exhibiting down-on-the-east displacements of nearly 610 m (2,000 ft). Root[62] has suggested that a blocky isopach configuration of Devonian shale units along strike to the southwest of the Mohawk Valley faults, together with localized thinning of the shale, were caused by reactivation of the structures at that time.

East-West-Striking and West-Northwest-Striking Faults The only east-west major faults of the region that were active in Paleozoic time occur more than 322 km (200 mi) from the Unit 2 site, and thus will be mentioned only briefly. The Electric fault[57] occurs along the Chatham Sag transecting the Findlay Arch, and extends from the Precambrian basement to Upper Silurian and possibly Lower Devonian strata. It does not appear to extend into New York (Figure 2.5-4).

In southern Pennsylvania, the Transylvania fault zone, which is more than 15 km (9 mi) wide, occurs east of the Appalachian Structural Front, and is believed to be a fundamental fracture in the continental plate[79] (Figure 2.5-4).

The Ottawa-Bonnechere graben (Figure 2.5-4) is largely confined to the Ontario and Quebec Provinces of Canada, and consists of major west-northwest and east-northeast-striking faults, with as much as 457 m (1,500 ft) of vertical displacement[80]. This fault system is aligned with a series of Mesozoic, alkalic, intrusive rocks which compose the Monteregian Hills. The isotopic ages yielded by these rocks range between 90 and 150 million yr before present (B.P.)[20]. Two faults of this system, the Gloucester and the Winchester Springs faults, have been extended into northern New York on the basis of geophysical investigations[81]. The Gloucester fault (Figure 2.5-4) is a member of the Ottawa-Bonnechere fault system[82]. It trends west-northwest[81]. The northeast side is downthrown about 549 m (1,800 ft). In Canada, the fault cuts through early Paleozoic sedimentary rocks and is believed to be a member of a zone of extension that had been active repeatedly long ago in the geological past. Radiometric dates of igneous rocks occurring within the Ottawa-Bonnechere (see Igneous Activity and Mineralization) indicate activity from Late Precambrian to Middle Ordovician and, again, in Mesozoic time[8,80].

Minor Structural Features Aside from the larger structures described above, the rocks of the Eastern Stable Platform and the Ottawa Basin subprovinces of the Central Stable Region, and the Appalachian Plateau portion of the Foldbelt tectonic province in New York State, exhibit persistent small-scale, structural

features that appear to be strongly characteristic of the northeastern Appalachian Basin (Figure 2.5-4).

In the Appalachian Plateau of New York State, joints occur in the sedimentary rocks from Cambrian to Devonian age[49,83] (Figure 2.5-14). Joints have also been reported in Mississippian, Pennsylvanian, and Permian age rocks in the Appalachian Plateau of western Pennsylvania and eastern Ohio[84]. Engelder and Geiser[49] compare the results of their studies of regional jointing with the results of Parker[83,85] and Nickelsen and Hough[84]. Three principal joint sets are recognized, namely Sets I, II, and III[83]. Sets I and II represent cross-strike and longitudinal joints, respectively, whose strikes shift about 30 deg in concert with the swing of fold axis trends in the Appalachian Plateau[86]. Set III is joint set whose attitude is constantly about N70E. Parker[83,85] recognized Set I as a coeval double set, but Engelder and Geiser[49] consider them as two sets (Ia and Ib) having formed at different times and within a different stress field. Engelder and Engelder[42], Engelder[43], and Engelder and Geiser[87] have related the formation of Set Ia to the formation of solution and pencil cleavage and fossil distortion, and thus the folds in the Appalachian Plateau. They note that these joints are sometimes calcite filled, a phenomenon also recognized by Nickelsen and Hough[84].

Some investigators have suggested that some of the regional joints formed as shear fractures, largely on the basis of geometry and information from small faults in the region. DeGroff[88] documented horizontal slickensides indicating strike-slip shear on east-northeast trending joints near Syracuse, NY. Dames & Moore concluded, on the basis of their measurements plus data from Sutton[68] in western New York, that the N75E and N45W joints formed as conjugate shears[89]. Nickelsen and Hough[84] also observed strike-slip on systematic Set I joints in Pennsylvania. Engelder[90] discounts this interpretation as a localized phenomenon.

An alternative explanation is that the joints may have formed in response to the vertical movements of the Appalachian Basin. Price[91,92] suggested that such a mechanism is plausible for the formation of a conjugate fracture set whose acute bisectrix is perpendicular to the long axis of a sedimentary basin that was subjected to nonparallel (asymmetric) uplift. However, many joints in central New York do not exhibit a shear fracture geometry. It is likely that these others formed as hydraulic tensile fractures[92].

West-northwest trending normal faults occur locally in the region and have been studied by Dames & Moore east of Rochester[89] and also at Nine Mile Point[93]. The faults at each locality dip both north and south and are mineralized by calcite, sulfides, and sulfates. The structural pattern is illustrated on a stereogram on Figure 2.5-15, which is a lower hemisphere contour diagram of poles to 42 small faults in the Somerset-Oak Orchard area of

western New York. The maxima in this diagram indicate three fault sets representing N75E-striking faults, dipping 60°S and 70°N; N50W-striking subvertical faults; and N77W-striking subvertical faults. These faults have similar orientations to the fracture sets recorded throughout New York State. The movement senses on each of these sets of faults are both strike-slip and dip-slip. In addition to poles to the high-angle faults, the stereogram contains a cluster of poles to low-angle discontinuities with a maximum near the center of the plot. These structures generally strike N15W and dip 15-20°W. Reverse displacements were exhibited by some of these structures. Nickelsen and Hough[84] also reported small, high-angle strike-slip faults with northwest and west-northwest strikes in the Appalachian Plateau of Pennsylvania.

The regional joints are generally believed to be post-Pennsylvanian and pre-Alleghanian[83-85], as evidenced by their relationship to folds and kimberlite dikes in the Eastern Stable Platform subprovince of the Central Stable Region, and the Appalachian Plateau portion of the Foldbelt tectonic province. Studies of mineralization in the Nine Mile Point vicinity tend to corroborate this interpretation (Section 2.5.1.2). However, Engelder and Geiser[49] interpret Set Ib to be related to stress changes attendant upon uplift and erosion. Engelder[90] further postulates that Set III formed in response to the contemporary stress field and is, therefore, the youngest. He suggests that evidence for this is present throughout the Central Stable Region. The west-northwest to east-northeast high-angle faults are believed to have formed in Paleozoic time with possible reactivation in the Mesozoic time (Section 2.5.1.2).

<u>Mesozoic Structures</u>

<u>Lake Ontario Homocline</u> The sedimentary strata of the Nine Mile Point region exhibit a gentle, southward gradient of 15 to 30 m (50 to 100 ft)/mi. Kay[80] attributed this feature to gradual differential subsidence of the basin concomitant with accumulation of the sediments. Studies of mineralization (Sections 2.5.1.2.3 and 2.5.1.2.4)[94] together with stratigraphic information (Section 2.5.1.1.2) have suggested that asymmetric uplift and erosion of the northern end of the basin occurred in Mesozoic time and is continuing at present (Section 2.5.1.1.4). This uplift resulted in gradual tilting of the strata southward. Hence, the Lake Ontario homocline probably began to form in Late Paleozoic to Mesozoic time. Because uplift has continued throughout the Cenozoic, it is likely that the homocline is continuing to develop, at least in the Lake Ontario region.

<u>Faults</u> The Ramapo fault system forms the western boundary of the Triassic-Jurassic Newark Basin in southeastern New York and northern New Jersey. The basin extends southeastward into Pennsylvania as the Gettysburg Basin, which is also bounded by a fault on the northwest side (Figure 2.5-4).

The Ottawa-Bonnechere graben in Canada (Figure 2.5-4) was active in Mesozoic time[80] when the majority of displacement on the principal faults of the system occurred. This deformation is attributed to major rift tectonic deformation associated with the formation of the Atlantic Ocean[95].

Kumarapeli and Saull[37] proposed that the tectonic episode that formed the Ottawa-Bonnechere graben also produced rift-related faults in the St. Lawrence Valley. They surmised that the Clarendon-Linden fault was also active at this time.

Mesozoic activity along the Demster-Mexico, Greene-Potter, Mohawk and Champlain Valley systems is suggested because:

1. The regional geometric configuration of these features suggests that rifting involved all of these features[29,62].

2. The spatial distribution of Mesozoic kimberlitic rocks along the zone is believed to be characteristic of rift tectonics.

3. The distribution of heat flow anomalies in the northeastern United States and Canada[96] correlates well with these structures. The rates of heat flow are such that the pattern of these anomalies suggests that these features were sources of high heat flow in Mesozoic time.

The absence of Mesozoic strata to record deformation of that age in the site region, however, makes it necessary to infer that deformation features in the plateau were active in the Mesozoic. This interpretation is strengthened by the occurrence of Mesozoic ultrabasic dikes along the preexisting fault zones[62].

<u>Cenozoic Structures</u>

Small-scale, low-angle thrusts in the Eastern Stable Platform subprovince of the Central Stable Region, and the Appalachian Plateau portion of the Foldbelt tectonic provinces and equivalent basin sediments in Ohio, have been reported with displacements ranging from a few feet to tens of feet[97,98]. These features have been called valley bottom faults, deemed to be postglacial, localized stress-relief phenomena. Andrews[99] reported a thrust fault near Rochester, NY, (Figure 2.5-4) with tens of centimeters of westward transport. Glacial till appears to be deformed. Andrews attributed this feature to the action of advancing glacial ice. Thrust faults of similar dimensions are represented by the maximum near the center of the stereogram of Figure 2.5-15. These features, although of uncertain age, are probably Cenozoic features because: 1) they have deformation aspects suggestive of formation relatively close to the earth's surface, and 2) they resemble thrust faults at the site (Section 2.5.1.2.3) which are known to be of Quaternary age.

Throughout the northern part of the Appalachian basin two other types of compressional, deformational features of postglacial age are commonly encountered (Figure 2.5-16). All known postglacial features in western New York are buckles, referred to in the literature as popups and/or broken anticlines. Buckles that were caused by man's activities are not included in this discussion (Section 2.5.1.1.5).

Two types of buckles have been described in New York State. The most commonly encountered type is a chevron-style anticline fractured along its crest. It typically coincides with a preexisting fracture or fault, which is situated along the axial plane[68]. The second type of buckle occurs as a broad, gentle sinusoidal fold with no apparent disruption at its crest[100]. The buckles are commonly asymmetric.

A survey of buckles in the Nine Mile region[94] revealed that their widths range from 1.5 to 12 m (5 to 40 ft) and their lengths are typically on the order of 30 m (100 ft), but in a few cases exceed 1.6 km (1 mi). Their amplitudes range from a few inches to 3.7 m (12 ft). The depths to which these buckles developed are not normally known, but are typically inferred to be from 3 to 9 m (10 to 30 ft).

Very small postglacial displacements along high-angle fractures have been reported in the Champlain and Hudson River valleys[59,101]. These features are shown on Figure 2.5-16, and may either be of tectonic origin or caused by ice wedging along hidden sheet joints at depth. However, shallow seismic reflection profiling of the sediments at the bottom of Lake George have revealed apparent faults with displacements up to 24 m (80 ft)[59].

Frequency diagrams show strikes of the axial planes of buckles and of postglacial faults (A and B, Figure 2.5-16). Most buckles strike northwest, roughly parallel to the interpreted axis of postglacial upwarp near Lake Ontario[102] (Section 2.5.1.1.4). This coincidence implies a possible causative relationship of glaciostatic rebound and postglacial buckling.

A similar association farther east can be seen regarding postglacial faults in the Champlain Valley region (Diagram B, Figure 2.5-16). Their strikes parallel the interpreted axis of postglacial warping there[103]. Although the northeast-trending structural grain in New England may have an effect on the orientation of the postglacial faults, the correlation of the strikes of these faults to the axis of tilting is noteworthy.

None of the site-observed structures show field evidence of having been generated by the instantaneous release of stored strain energy. This observation is supported by the virtual aseismic nature of the region surrounding the site over the period of the historical record (Section 2.5.2.3.2), where the

largest earthquakes experienced have been limited to MMI IV or magnitude 3.0 (duration) from instrumentally-recorded events.

Although there have been recorded instances of earthquakes related to crustal unloading in southeastern New York[288], there have been no instances in the site region in which popups have caused vibratory ground motion. Even if the assumption were made that all of the seismic events within a 40-km (25-mi) radius of the site were attributable to popups, the ground motion produced would be no greater than that caused by a MMI IV or magnitude (Mc) 3.0 earthquake.

Moreover, the structures observed onsite (i.e., the Cooling Tower fault and the drainage ditch structure), as described in Section 2.5.1.2, no longer have the capability to accumulate or store sufficient strain energy to generate an earthquake through instantaneous strain relief. Additionally, it has been demonstrated that in-situ stresses at the site are not favorably oriented, with respect to the Cooling Tower fault or drainage ditch structure, to cause renewed movement.

Igneous Activity and Mineralization

The oldest post-Grenville igneous activity in the region is represented by a swarm of Late Precambrian to Medial Ordovician diabase and andesite dikes in the regions of the Frontenac Axis and the Taconic Structural Front (Figure 2.5-4)[8, 104-106]. This was likely associated with Late Precambrian rifting in the St. Lawrence and Champlain-Hudson regions[37].

Kimberlite, peridotite, and lamprophyre dikes and rocks have also intruded the Appalachian foreland from southern Pennsylvania to the Adirondack region (Figures 2.5-4 and 2.5-11). Available radiometric dates (Section 2.5.1.1.2) indicate a Triassic to Jurassic time of emplacement for the dikes. Igneous rocks of this affinity are known to be unique for characterizing extensional tectonic environments associated with major horsts and grabens[21,107]. The spatial distribution of these rocks corresponds closely with the location of apparent basement fault zones. The Greene-Potter zone has these dikes associated with it, and it appears that others are associated with the Demster-Mexico zone, the Indian River zone, and the Mohawk and Champlain Valley zones (Figures 2.5-4 and 2.5-13 and Section 2.5.1.2.4). Hence, it is likely that portions of these zones were active in Mesozoic time, an era when crustal extension associated with the divergence of continents to form the present Atlantic Ocean characterized the tectonic environment.

Alkalic plutonic bodies intruded along the eastward extension of the Ottawa-Bonnechere graben in Cretaceous time (Section 2.5.1.1.2). This igneous episode appears to have been the last in the Nine Mile Point region.

Epigenetic minerals occur along fractures and faults throughout the northeast end of the Appalachian Basin. These deposits are characterized by the predominance of sulfides, sulfates, and associated calcite gangue. The assemblage commonly includes sphalerite, galena, pyrite, barite, celestite, gypsum, chalcopyrite, fluorite, pyrrhotite, and marcasite. These minerals are not abundant at a single location in the Paleozoic rocks and, therefore, presently are not of economic significance. However, they have been mined extensively where they occur in the Precambrian basement of the Canadian Shield and Adirondack Massif.

The distribution of known occurrences of these minerals is shown on Figure 2.5-17. From the data, there appears to be a tendency for the mineralized fractures and faults to be oriented west-northwest. The occurrence of the minerals does not appear to be related either to the known centers of igneous activity or to the local hydrothermal effects of intrusions. They were more likely deposited from principally connate, epigenetic basin brines(108), with one exception mentioned below.

Several studies of fluid inclusion homogenization temperatures of these minerals have been reported. The homogenization temperature approximates the formation temperature of the mineral. The temperature data have been summarized in Table 2.5-1, and their locations are shown on Figure 2.5-17.

The data indicate that the mineralization is associated with temperatures of 122° to 142°C (252°-288°F)(109,110); however, data from Ontario, NY, give homogenization temperatures averaging 250°C (482°F) with one high temperature of 300°C (572°F)(89). Secondary fluid inclusions are also recorded ranging from 75° to 125°C (167° to 257°F)(89,110). The minerals that yield temperatures above 200°C (392°F) (Ontario) occur on a small west-northwest-striking normal fault, whereas the other minerals occur as fissure veins (Madoc) or along horizons affected by solutioning (Penfield). From previous studies, the mineralization at Ontario, NY, is interpreted to be of hydrothermal origin(89). Examination of the aeromagnetic map of that area (Figure 2.5-9) reveals a very striking cluster of positive anomalies (approximately 1,000 gammas) at this locality. From this information, one can infer that one or more magnetic bodies occur in the basement rocks, suggesting the presence of an ancient igneous intrusion. Hot fluids emanating from this body may have permeated the Paleozoic rocks along the small faults near Ontario, NY. It is noteworthy that this seems to be the only reported occurrence of quartz mineralization in the site region. However, one may conclude that, throughout the region, the large area affected by elevated temperatures suggests a regional paragenesis for the mineralization such as the heating of the basin brines caused by the regional geothermal gradient.

An absolute age of the regional geothermal mineralization has not been determined because of the absence of minerals datable by

radiometric methods. The minerals are not clearly associated with known igneous intrusions, and thus cannot be dated using the ages of these intrusives. However, estimates of the geothermal gradient and the depth of burial at the time of mineralization can be made from paragenetic studies. This was done at the Nine Mile Point and New Haven sites (Sections 2.5.1.1.3, 2.5.1.1.4, and 2.5.1.1.6). From this information, together with knowledge of the regional geologic history, one may infer that the epigenetic mineralization ranges in age from Middle Paleozoic to Mesozoic in the Nine Mile Point area.

Modern thermal springs occur near Albany, NY (Figure 2.5-18). Lebanon Spring, for instance, has a maximum surface water temperature of 22°C (72°F) and Sand Spring, in the northwest corner of Massachusetts, a temperature of 24°C (75°F)[111]. Carbonated mineral springs of Saratoga Springs, NY, and vicinity occur 16 km (10 mi) west of the Taconic Structural Front near Albany.

The paucity of thermal springs in the Nine Mile Point region indicates that there are minor variations in the heat flow caused by convection and conduction. Heat flow data of New York and Pennsylvania[96] (Figure 2.5-18) confirm this. Low heat flow gradients are common in the region of central and western New York State. The cause of this phenomenon is not known, but two models have been conceived to explain it[96]. One explanation is that the Precambrian basement rocks consist of radioactively-depleted igneous rocks similar to those of the Adirondack Massif, thus explaining the low heat flow values. The other possibility is that erosion of the basement rock and deposition of Appalachian Basin sediments of low radioactivity at least partly account for the relatively low heat flows of the region. Within this region, however, there is a weak northeast-trending anomaly of relatively higher heat flow that coincides with the Greene-Potter and Demster-Mexico structural zones. It seems that heat is escaping relatively more rapidly along the basement fracture zones. A similar possibility exists for the Champlain Valley rift zone. The results of chemical analyses of the Saratoga waters[112] which occur within the rift zone suggest localized heat sources. Dissolved carbon dioxide in these waters derives from connate water mixed with meteoric water. Isotope data suggest that the connate CO_2 is thermally generated. However, the magnitude of remnant heat from a cooling Mesozoic pluton would be insufficient to explain the phenomenon. Young and Putman[112] instead suggest that embryonic igneous activity of Miocene to Quaternary age, whose effects are not strong enough to be detected as thermal surface waters (48° to 56°F) at Saratoga, is responsible for the carbonation. Therefore, it seems that the highest heat flow rates are associated both with radiogenic heat production from cooling plutons, such as in New England[96], and from anomalous thermal conduction associated with possible deep-seated northeast basement fracture zones[112].

Cross-Strike Lineaments

Odom and Hatcher[113] classify structural lineaments (Group 11) as either basement controlled or supra-crustal. There are two types of basement-controlled structural lineaments: active (reflecting motion of the basement), and passive (reflecting basement surface irregularities). Supra-crustal cross-strike structural discontinuities are broad, diffuse, transverse zones of structural disruption in the overthrust belts such as those characteristic of the Appalachian Basin[114]. Typically, folds of various scales, longitudinal faults, and simple Bouguer gravity anomalies and geomorphologic features either terminate or change style across or between these features. Where structural lineaments have been studied, they have been found to have very long geologic histories extending back to Late Precambrian-Cambrian time[113]. Some contain regions of contemporary seismicity and in general warrant careful consideration with respect to nuclear plant seismic design.

Four prominent cross-strike features are postulated in Pennsylvania, New York, and southern Ontario[40] based principally on the regional configuration of simple Bouguer gravity anomalies and the regional aeromagnetic signature (Figure 2.5-19). There is little compelling geologic evidence to support the speculation that these lineaments are direct manifestations of fundamental discontinuities emanating from the Precambrian basement. However, what evidence there is may be significant. The possible relationship of the New York-Pennsylvania lineaments to historic seismicity is speculative. However, Diment, et al[40], suggest that regions of intense seismic activity may coincide with one or more of these lineaments, implying that the crust is segmented by both northwest and northeast discontinuities, and that the interior portions of these blocks are relatively aseismic. This viewpoint is supported by Fakundiny's review of available geologic and geophysical data[115]. These blocks are approximately 145 km (90 mi) square. Moreover, Fakundiny recognizes possibly less distinct east-west discontinuities extending across central New York through the Finger Lakes. It is noteworthy that the Unit 2 site is situated within the interior of one of these crustal blocks. The geologic evidence of the nature of the New York-Pennsylvania lineaments is briefly reviewed here.

Lineament G (Figure 2.5-19) is one of several lineaments in this area of Pennsylvania. Mapping along the lineament has revealed:

1. Localized structural disturbance in the form of complex thrust and tear fault combinations[116].

2. A down-faulted structural block in the Paleozoic strata, the Tipton Zone[117], bounded by northwest strike-slip faults (Figure 2.5-4).

3. Localized zones of high fracture density[116].

4. Water and wind gaps in topographic ridges[118].

5. Pb-Zn and Cu mineralization of fractures and faults[118].

6. Localized, abrupt termination of small-scale folds and faults[118].

Studies in the vicinity of Lineament X (Figure 2.5-19)[45,48] have detected possible north-northwest-trending horsts and grabens, generally below the Salina salt, but also above it. High magnetic contrasts across the edge of a block bounded by the lineament are also present. In addition, a small fault along the west edge of Lake Cayuga, NY, appears to exhibit a change in movement sense from pre- to post-Ordovician time. These three lines of evidence suggest basement structural control along the lineament. Podwysocki and others report evidence for a different, namely thrust-faulting, origin of this block[48]. This consists of: 1) seismic well log and mine data that show thrust faults originating in the salt, 2) deformed fossils along the block's margins, and 3) change of fold axis attitude near the margins of the block.

There is little corroborative evidence of northwest structure along Lineament F (Figure 2.5-19). A system of Hadrynian diabase dikes[105] near the Frontenac Axis (Figure 2.5-4) outlines a transverse fracture pattern with deep crustal connections extant in early Paleozoic time. However, no major northwest discontinuities are known in the area along Lineament F. Recent focal mechanism solutions from the southwestern Adirondacks[119] indicate northwest-striking nodal planes (Section 2.5.2).

Lineament E (Figure 2.5-19) coincides approximately with the region of the Ottawa River Valley wherein the Ottawa-Bonnechere System occurs. Major displacements, emanating from the basement, and deep-seated intrusives of Cretaceous age along strike of the fault zone, all suggest that this lineament is a fundamental structural discontinuity in Canada. Evidence is less compelling in New York State, except for the Gloucester and Winchester Springs faults (mentioned above), which appear to terminate in the central highlands of the Adirondack Massif.

Contemporary State of Stress in the Region

A review of reports of contemporary stress determinations in the Lake Ontario region was performed:

1. To evaluate the stress conditions in the region of the Unit 2 site with regard to the relative stability of the crust, and the degree of stability of certain geologic structures.

2. To interpret the meaning of the stress conditions encountered at the Unit 2 site.

The results of this review are presented in Section 2.5.4.1.4, and the specific details are contained in a previous report to the NRC[94]. The first reason listed above, however, is of relevance to considerations of regional geology.

From reports containing available contemporary stress data, it is possible to obtain only a general understanding of the stress field in the region. Consequently, it is very difficult to assess the degree of stability of geological structures such as those described in this section (Figure 2.5-4). The difficulty is attributed to the complex nature of the strain energy distribution in a fractured, anisotropic rock mass.

The stress field determined from overcoring measurements in a block of limited volume at the site (Section 2.5.4.1.4) clearly demonstrates this complexity. Nevertheless, some useful general characteristics of the regional stress field are noted.

The entire region surrounding Lake Ontario is characterized by a stress field that is considerably different than that to be expected solely due to present gravitational loading. The state of stress is manifested by the development of postglacial deformational structures (popup structures), and by the occurrence of shallow seismic events of low magnitude (Section 2.5.2). The presence of high horizontal stress is confirmed by the in situ measurements made at many locations[120,121].

The regional stress field appears to be spatially continuous and homogeneous in its character from one locality to the other. The most consistent observation regarding this field is that the greatest and intermediate principal stresses are generally horizontal or subhorizontal. The least principal stress tends to be nearly vertical. The trend of the greatest principal stress, as indicated by focal mechanism solutions, hydrofracture testing, and the average orientation of postglacial folds, is consistently northeast to east-northeast. However, for many possible reasons, overcore measurements made at shallow depths reveal a widespread variation of the orientation of the maximum horizontal stress, making it difficult to identify a representative trend (Section 2.5.4.1.4). This characterization is in general accordance with the definition of the Central Lowlands/Canadian Shield stress province[121].

Focal mechanism solutions of earthquakes that occurred in the region indicate that the greatest principal stress may be inclined at a shallow angle with respect to the horizontal plane.

A southward plunge of this stress is suggested for the area north of the Nine Mile Point site (Figure 2.5-20). The slip planes associated with shallow seismic events generally strike northwest or north-northeast, and dip steeply in most instances. Slip has been either reverse, strike-slip, or a combination thereof. Two focal mechanism solutions with oblique-normal slip have been

recorded in central New York (Section 2.5.2). Section 2.5.2 presents a discussion of the distribution of regional seismicity and the correlations of certain seismic events with known geological structures.

The magnitude of the maximum horizontal normal stress in the region is high in the depth interval from the surface to about 518 m (1,700 ft) (Section 2.5.4.1.4). Magnitudes range from several hundred to several thousand psi. Averaging the data gives a magnitude of this stress in excess of 98 kg/sq cm (1,400 psi), which is a fair indication of the order of stress to be expected in the Lake Ontario region. The average stress difference in the horizontal plane is also high, and is approximately equal to 70 kg/sq cm (1,000 psi).

It is not possible to assess the relative stability of geological structures on the basis of this limited knowledge of the regional stress field. One would not expect reverse-slip on high-angle faults (such as the Clarendon-Linden fault) with the maximum principal stress being horizontal to subhorizontal. However, the recent focal solutions by Hermann[122] for two Attica events indicate that the intermediate principal stress is sharply inclined with respect to the earth's surface. From this analysis, one must infer that a component of strain in a vertical plane is related to this shear displacement on the fault. For this reason, it is important to consider the distribution of contemporary vertical crustal movements in the region of the Unit 2 site, and the possible influence of these movements on the tectonic features in the region.

<u>Contemporary Vertical Crustal Movements</u>

Contemporary vertical crustal movement has been evaluated in the Lake Ontario region to identify the general trends indicated by available precise leveling data so that correlations with regional stress analyses can be made.

This investigation of recent vertical crustal movements is based on: 1) releveling data in Canada as interpreted by Vanicek[123], 2) tide gauge data for Lake Ontario[124], and 3) releveling data in New York State made available by the National Geodetic Survey (formerly U.S. Coast and Geodetic Survey). All such data were derived from established survey lines, which have permanent bench marks, whose elevations were determined relative to an initial bench mark. Resurveying the lines after a period of years redetermines the elevations of the bench marks relative to the initial bench mark. The difference in elevation of a bench mark between two surveys denotes relative movement of the mark with respect to the starting point. To determine relative motion of the crust on a regional scale, the starting points of all such lines must be subsequently related to a common base such as msl. The exact procedures employed in such surveys have been discussed by Rappleye[125].

Various orders of survey lines have been established by the National Geodetic Survey. The increasing order of the lines indicates an increased allowable error. For example, a second order survey has approximately twice the allowable error of a first order survey. Thus, to ensure that the most accurate data base was analyzed, only first order survey lines were used in this study.

For southern Ontario, Vanicek[123] reduced and processed all usable releveling data compiled by the Geodetic Survey of Canada. He used digital surface-fitting techniques to contour the rates of movement relative to a common point just north of Lake Huron, and his results have been applied to this study without further qualification. For Lake Ontario itself, Kite[124] interpreted tide gauge measurements to infer relative vertical movements of the lake basin. In the immediate area south of Lake Ontario, in New York State, no regional interpretations have been performed. Therefore, analysis of the basic data was performed as part of the study.

A detailed presentation of the manner in which the data from the three sources mentioned above was analyzed is contained in a previous report to the NRC[94].

The concept of absolute crustal movement (with regard to the "center" of the earth) is not considered in this analysis. Primarily because of cyclical (both long and short term) variations in sea level, there is no agreement between investigators as to what constitutes a reliable frame of reference for the resolution of "absolute" crustal movement. Geoid changes, as described by Morner[126], further complicate the concept. In the site area, the temporal motion of the crust relative to Cape Vincent contributes useful data leading to the interpretation on Figure 2.5-21. Moreover, the expected standard deviations calculated for the rates of movement reported for the Canadian data[123] and for the eastern New York State data[58] are on the order of about 1 mm/yr (0.039 in/yr). This suggests that the overall pattern of the contours of Figure 2.5-21, whose range in relative velocity over the study area amounts to between 5 and 6 mm/yr (0.20 and 0.24 in/yr), would be, for all practical purposes, unaffected by maximum expected deviations.

The results of this study (Figure 2.5-21) are considered to be more qualitative than quantitative because of inherent limitations in the resolution of vertical crustal movements[58,126,127].

The general trends of crustal movement in the Lake Ontario region (Figure 2.5-21) are identified with respect to Cape Vincent, and suggest a pattern of relative crustal velocity in the region of interest which conforms to the structural framework and postglacial crustal movements.

Historically, general uplift in the region is centered to the north and northeast, and is attributed to the pattern of crustal tilting as a result of glacial unloading (Section 2.5.1.1.4). Based on geomorphological evidence, the zero isobases established for the ancestral lakes Algonquin and Nippissing[124] have been projected southeastward to the general region of the site. These lines of equal movement parallel the overall trend of the velocity contours of Figure 2.5-21, and lend support to the interpretation. From isobases on the glacial Lake Iroquois shoreline[94], a relative velocity differential between Niagara Falls, NY, and Cape Vincent can be calculated. This shows that, over the past 12,000 yr, Cape Vincent has risen, relative to the Niagara region, at a rate of 7 mm/yr (0.28 in/yr). Figure 2.5-21 shows, from modern leveling data, a velocity differential between these two points of 2 mm/yr (0.08 in/yr). Considering the limitations of both types of data, it is concluded that this differential only qualitatively suggests that the rate of relative vertical crustal movement in the Lake Ontario region may be decreasing.

The contours of Figure 2.5-21 indicate the general regional trends discussed above, but display a local perturbation of this trend apparently caused by the relatively rapid uplift of the Adirondack Massif[58,128] superimposed on the regional trend. Also, the contours reflect the presence of the Frontenac Axis. Results of the contemporary vertical movement measurements considered corroborate the crustal tilting based on geomorphological studies presented in a previous report[94] (Sections 2.5.1.1.2 and 2.5.1.1.4). The local perturbations in the regional trend (Adirondack Massif, Frontenac Axis) are explained in terms of the present-day tectonic regime. Structure contours on the Precambrian basement in the site area (Figure 2.5-13) virtually parallel the velocity contours of Figure 2.5-21, suggesting large-scale crustal control rather than localized, surficial mechanisms.

In summary, the consistency exhibited by the velocity contours of Figure 2.5-21, the ancient lake isobases, and known geologic structure indicate that real motion of the crust is being observed, and that measurement errors and bench mark instability probably do not dominate. It follows that if the velocity contours represent actual vertical crustal movement, the general pattern and distribution of these contours may be related to the orientation of predominant regional stress axes because the contours reflect nonhomogeneous strain of the crust being superimposed on remnant gravitational and tectonic strain energy stored in the crust from the geologic past. If one accepts that this relationship does exist, then it is possible to make certain inferences regarding the stability of major structures in the region today.

First, it appears that the Lake Ontario homocline[80] is continuing to develop in modern times. The orientation of the principal stresses[120] as well as the isobases (Figure 2.5-21) are

consistent with the orientation of the homoclinal axis (Figure 2.5-13). Second, the traces of the only major faults of the region that exhibit contemporary seismic activity (the Clarendon-Linden fault and faults of the Ottawa-Bonnechere system) are either parallel or near parallel to the trends of the isobases of uplift or subsidence. By contrast, the trace of the Demster-Mexico feature which geologically resembles the Clarendon-Linden is oriented almost perpendicular to the isobase trends (Figures 2.5-13 and 2.5-21). Moreover, no contemporary seismic activity has been recorded along the Demster. If the isobases are locally parallel to the orientation of the intermediate principal stress, then the maximum-minimum principal stress plane is vertical and locally perpendicular to the trend of the isobase. Anderson's[129] theory of faulting assumes that when fault-slip occurs, giving rise to release of seismic energy, the line of the intermediate principal stress is within the plane of slip. Thus, where the isobase lines are parallel or near parallel to the trace of a steeply dipping fault, there is a greater tendency for it to be unstable, compared to the case where the isobases are normal to the trace. Although fault stability is dependent on many factors, it appears that the relationship just described is one that must be satisfied before fault-slip can occur through natural causes.

2.5.1.1.4 Geologic History

The geologic history of the Eastern Stable Platform tectonic province (Section 2.5.2) in the Unit 2 site region is characterized by cyclic vertical crustal movements. From late Precambrian time until Mesozoic time, the region's tectonic history has been predominated by epeirogenic movements attendant upon orogenesis and rifting occurring further to the east. Crustal movements of this type have continued to the present time, long after the cessation of Paleozoic orogenesis. The relatively underformed nature of the rocks in the site region reflect this nonorogenic history. The sequence of sedimentation and erosion within the northern Appalachian Basin provides a record of the vertical movements of the site region.

<u>Precambrian Time</u>

The Precambrian basement rocks exposed to the north and east of the site (Figure 2.5-3) reflect a complex history. The rocks of the Central Metasedimentary Belt were metamorphosed during the Grenville orogeny, approximately 1.1 billion yr ago. This event was followed by an extended period of crustal uplift and erosion accompanying late Precambrian rifting, which was related to the initial formation of an ocean basin to the east[29].

<u>Paleozoic Era</u>

The Paleozoic Era in the northern Appalachian Basin was characterized by relatively continuous crustal subsidence. Sediments within the basin were derived from the active orogenic

region to the east. The stratigraphic sequence in the site region (Figure 2.5-7) reflects sedimentation that occurred during three separate tectonic cycles in Paleozoic time.

The lithologies underlying the site resulted from the first cycle. These rocks form a wedge that thickens to the south and are derived from the Precambrian terrain to the northeast and east (Figure 2.5-6). The basal units of this sedimentary wedge consist of an Early Cambrian clastic sequence (the PotsdamSandstone) and an Ordovician carbonate sequence (the Beekmantown, Black River, and Trenton Groups), both of which were deposited in a relatively stable shelf environment. These rocks are disconformably overlain by a Late Ordovician-Early Silurian clastic sequence which constitutes the Utica shale and the Lorraine Group (Whetstone Gulf and Pulaski Formations, the Oswego Sandstone, the Queenston Formation, and the Grimsby Formation[16]. This sedimentary sequence represents a transition from a shelf to a terrestrial environment of deposition, reflecting a westward regression of the Ordovician epicontinental sea.

The Queenston and Grimsby Formations are terrestrial in origin derived from highlands to the east, and reflect the culmination of the Taconic Orogeny 435 million yr ago. The second tectonic cycle of continuing subsidence also began with a marine transgression producing a second carbonate sequence. This Siluro-Devonian sequence includes the Clinton, Lockport, Salina, and Helderberg Groups presently exposed in Central New York. These stable shelf deposits are overlain by a Middle to Late Devonian sequence of clastic sediments. The uppermost Devonian strata are equivalent to the sediments of the Catskill Delta derived from highlands to the southeast, and as such reflect the peak of the Acadian Orogeny 345 million yr ago.

A third Paleozoic tectonic cycle related to the Alleghanian Orogeny (250 million yr ago) is only reflected by the presence of Carboniferous and Permian clastic lithologies exposed in the extreme southern site region (Figure 2.5-3). These formations were derived from the southeast, east, and north. It is interpreted that uplift of the site region began in Late Paleozoic time. Hence, it is probable that major sedimentation in the basin related to the Alleghanian Orogeny did not occur in much of the site region.

<u>Mesozoic Era</u>

The site region during the Mesozoic Era underwent differential crustal uplift, extension, and attendant erosion of the Paleozoic strata as evidenced by the absence of Late Paleozoic to Pleistocene sedimentation in upstate New York. It is likely that sediments of the Triassic-Jurassic basins and the Atlantic Coastal Plain were partially derived from this area[130]. The differential uplift of the region ultimately caused the gentle southward regional gradient of the strata of approximately 9.5 m/km (50 ft/mi), which is known as the Lake Ontario Homocline[80].

It is not known exactly how much uplift has occurred in the region since the end of Paleozoic time. However, a thick sequence of sediments must have overlain the presently exposed Paleozoic rocks judging from their highly lithified nature. Gilluly's[130] estimation that 3,650 to 4,889 m (12,000 to 16,000 ft) of sediments have been removed from this region since the end of the Paleozoic era, based upon his estimates of the volume of younger sediments along the present Atlantic Continental margin, appears to confirm this idea. Chamberlin[131] has estimated that about 4,570 m (15,000 ft) of stratigraphic section have been removed by erosion since the Alleghanian Orogeny, based upon the projected crests of eroded folds. Gilluly[130] further indicates that this amount of denudation in the region, during the past 225 million yr, compares favorably with his own estimate of present-day erosion rates.

Gold and Marchand[20] and Philpotts[132], in their investigations of the Oka Intrusive Complex, which is part of the Monteregian Hills, reported the presence of Paleozoic rocks as young as Devonian included as xenoliths in diatreme breccia pipes. Since the Oka Complex is surrounded by Precambrian rocks at the present erosional level, this would imply that the removal of a considerable sequence of early Paleozoic sediments has occurred. Gold and Marchand[20] interpreted a depth of emplacement for the Oka Complex of at least 2,440 m (8,000 ft) based upon the projection of a nearby stratigraphic section into this area. Isotopic age determinations on biotite from the Oka Complex yielded ages of 95 $\pm$ 5 million yr (K/Ar) and 114 $\pm$ 7 million yr (Rb/Sr)[133], placing the age of crystallization in Cretaceous time, and indicating that 2,440 m (8,000 ft) or more of uplift has occurred since that time.

Cenozoic Era

The Cenozoic tectonic environment of the region has also been dominated by continuous uplift and erosion. However, during the last 1.8 million yr in the Pleistocene Epoch, the glacio-isostatic effects of several continental ice sheets have been superimposed on this overall uplift trend. Each of the ice advances was separated by warm interglacial stages, with each advance having its own effect upon the downwarping of the earth's crust. During the interglacial stages the crust underwent isostatic rebound.

The most recent glaciation (Wisconsinan) reached its maximum extent approximately 18,000 yr ago and covered the entire site region. The maximum extent of the Late Wisconsinan ice across the site region is marked by the Valley Head Moraine system south of Tully, NY (Figure 2.5-22). Although there is no absolute date on the oldest Valley Heads Moraine segments, a Carbon-14 date of 13,320 $\pm$ 200 yr B.P. on a mastodon pelvic bone found in valley train sediment gives a minimum age for some of the moraine segments[134]. The minimum age of all of the Valley Heads Moraine

system is marked by the oldest absolute date of Lake Iroquois sediment, 12,600 $\pm$ 400 radiocarbon years B.P.[135]. Although most of the moraine segments of the Valley Heads system are pre-Port Huron in age (Figure 2.5-23), some of the younger sediments may be early Port Huron[136].

During late Port Huron time the ice readvanced to the southwest, as evidenced by the relative ages and positions of Stanwix and Camden Moraine segments (Figure 2.5-22). Various moraine segments have been mapped to the north of the Stanwix-Camden ice-margin. The youngest moraine segment, the Oswego Moraine, is located just to the south of the Nine Mile Point site[137]. Fairchild correlates this moraine segment with the Rochester Moraine to the west. Recent work suggests that the Oswego Moraine correlates with the Carlton Moraine, a small moraine located just north of the Rochester Moraine[138,139]. A terminal position of a glacial advance during the Valders Phase (11,300 to 10,800 yr B.P.) has not been identified south of the Ontario Basin.

With the disappearance of the Stanwix ice, about 12,600 yr B.P., Lake Hyper-Iroquois of Fairchild[137] was established with an outlet from Rome, NY. Lake Hyper-Iroquois was ponded between the retreating ice and the drift barrier at Rome. A separate lake was contained in the western part of the Ontario Basin by the ice that persisted in the central part of the basin at this time[140]. This lake drained into Lake Hyper-Iroquois by way of the Fairport-Palmyra Channel (Figure 2.5-22). As the ice receded to the north, Lake Hyper-Iroquois, drainage continued to the southwest through the Rome outlet. This represents the highest stand of Lake Iroquois and is marked by a well-defined beach that is easily traced, except on the northeastern boundary (Figure 2.5-22) where ice was still present[141].

Continued recession of the ice resulted in the opening of new outlets and the subsequent lowering of lake levels. Coleman[141] proposed a low water stage in the Ontario Basin which he referred to as the Admiralty Beach, to account for entrenchment of the Trent Valley beyond the present river mouth. When ice receded from the St. Lawrence Valley area, the St. Lawrence River cut across the St. Lawrence sill and the Ontario Basin was lowered to the Admiralty level. Prest[142] defined the Admiralty level as 7.6 m (25 ft) above the level of the Champlain Sea or about 60 m (200 ft) below present sea level. The absence of marine fossils on the Ontario Basin led Coleman[141] to also conclude that marine waters from the Champlain Sea did not invade the area.

Sutton, Lewis, and Woodrow[143] identified four lake stages below the Lake Iroquois beach along the eastern shore of Lake Ontario: 1) Sandy Creek stage at approximate el 91 m (300 ft) above msl, 2) Skinner Creek stage at approximate el 78 m (256 ft) above msl, 3) dune stage about 9 m (30 ft) below present lake level (dated at 4,810 $\pm$ 180 radiocarbon yr B.P.), and 4) North Pond stage, a minor transgression about 0.6 m (2 ft) above present lake level.

The Sandy Creek and Skinner Creek stillstands probably occurred while Lake Iroquois was being drained to the low stage (Admiralty), and the Dune and North Pond stages occurred during the filling of present Lake Ontario.

Additional lake level stages that were lower than at present have been identified in the Ontario Basin by Lewis and Sly[144]. They mapped several notches in the Toronto shelf and concluded that there is a persistent notch approximately 14 m (45 ft) below present lake level in the Ontario Basin. This theory is supported by Woodrow and others in their observation of a boulder concentration between 8 and 15 m (26 and 49 ft) below lake level in Braddock Bay near Rochester, NY[145].

Modern Lake Ontario was initiated after the low water Admiralty stage in the Ontario Basin. The water level has risen continually from the Admiralty stage to its present elevation (Figure 2.5-24) because the St. Lawrence River outlet was elevated above the stage level due to glacial rebound.

2.5.1.1.5 Geologic Hazards

Both natural and induced geologic hazards must be considered regarding the safety of Unit 2. Types of natural geologic hazards include seismicity and fault movement, natural gas pressures, shoreline erosion, landslides, subterranean collapse and attendant ground subsidence, and volcanic activity. Possible induced hazards include groundwater fluctuations and bedrock excavation and extraction of geologic materials. Only some of these hazards are relevant to the Unit 2 site, and these are discussed below.

<u>Natural Hazards</u>

<u>Seismicity and Fault Movement</u> In consideration of possible tectonic hazards to nuclear plants, Odom and Hatcher have characterized faults of the Appalachian Foldbelt as a means of assessing their potential for reactivation[113]. The faults of the region discussed in this section belong to six of the 13 groups of fault types characterized by Odom and Hatcher:

1. Group 3 Bedding plant thrusts and decollements

2. Group 7 High-angle reverse faults

3. Group 8 Strike-slip faults

4. Group 9 Normal (block) faults

5. Group 11 Structural lineaments (enigmatic faults)

6. Group 13 Faults related to geomorphic phenomena

In Section 2.5.1.1.3, the major and minor faults of the region and their origin are described. Each of the structures belongs to one of the six categories. In a generic sense, only faults of Groups 7, 8, and 11 are considered to have a high probability of natural reactivation, that is, to be seismogenic. Odom and Hatcher do not consider faults of Group 7 to occur in the site region. Although structures like the Clarendon-Linden experienced high-angle reverse slip in the geological past, Odom and Hatcher consider features like these to be more accurately characterized as strike-slip/block faults. Nevertheless, they caution that if a known major earthquake event has occurred near a feature of this type, that possible reactivation cannot be ruled out. Faults of Group 8 would require restoration of the "proper stress regime" if the fault were not annealed. Faults of Group 11 (Section 2.5.1.1.3) must be treated with caution inasmuch as some are known to contain regions of contemporary seismicity (Section 2.5.2). Features of the other three groups are virtually nonseismogenic; however, adjustments or movements at slow strain rates are possible (Section 2.5.1.2.3).

Structures of Groups 3, 7, 8, 9, and 11 may all be subject to induced seismicity if the in situ loading conditions are changed in association with engineering activities, such as reservoir impoundment or hydraulic mining. The Clarendon-Linden fault is a relevant example in the Unit 2 region (Sections 2.5.1.1.3 and 2.5.2.3).

The seismically active faults of the region are described in Sections 2.5.1.1.3 and 2.5.2.3. The effect of seismic events associated with these features on the Unit 2 facility is accounted for in the seismic design for Unit 2.

Time-dependent displacements of small magnitude are possible along large and small faults and fracture zones in regions of high in situ bedrock stress. Under certain conditions, buckling can occur. These stress relief phenomena are considered part of Group 13[113]. This group is very relevant to the Nine Mile Point region and is discussed in Sections 2.5.1.2 and 2.5.4.

<u>Natural Gas Pressures</u> The sedimentary strata underlying Oswego and surrounding counties are known to contain natural gas. Since gas well drilling began in the late 1800s, four gas fields have been identified within a 48-km (30-mi) radius of the site (Figure 2.5-13), namely, the Sandy Creek field, Pulaski field, Fulton/North Volney field, and Baldwinsville field. Although gas quantities and production rates from each of these fields have been historically low, recent increases in gas prices have renewed interest and drilling activity in some of them.

The gas-bearing formation in all these fields is the Trenton Limestone. Due to the generally low primary porosity of this unit, gas accumulation is probably fracture controlled. Interpretation of gravity anomaly data and previous drilling information suggests the possibility of a pinch-out type

structural trap on the flanks of a basement high, underlying the Tug Hill Plateau.

In map view, the gas fields of central New York State can be seen to occur along a north-northeast trend (Figure 2.5-13). Some recent drilling activity[146] to investigate the possibility that these small known fields may be parts of a larger, continuous field, has located gas outside of and south of the Pulaski field along this trend. Historically, wells drilled in the Pulaski field have yielded gas pressures between 11 and 46 kg/sq cm (150 and 650 psi), with initial productions between 11 and 70 kg/sq cm (160 and 1,000 psi), at depths between 200 and 435 m (650 and 1425 ft)[147]. Very minor gas pressures were noted during drilling activities and excavation activities at the plant site. Gas samples from the reactor excavation and intake/discharge tunnel shaft were collected and analyzed by gas chromatography for its constituents. The results showed slightly more than 93 percent of methane and slightly less than 6 percent nitrogen with small amounts of ethane and carbon dioxide. Tests for hydrogen sulfide indicated maximum concentrations of less than 3 ppm hydrogen sulfide, indicating no significant concentration of highly toxic concentrations in the gas. The sulfide concentration in the gas is chemically compatible with the concrete. Higher concentrations (50 ppm) of soluble sulfates in the groundwater were also found compatible with the concrete. The presence of small quantities of natural gas at the site, therefore, does not pose any geologic hazard to the plant site.

<u>Shoreline Erosion</u> Rapid and intermittent to progressive erosion of the southern shore of Lake Ontario has been documented in historic time. The cause of this erosion is twofold. First, erosion is predominantly induced by storms and wave action related to the prevailing north to north-westerly winds. Second, studies in the region have also shown that progressive vertical crustal uplift has occurred during the time period from 13,000 yr ago to recent time (Section 2.5.1.1.4). This movement has been attributed in part to glacio-isostatic rebound as a result of recession of the most recent Pleistocene ice sheet. Consequently, the outlet of the St. Lawrence River is currently rising with respect to the shoreline at a rate of 2 mm/yr (0.08 in/yr) (Section 2.5.1.1.3). Therefore, the lake is gradually, although slowly, encroaching to the south with attendant soil and bedrock erosion.

A dike was built extending from the existing dike in front of Unit 1 on the west to a point where the ground rises naturally to el 80 m (263 ft) east at Unit 2. The maximum probable wave run-up at the site was calculated to be el 80 m (263 ft) (Section 2.4.5). The dike prevents waves from reaching unit structures and thus eliminates the hazard of shoreline erosion at the site.

<u>Landslide Potential</u> Most of the bedrock surface is covered by dense glacial till that is blanketed by vegetation. The presence

of these materials, coupled with the relatively low relief of the area, indicates that landslide potential is nonexistent.

Subterranean Collapse and Attendant Ground Subsidence There is no threat of subterranean collapse and attendant ground subsidence to the plant site because the rock units beneath the site, to depth of about 305 m (1,000 ft), are siliceous and not prone to dissolution.

Volcanic Potential Volcanic potential at the plant site is nonexistent.

Induced Hazards

Bedrock Excavation The flat-lying sedimentary rocks of the Lake Ontario region are characterized by conditions of high, horizontal, and subhorizontal in situ stress (Sections 2.5.1.1.3 and 2.5.4.1.4). In addition, the layered nature of the well-lithified bedrock imparts a pronounced anisotropy to the rock. If an excavation is made into bedrock in this region, then stress relief occurs in two forms: instantaneous, elastic relief, and a viscous, time-dependent relief. The time-dependent relief of stored stress, loosely termed rock squeeze (Section 2.5.4.1.4), typically results in displacements along bedding planes and can pose potential problems to engineered facilities because if the displacements are resisted by a structure, then a stress transfer from the bedrock to the structure can be expected. These induced stresses typically build up in excess of the yield stress of the construction materials and failure can result. The rock squeeze phenomenon is relevant to the Unit 2 site and is discussed in detail in Section 2.5.4.1.

Groundwater Fluctuations If equilibrium conditions in the bedrock are disturbed, such as by excavation, a stress gradient around the perturbation is developed, and time-dependent displacements ensue until equilibrium conditions are attained again. Equilibrium for bedding plane sliding is dependent upon the relationship of two factors: the shear force acting along the bedding, and the shear resistance to sliding. If the vertical loading on a bedding plane (which has a free boundary) is reduced, then the tendency for slippage is increased and the converse is also true. Therefore, phenomena that induce changes of the vertical effective stress are related to the potential for bedding slip displacements. Principal among these phenomena, which are related to man's activity, is the fluctuation of the groundwater table.

No major water level changes at the Unit 2 site or within the vicinity of the site are expected. Considerations of lowering the water level of Lake Erie (International Lake Erie Regulation Working Committee, 1982) and the effects on water levels in Lake Ontario and the St. Lawrence River are in progress. A rise in the Lake Ontario level would favor the tendency to reduce the vertical effective stress in the bedrock. However, the report

concludes that although such regulation would increase the maximum water level of Lake Ontario, the overall economic impact would be negative, and termination of the study is recommended[148].

Extraction of Geologic Materials The potential for surface and subsurface mining that would directly affect the Unit 2 site and foundation stability is negligible. There are no economically viable evaporite deposits beneath the site radius, and no mineable ores have been discovered at or near the site (within about 80 km, 50 mi).

Carbonate rocks crop out no closer than 16 km (10 mi) from the site, and sand and gravel deposits are small and localized in the Nine Mile region. Blasting in rock quarries is not expected to be felt at the site or to exceed the background intensity of the natural seismicity of the region. Excavation of sand and gravel has no effect on the site.

2.5.1.2 Site Geology

This section is a detailed summary of the geology of the Nine Mile Point site. With the exception of Sections 2.5.1.2.4 and 2.5.1.2.5, the information presented here is specifically related to the site proper. Section 2.5.1.2.4 addresses the Demster Structural Zone and associated structures (Figure 2.5-25). The relationships between the Demster Structural Zone and the smaller-scale structures observed in the immediate vicinity of the site are discussed in Section 2.5.1.2.5.

The surface geology of an 8-km (5-mi) radius around the Nine Mile Point site is presented on a map in Figure 2.5-25. A cross section of this area is presented in Figure 2.5-26.

2.5.1.2.1 Physiography

The site is located approximately 10 km (6.2 mi) northeast of Oswego, NY, and is bounded on the north by Lake Ontario and on the south by Lake Road (Figure 2.5-27). It is adjacent to Unit 1 to the west and James A. FitzPatrick Nuclear Power Plant to the east. Much of the site area was reworked and covered with fill during the early 1940s for Camp Oswego, U.S. Military Reservation. The site lies near the Erie-Ontario Lowlands subdivision of the Central Lowlands Physiographic Province (Section 2.5.1.1). Overall, the site morphology reflects a bedrock surface modified by repeated Pleistocene glaciations that eroded weathered rock and deposited glacially-derived sediments. However, the site does not display any of the prominent drumlins that are characteristic of the Erie-Ontario Lowlands. The site lies within the Lake Iroquois lake plain, and lacustrine sediments of possibly two pre-Lake Ontario water levels subdue the surface of glacial till overlying bedrock. Surface drainage is poorly integrated as evidenced by bogs and swampy land in low-lying areas.

The topography of the site reflects the slightly irregular bedrock surface (Figure 2.5-28) modified by glacial drift. It is flat to gently rolling with elevations ranging from 75 m (246 ft) above msl along the lake shoreline to 82 m (270 ft) near the southern end of the site (Figure 2.5-27).

There are no areas of landsliding, subsidence, uplift, or collapse at the site. Cavernous or karst terranes are not present in the site area, because the bedrock is siliceous and not prone to dissolution.

2.5.1.2.2 Stratigraphy

Unlithified Sediments

Unlithified sediments have been identified and their approximate ages determined on the basis of stratigraphic position, sediment type, pollen stratigraphy, C-14 dating, grain size distribution, and mineralogical analyses[94]. A generalized stratigraphic column of these sediments is shown on Figure 2.5-29. The sediments in order of decreasing age are: till, Lake Iroquois clay and silt, Sandy Creek time-equivalent sands, and marls, silts, and peat.

Figure 2.5-30 is a generalized cross section across the southern portion of the site, illustrating the facies distribution in the unlithified sediments. These surficial sediments in the site area were examined and mapped in the Cooling Tower Excavation, Pit 1, and Trenches 3, 4, and 5 (Figure 2.5-28). Detailed cross sections from the trenches and building excavations are presented in Section 2.5.4.3. Descriptions of the units composing the stratigraphic sequence of unlithified sediments are presented below.

Till Four varieties of till are present on the site. Two types of gray till, a brown till, and a proglacial lake till are distinguishable on the basis of color, texture, and composition. Field relationships and mineralogic analyses indicate that till from only the Late Wisconsinan glacial stage is present at the site. These tills were most likely deposited immediately prior to and during the existence of Lake Iroquois, and are probably equivalent in age to tills deposited during the Port Huron glacial advance (12,900 to 12,000 yr B.P.)[94].

Generally, gray till up to 1.8-m (6-ft) thick has been deposited across the site and directly overlies either bedrock or, in places, a 1-in layer of gray sand. Two units of gray till are distinguished primarily by the size, angularity, and composition of the rock clasts[94]. Notably, one unit contains exotic clasts.

A distinctive brown till as much as 3-m (10-ft) thick overlies the gray till and bedrock in the southeastern portion of the site. Locally, the brown till also interfingers with, and is in vertical contact with, the gray till (Figure 2.5-29). The brown

till consists of rounded to subrounded exotic rock fragments in a fine-grained silty sand matrix. It is distinguished from the gray till on the basis of color, inclusions of stratified drift, larger percentages of well-rounded foreign clasts, and a coarser texture, and may, in part, represent an ablation till[94].

Locally, up to 0.6 m (2 ft) of till that was deposited in an ice marginal lake overlies both the gray till and the bedrock (Figures 2.5-31 and 2.5-32). This till consists of a dark gray silty sand with subrounded rock fragments of mostly gray sandstone and minor percent black limestone. It interfingers with and is overlain by as much as 0.3 m (1 ft) of light gray stratified silt and sand with some subrounded to rounded gravel and light gray silt clasts. This till is poorly stratified and grades upward into the laminated clayey silts of Lake Iroquois. It was deposited as the ice margin receded northward and sediment from the receding ice was reworked and redeposited in the Lake Iroquois basin[94].

<u>Lake Iroquois Deposits</u> The deposits of proglacial Lake Iroquois are deep water sediments up to 1.2-m (4-ft) thick which directly overlie gray till, bedrock, or ice marginal lake till where they occur in the site area. These sediments consist of laminated to massive, reddish brown or gray clayey silt or silty fine sand with lenses and laminations of fine to medium sand and a little gravel.

The age of Lake Iroquois is bracketed by C-14 dates from various locations along the shoreline. At Lewiston, NY, wood from a spit was dated at 12,600 ± 400 radiocarbon yr B.P.[136]. This date represents a maximum age for the initiation of Lake Iroquois. Karrow et al[149] date a post-Iroquois lake level stand, 12 m (40 ft) below the crest of the Iroquois beach in the Hamilton area, at 11,570 ± 260 radiocarbon yr B.P. This date gives a minimum age for the extinction of the Iroquois high stand. If the deep water sediments onsite are Lake Iroquois sediments, they probably were deposited between 12,600 ± 400 and 11,570 ± 260 radiocarbon yr B.P. However, no organic material was found within the Iroquois sediments that was suitable for C-14 dating. Hence, the minimum age of these sediments is inferred as pre-Sandy Creek (Figure 2.5-29).

The pollen stratigraphy of the deep-water lacustrine sediments has been correlated with C-14 dates, and it also suggests that the deep water sediments were deposited approximately 12,000 yr B.P.[94]. These Iroquois sediments are generally low in total pollen content; however, the pollen that does exist is dominantly spruce and pine. Data from samples AS-22 and AS-16 (Figure 2.5-33) illustrate a decrease in the percent of spruce pollen found in the shallow water deposits (Sandy Creek) when compared to the underlying deep water Iroquois sediments in Trench 3 (Figure 2.5-28). Sample locations are shown on Figures 2.5-34 and 2.5-35. The relative percentages of spruce pollen suggest

that the shallow water sediments (Sandy Creek) are similar in age to the upper spruce pollen zone (Zone A4, about 10,000 yr B.P.), and that the Iroquois sediments are similar in age to the lower spruce pollen zones (Zones A1 and A2, about 12,000 yr B.P.).

<u>Sandy Creek Equivalent Sand</u> In the southern portion of the site, shallow water deposits overlie Lake Iroquois sediments, till, or bedrock. The shallow water sediments consist of up to 0.9 m (3 ft) of thin bedded silt, fine to medium sand, and clay, and do not occur above el 82 m (270 ft). The bedding varies from planar to wavy rippled and cross-laminated ripple drift.

The identification of the shallow water sediments as Sandy Creek time-equivalent deposits is based on detailed studies described in previous reports[94]. The Sandy Creek shoreline was formed during the lowering of lake level in the Ontario Basin to the Admiralty stage[143]. Terraces between el 82 and 88 m (270 and 290 ft) along streams in the site area and near Mexico Bay were identified from air photos, topographic maps, and field reconnaissance and are inferred to be equivalent to Sutton's shoreline identification further north. Just west of Demster Beach a distinct break in slope and change in air photo tone occurs at el 82 m (270 ft). The slope gradient above 82 m (270 ft) is steeper than the slope gradient below this elevation. This break in slope as well as the terraces suggests a possible relation to the Sandy Creek stillstand, therefore placing the shoreline at approximately el 82 m (270 ft) in the Mexico Bay area.

The relative ages inferred for samples of the shallow water sediments and the aforementioned pollen stratigraphy indicate that they are representative of the middle to upper A pollen zone (Figures 2.5-30 and 2.5-33). This, in conjunction with the observation that these sediments have not been identified above el 82 m (270 ft) at the site, suggests that the shallow water sediments were deposited during or just prior to the Sandy Creek lake level stillstand.

<u>Marl</u> The shallow water deposits locally grade into a marl unit up to 0.6 m (2 ft) thick (Figure 2.5-30). The light tan marl consists of silt to fine sand-size calcite fragments, clay, and abundant freshwater fossil fragments. The macrofossil content increases upward culminating in a highly fossiliferous zone in the upper 7.6 cm (3 in). The pollen assemblage within the marl is similar to that of the shallow water deposits, suggesting that it also falls in the A zone (Figure 2.5-33)[94]. A C-14 date of 12,545 ± 330 radiocarbon yr B.P. was obtained from pelecypod and gastropod shells in the top 2 in of the marl. C-13 analysis suggests that the fossils may have been naturally contaminated by older carbon. Thus, the C-14 age probably is older than the true age of the marl; however, the error of the date would probably be less than about 2,000 yr[94]. This suggests the marl is

time-equivalent with the lower part of the peat in Trench 4 (Figure 2.5-30).

Silt Outside the area of marl deposition, the shallow water deposits grade into nonorganic massive to medium bedded silts and silty fine sands (Figure 2.5-30) which in turn grade into organic silts. This sequence represents a transition from a high energy environment of the Sandy Creek time-equivalent sands to marshy areas above lake level. In part, these may be eolian silts and fine sands and are probably time equivalent with the marl.

Peat In the Cooling Tower Piping Trench (Figure 2.5-30, CTT-1) and in Trench 4, peat is exposed in thicknesses up to 0.9 m (3 ft). However, in Trench 4, the peat is restricted to a depression on the west wall where it overlies and is in gradational contact with the shallow water deposits identified as Sandy Creek time-equivalent sands.

C-14 dates obtained using samples from this peat (Figure 2.5-30) provide minimum absolute ages of the shallow water deposits. One reliable date of 11,260 ± 190 radiocarbon yr B.P. was obtained using samples from the basal woody peat[94]. The basal woody peat is overlain by approximately 20 cm (8 in) of peat composed almost entirely of sphagnum moss. Samples of this mass yielded ages of 10,400 ± 255 and 10,060 ± 125 radiocarbon yr B.P.[94], which are consistent with the age of the underlying wood, and the pollen assemblage, which suggests the base of the peat is in the A pollen zone. The decrease in spruce pollen and the increase in hemlock and hardwood (Figure 2.5-33) suggests that the top of the peat falls close to the B-C pollen zone boundary or about 8,500 yr B.P. Therefore, deposition of the peat began before 11,260 ± 190 radiocarbon yr B.P., continued at least through 10,400 ± 255 yr B.P. and possibly through 8,500 yr B.P. Thus, a minimum absolute age can be inferred for the underlying shallow water sediments (Sandy Creek sands) as 11,260 ± 190 radiocarbon yr B.P. Because the peat is overlain by artificial fill, no minimum age of peat deposition could be obtained.

Bedrock Stratigraphy

The strata cropping out in the general vicinity of the Nine Mile Point site constitute part of a major Late Ordovician deltaic complex that originated during a progressive uplift along the northern and central Appalachians. This uplift signaled the beginning of a mountain building episode (the Taconic Orogeny), which resulted in the westward regression of the Ordovician sea, and exposure of the sediment source area. The sediments were transported generally westward, forming a wedge-shaped clastic mass in New York, western Pennsylvania, and northeastern West Virginia.

The clastic wedge (Figure 2.5-5) is characterized by an upward trend toward a shallowing environment of deposition that culminated in the deposition of the terrestrial Ordovician

Queenston Formation. The Queenston represents westward migrating deltaic sediments deposited on a broad alluvial plain. The deposits graded westward into the terrestrial and shallow marine, unfossiliferous Oswego Sandstone, which in turn passes into the littoral and infralittoral sediments of the Lorraine Group. Thus, these Late Ordovician clastic sediments represent a series of transitional marine-terrestrial, depositional regimes that show an overall east-to-west regressive pattern[150]. During any given time interval the subaerial environment of deposition in the eastern part of the wedge had its subaqueous equivalent in the western part.

The results of drilling programs completed for stratigraphic and other phases of investigation at the Nine Mile Point and FitzPatrick sites prior to 1972 are summarized and discussed in the Nine Mile Point Unit 2 PSAR[82]. All explorations using borings at the site are described in Section 2.5.4.3. Drilling programs completed between 1976 and 1981 have explored a stratigraphic interval of approximately 135 m (442 ft) (Figure 2.5-36) and have yielded more detailed information regarding site bedrock stratigraphy than was available at the time of completion of the PSAR. Although the lithic character of the bedrock units at Nine Mile Point is distinct, the contacts between them are gradational and not readily recognizable. Hence, the stratigraphic terminology for the supplementary geologic exploration, as well as the positions of stratigraphic contacts selected, are different from those used in the earliest, general exploration at the site.

Stratigraphic Nomenclature The stratigraphic nomenclature at the formation level for the Unit 2 site is in accordance with that used by other workers in the region[81,150-152]. However, the positions of these contacts and of informal subdivisions of the formations are at variance. This was brought out following a major geologic investigation at a proposed site for a nuclear facility near New Haven, NY, for New York State Electric and Gas in 1978.

Dames & Moore personnel examined the rock cores from the borings drilled for the New Haven study site to reconcile the conflicts of opinion. It was evident that the New Haven and Nine Mile Point stratigraphic columns are identical. At each site, the column has been subdivided into identical correlative units; however, there is a difference in the stratigraphic nomenclature employed. Figure 2.5-36 compares the two different terminologies. For additional information and further clarification, refer to the response to NRC Question F231.18d in Appendix 2N. It should be noted that the stratigraphic subdivision (i.e., the placement of the unit boundaries) is identical. The identification of the Oswego/Pulaski interface at the Nine Mile Point site is in accordance with its identification in the excavations of the James A. FitzPatrick site by Professor Newton E. Chute, an expert in the stratigraphy of upstate New York[153].

Figure 2.5-36 is a composite stratigraphic column illustrating the lithology and stratigraphic succession of the explored bedrock section. The boreholes, which penetrated various portions of the section, as well as the stratigraphic positions of the main structures of the Unit 2 power complex, are also depicted. The identification and correlation of stratigraphic units is based on examination of the rock cores and the use of downhole geophysical logs.

The explored strata belong to two major stratigraphic divisions of the previously-mentioned clastic wedge. These divisions are the Oswego Sandstone and the Upper Lorraine Group (Figure 2.5-36). The Lorraine Group explored onsite is represented by the Pulaski Formation and a portion of the Whetstone Gulf Formation.

Oswego Sandstone The Oswego Sandstone consists of unfossiliferous, greenish-gray, medium- to fine-grained, massive sandstone and is rather uniform and monotonously similar. Thin black shale and siltstone beds are minor, although clay galls (shale intraclasts) are common. The sandstone is characteristically composed of subangular and subrounded quartz grains, sometimes with well-rounded rock fragments and a small amount of feldspar and clay matrix. Throughout the Oswego Sandstone, numerous sedimentary structures were observed, which suggests that the strata were deposited in shallow water in a near-shore environment.

The lowermost 5 m (15 ft) of the Oswego Sandstone is termed the Transition Zone, where the bedrock consists of alternating, laminated to thick-bedded, fine- to medium-grained sandstone, argillaceous sandstone, dark siltstone, and shale (Figure 2.5-36). Only trace fossils, such as feeding trails and worm burrows, are present. There is a general trend toward bed thickening and decreasing clay content, upward through the unit, reflecting a change in the environment of deposition.

The top of a distinctive 15- to 30-cm (6- to 12-in) thick bed of black shale has been used as the base of the Oswego Sandstone. This bed occurs about 0.6 m (2 ft) above the uppermost fossiliferous horizon within the strata underlying the Oswego Sandstone (Figure 2.5-36), and has been identified in most of the borings and in the reactor excavations for Unit 2[94]. The thickness of the Oswego Sandstone thus defined varies from 10 m (35 ft) near the lakeshore to nearly 37 m (120 ft) in the southern portion of the site at boring OC-1[94]. This increase in thickness results from the combined effects of topographic relief and a regional gradient of approximately 9.5 m/km (50 ft/mi) (0.5 deg) to the south.

Lorraine Group The Lorraine Group is composed of two main intergrading lithologic units, the Pulaski Formation and the underlying Whetstone Gulf Formation. Both consist of a

fossiliferous sequence of alternating black siltstone and shale, gray sandstone, and dark gray argillaceous sandstone. The overall sequence is characterized by abrupt lithologic variations and slight gradation of one lithic type into another. There is a subtle upward increase in the amount of arenaceous material and in the thickness of bedding throughout the Lorraine Group. Also, the amount of shale increases in the middle and lower portions of the Pulaski Formation and in the upper portions of the Whetstone Gulf Formation.

<u>Pulaski Formation</u> The Pulaski Formation has been informally subdivided into three units on the basis of lithologic composition and bedding thickness (Figure 2.5-36). The youngest division, Unit A, is medium- to thick-bedded and consists of mottled, dark gray argillaceous sandstone (referred to as graywacke) interbedded with light gray sandstone and few beds of dark gray shale and siltstone. Bedding is irregular, and bedding thickness seldom exceeds 46 cm (18 in). The sandstone beds contain marine fossils and transported fossil debris. It is characterized by sedimentary structures that represent the transition from the predominantly shallow marine sedimentation of the underlying strata to the tidal flat and barrier bar environment of the overlying Oswego Sandstone. The stratigraphic thickness of Unit A is approximately 12 m (40 ft).

Unit A is underlain by 4.5 to 6 m (15 to 20 ft) of strata designated as Unit B. The top of this unit is marked by a 0.9- to 1.2-m (3- to 4-ft) thick bed of light gray, medium-grained, well sorted, fossiliferous sandstone. The unit as a whole consists of interbedded light gray sandstone, black siltstone, and shale with few beds of dark gray mottled graywacke. The bedding is regular and massive and commonly attains thicknesses of several feet.

Unit C consists of dark gray to black siltstone and shale interbedded with light gray sandstone. Approximately 6 m (20 ft) below the top of the unit, these sandstone intercalations become more numerous and thicker. Throughout Unit C, the bedding is characteristically thin with a trend toward increasing bed thickness with depth. This trend and the absence of intercalations of argillaceous sandstone distinguish this unit from the overlying sequences. The total thickness of Unit C is approximately 12 m (40 ft).

The thickness of the Pulaski Formation (Units B and C) gradually increases approximately 1.5 m (5 ft) from east to west in the vicinity of the power block. This thickness increase is believed to be related in part to the original sedimentary configuration of the formation. However, the occurrence and nature of deformational structures in this portion of the stratigraphic section is also partly responsible for the observed thickening (Section 2.5.1.2.3).

Whetstone Gulf Formation The Whetstone Gulf Formation underlies the Pulaski Formation and extends to the greatest explored depth at the site (Figure 2.5-36). The gross composition of the formation is quite similar to Unit C of the Pulaski Formation. It consists of a well-bedded sequence of dark gray shale, siltstone, and light gray medium-grained sandstone. Fossils are common and occur in silty shale and siltstone as well as in cross-bedded sandstone. The silty shale tends to grade downward into less silty, fissile black shale.

In the upper portion of this formation, sandstone intercalations are relatively thin and far apart. They become more common in the middle portion of the formation where they form beds with thicknesses up to several feet. In the lowest part, the sandstone intercalations are thinner, but they constitute a significant percentage of the overall composition of the section. The sandstone content can be used to divide the formation into two units. The top of the formation, Unit A, has few sandstone intercalations and is approximately 9 m (30 ft) thick. The rest of the explored section is categorized as Unit B.

2.5.1.2.3 Structure

The Nine Mile Point site is situated within the Eastern Stable Platform Tectonic Province (Sections 2.5.1.1 and 2.5.2.2). The site area is considered relatively tectonically stable and is free of major tectonic structures. The relatively undeformed nature of the rocks at the site reflects this stability. Since the beginning of Paleozoic time, about 570 million yr ago, the site has been subjected to little more than epeirogenic crustal movements. The broad-scale effects of these movements have resulted in the accumulation of a thick sequence of sedimentary rocks at the site and the subsequent rotation of these strata to the south with an average gradient of 9.5 m/km (50 ft/mi) (0.5 deg). During the Cenozoic Era, the site bedrock was also affected by broad crustal warping induced by isostatic loading as a result of continental glaciation.

The foregoing geologic processes have produced a number of relatively small structures observed at the site. These structures consist of systematic fracture sets, moderately to steeply dipping faults, and shallowly dipping faults with associated low amplitude folds. Investigations of the age, origin, extent, and significance of these geologic structures were conducted at the site[94,154] and submitted to the NRC. Moreover, additional information was provided as formal responses to questions regarding the 1978 report from the NRC in October 1979 and July 1980 (Questions 361.1 through 361.35; Docket No. 50-410). This section provides a synopsis and synthesis of information pertaining to the geologic structures on the Unit 2 site.

Fractures

The attitudes of 282 fractures were measured at Nine Mile Point to detect site-wide fracture trends. Poles to these fractures are plotted and contoured on lower hemisphere equal area projections (one each for the northern and southern parts of the site) (Figures 2.5-37 and 2.5-38). In the northern part of the site two predominant vertical fracture sets are present and are oriented N42W and N72E (Figure 2.5-37). In the southern part of the site (cooling tower piping trench) similar fracture trends are present. There, the greatest number of fractures strike about N50W, effectively the same northwest trending set as in the northern part of the site. Smaller concentrations of data points define sets with average strikes ranging between N68E and N24E (Figure 2.5-38), which are all considered the equivalent of the east-northeast oriented set in the northern part of the site. Because of their predominance across the entire site, the northwest and east-northeast trending vertical fractures will be the focus of the remainder of this discussion.

Members of both fractures sets are very similar in character. They are generally nearly planar and developed approximately normal to bedding. Fractures of each set onsite commonly contain calcite and sulfide mineralization. They are best developed within the standstone members of the stratigraphic section. At boundaries with more argillaceous strata, the fracture attitudes either change or they terminate. Very few members of these two fracture sets extended continuously through significant thicknesses of strata. However, their occurrence was noted throughout the stratigraphic section.

The acute angle between nearly vertical fracture sets is about 58 to 70 deg. This suggests that the fractures formed, at least locally, as conjugate fractures with the greatest and least principal compressive stresses oriented west to west-northwest and north to north-northeast, respectively.

No data are available from the site that would permit a direct determination of the age(s) of the major fracture sets. However, studies by Parker[83,85], Engelder and Geiser[49,87], and Engelder[90] provide the basis for interpreting the site fractures as being related to late Paleozoic Alleghanian tectonics (Section 2.5.1.1.3). Parker[83] concluded that the three regional fracture sets (I, II, III) are all older than the time of emplacement of numerous periodotite and kimberlite dikes in the early and medial Paleozoic rocks in central New York. One such dike, at Portland Point, NY, near Ithaca, yielded a potassium-argon (K/Ar) mineral age on phlogopite of 155 ± 4 million yr[155]. Zartman[17] has obtained similar dates with the rubidium-strontium (R/SR) technique (Section 2.5.1.1.2). Investigations by Engelder and Geiser[49] are in accordance, except that Fracture Set III is interpreted to postdate Alleghanian tectonics, forming, instead, in response to the contemporary stress field[90]. Fracture Set III strikes east-northeast nearly ubiquitously. Nevertheless, the fractures onsite are equivalent in their time of origin to the

older fractures of Sets I and II of Parker[83] and Ia, Ib, and II of Engelder and Geiser[49].

Mineralization of the fractures onsite provides further information regarding the age of the fractures, which is consistent with the foregoing conclusion[94] (Section 2.5.1.2.6). A study of the fluid inclusion temperatures from the calcite on fracture surfaces indicates that these minerals were deposited by fluids with a temperature range of approximately 160° to 100°C (320° to 212°F). Based on these temperatures, it may be inferred that these minerals were formed at burial depth of 2 to 3 km (1.2 to 1.8 mi). It may also be inferred that such great burial depth is more consistent with the pre-Jurassic age suggested by Parker[83] than the time of development of the contemporary stress field as inferred by Engelder[90]. For additional information and further clarification regarding temperature-depth relationships of the minerals in fractures and stress regime changes recorded by these minerals, refer to the response to NRC Question F231.17 in Appendix 2N.

High-Angle Faults

Three high-angle faults striking west-northwest, namely, the Barge Slip fault, the Drainage Ditch fault, and the Cooling Tower fault, occur at or adjoining the Unit 2 site (Figure 2.5-28). The Barge Slip fault dips 60 to 65 deg southward; the other two faults dip 55 to 70 deg to the north. All three faults have similar structural characteristics. Most of the information concerning these faults was gained from a detailed investigation of the Cooling Tower fault[94]. However, all available surficial exposures of the Barge Slip and Drainage Ditch faults were examined, together with reports of previous investigations of these structures[93,153].

The high-angle faults display several common characteristics. All three appear to be subvertical strike-slip faults in surficial exposures within the Oswego Sandstone. However, they display the geometry and displacement of a normal fault within the Lorraine Group. They each contain occurrences of calcite and sulfide mineralization associated with the strike-slip and normal slip deformational fabrics. Furthermore, the homogenization temperatures and paragenesis of the mineralization associated with the different episodes of deformation are similar for each fault.

Some aspects of the geometry and deformation along the Drainage Ditch and Cooling Tower faults (the two north-dipping faults) differ from those of the Barge Slip fault. Each of these faults is coincident with the axial plane of an asymmetric chevron fold or monocline. They exhibit reverse slip, stratigraphic displacements in addition to the aforementioned deformation. Also, in surficial exposures of each of the north dipping structures, the plane of the fault was displaced by translation of the adjacent strata along bedding planes (bedding plane slip).

Of the three high-angle faults, only the Cooling Tower fault had clearly deformed the overlying Pleistocene sediments. The effects and mechanism of this deformation are presented under the heading Reverse-Slip Displacements.

The total inferred lateral extent of the high-angle faults is represented by the fault traces shown on Figure 2.5-28. The fault traces are assumed to be relatively linear. This assumption proved to be valid by direct observations elsewhere, especially along the Cooling Tower fault. Few data are available regarding the southeast extent of the Barge Slip fault. The minimum lateral extent of this feature is about 670 m (2,200 ft) (Figure 2.5-28). The west-northwest extent of the Barge Slip fault can only be inferred to be located west of its intersection with the excavation for the lake water tunnels of the James A. FitzPatrick plant.

The inferred western extent of the Drainage Ditch fault is located as shown on Figure 2.5-28 because the lake water tunnels for the Unit 2 site did not encounter the fault. The east-southeastern extent of the Drainage Ditch fault was determined when the extension of the compression buckle or tepee fold at the James A. FitzPatrick plant site was investigated[93,153] by examination of aerial photographs, seismic refraction surveys, and test excavations. These studies led to the conclusion that, east of the James A. FitzPatrick facility, the buckling (of the bedrock layers) dies out and the fracture resolves into a local system of close jointing[153].

Seven trenches and pits were dug to investigate the lateral extent of the Cooling Tower fault. The western extent of the fault is inferred, as shown, because Trenches 1 and 2 revealed no evidence of faulting (Figures 2.5-28 and 2.5-28A). It is difficult to precisely locate the east-southeastern extent of the Cooling Tower fault. However, most of the characteristics of this fault, including magnitude of displacement and degree of cataclasis, appears to be similar to those of the Drainage Ditch fault. Hence, it is logical to assume that the length of the Cooling Tower fault is not significantly different from the length of its analog. This reasoning suggests that the Cooling Tower fault does not extend beyond the location shown on Figures 2.5-28 and 2.5-28A.

The length of the three high-angle faults, the Barge Slip, Drainage Ditch, and Cooling Tower, are discussed further in the response to NRC Question F231.4 in Appendix 2N.

At the request of the NRC, the lengths of the Cooling Tower and Drainage Ditch faults were investigated by performing proton magnetometer surveys across the established traces of both faults. The surveys found no magnetic gradients that could be interpreted to represent faults across the traces of the structures or at a hypothetical projection to the basement

(Response to NRC Question 361.26, Docket No. 50-410). For additional information and further clarification, refer to the response to NRC Question F231.5 in Appendix 2N.

The depths of the high-angle faults are inferred on the basis of information gathered from the subsurface investigation of the Cooling Tower fault (Section 2.5.4.3)[94]. This fault was investigated by drilling two rows of closely spaced vertical boreholes perpendicular to the strike of the fault (Figure 2.5-28). Detailed stratigraphic and structural logs were prepared from the cores extracted from the boreholes[94]. From these logs, detailed correlation charts were prepared (Figures 2.5-39 and 2.5-40), and these charts were used to prepare geologic cross sections (Figures 2.5-41 and 2.5-42). As shown on the cross sections, the principal structural element in the subsurface is a 60- to 65-deg northward dipping discontinuity. The structure extends from the top of the bedrock down to the explored depth of 82 m (270 ft). Below a depth of 60 m (200 ft), the sense of fault displacement is normal and the magnitude of offset is about 0.3 m (1 ft). Because of the similar structural character of the high-angle faults, it is inferred that they all extend to a depth similar to that attained by the Cooling Tower fault. Based on the small magnitude of the observed displacement and the relatively short length of this structure, it seems inappropriate to infer that this fault extends much deeper than 82 m (270 ft). Therefore, it is believed that the high-angle faults only extend to depths of several hundred feet and that they do not extend to the basement. The results of the magnetometer survey discussed above support this inference because these reflect only basement effects, and there is no evidence of any offset. The depth of the normal displacement of the Cooling Tower fault is addressed further in the response to NRC Question F231.4 in Appendix 2N.

The investigations of the three high-angle faults at and adjoining the Unit 2 site revealed evidence of a history of multiple displacements of different senses along the faults[94]. It was found that all these faults had moved simultaneously. They initially experienced strike-slip followed by normal slip. In addition, the two northward-dipping faults (Cooling Tower and Drainage Ditch faults) have been affected by buckling of the bedrock along the fault planes both laterally and with depth. The buckling mechanism resulted in the development of reverse-slip displacements along the fault planes, but restricted to the upper 60 m (200 ft) of the bedrock mass. Each episode of deformation is discussed below.

<u>Strike-Slip Deformation</u> The three high-angle faults commonly display characteristics indicative of strike-slip deformation. The amount of lateral displacement is known only for the Cooling Tower fault. In one exposure a sedimentary channel crest (an interfluve ridge) within the Oswego Sandstone was displaced 0.9 m (3 ft) in a left-lateral sense across this fault (Figure 2.5-43). Indirect evidence of strike-slip faulting, such as the character

of the shear and fracture fabric in proximity to the faults, as well as the occurrence of both horizontal slickensides and slickensides with gentle rakes on fracture surfaces, are also present.

In surficial exposures, the high-angle faults are vertical, or nearly so (Figure 2.5-44). However, subsurface investigation with vertical borings demonstrated the presence of a fault dipping approximately 60 deg. Two inclined borings (A-1 and A-2, Figure 2.5-41) were drilled to investigate whether a vertical strike-slip fault, independent of the moderately dipping fault, also existed at depth. The angle borings demonstrated that a vertical fault does not exist below a depth of 27 m (90 ft) (Figures 2.5-41 and 2.5-42). Apparently, the vertical, strike-slip portion of the fault exists only in the massive strata of the Oswego Sandstone.

Strike-slip displacements predate other deformations that occurred on the high-angle faults. This interpretation is supported by both mineralogical and structural relationships[94]. The strike-slip faults probably formed at the same time as the systematic fracture sets at the site. Both the fracture sets and the strike-slip faults contain vein mineralization with homogenization temperatures (160° to 120°C, 320° to 248°F) similar to those determined for quartz grain clasts in the host rock (176° to 147°C, 349° to 297°F). This indicates that this deformation occurred after diagenesis and represents the onset of brittle bedrock deformation. For additional information and further clarification regarding interpretation of homogenization temperatures of fluid inclusions in quartz grain clasts, refer to the response to NRC Question F231.6 in Appendix 2N.

<u>Normal-Slip Deformation</u> Each of the high-angle faults also exhibits some characteristics of normal-slip deformation. Normal-slip stratigraphic displacements on the Barge Slip and Cooling Tower faults are 40 cm (16 in) and 15 to 30 cm (6 to 12 in), respectively. No normal-slip displacement was detected along the Drainage Ditch fault[94]. However, several indirect indicators of normal-slip were observed along these faults. Shear fracture geometry consistent with normal faulting was observed in a number of exposures[94]. Moreover, slickensides with steep rakes are present on some of these surfaces as well as on the main shear surfaces of the brecciated zones. Drag folds with normal shear sense (Figures 2.5-45 and 2.5-43) were detected during both the surface and subsurface investigations of the Cooling Tower fault[94]. Many fractures associated with normal-slip deformation contain calcite and sulfide mineralization.

The results of field studies indicate that normal-slip deformation postdates strike-slip faulting but predates other deformations that affected the high-angle faults[94]. Structural relationships such as the overprinting of slickensides with steep rakes on slickensides with gentle rakes, and the truncation of

strike-slip fracture sets by shear zones with a normal-slip fabric (Figure 2.5-46), suggest that normal-slip followed the strike-slip deformation. Mineralization studies at the site confirm this relationship[94]. Homogenization temperatures of calcite mineralization associated with normal fault deformation fabric (116° to 73°C, 249° to 163°F) were typically lower than those associated with diagenesis and strike-slip faulting (176° to 120°C, 349° to 248°F). Based on the paragenetic sequence developed for the site (Section 2.5.1.2.6), mineralization of the lower temperature range was emplaced later than diagenesis and strike-slip faulting.

Bedding-Plane-Slip Displacement The effects of displacements resulting from bedrock translation along bedding planes has been noticed at numerous locations at the site. Bedding-plane-slip accompanied reverse-slip displacements on the high-angle faults as well as the development of the low-angle thrust structures. The age and relationship of bedding-plane-slip to these deformations is included in the discussions specifically addressing these topics and also in the response to NRC Question F231.16 in Appendix 2N. The maximum depth at which bedding-plane-slip has been detected is addressed in the response to NRC Question F231.9 in Appendix 2N.

Excluding the thrust structures, the most prominent examples of bedding-plane-slip displacement were observed in the surficial exposures of the high-angle faults. These displacements distorted the original configuration of the faults, resulting in considerable variation in the thickness and attitudes of the breccia zones of these originally steeply dipping faults (Figure 2.5-47).

Reverse-Slip Displacements The Drainage Ditch and Cooling Tower faults are both coincident with the axial plane of an asymmetric chevron-like fold or monocline (Figures 2.5-48 through 2.5-50). These folds resulted from a third episode of deformation occurring along the north-dipping high-angle faults. It was characterized by reverse-slip displacements accompanied by bedding plane slip and dilation of the bedrock within 60 m (200 ft) of the surface. Field evidence clearly indicates that this deformational episode consisted of two phases: 1) preglacial buckling along high-angle faults, and 2) postglacial buckling on the fault.

The total reverse separations across the Drainage Ditch and Cooling Tower faults are 0.46 m (1.5 ft) and 1.5 to 1.8 m (5 to 6 ft), respectively[94]. However, the reverse stratigraphic separations along the shear zones in surficial exposures vary from zero to 0.9 m (3 ft) (Figure 2.5-43). These displacements are less than the total separation because they do not include all the reverse separation resulting from monoclinal rotation. The shear sense indicated by drag folds, within the breccia zones (Figure 2.5-47) and in the adjacent bedrock (Figure 2.5-51), is consistent with this reverse displacement sense.

At five exposures along the Cooling Tower fault, the layered sequence of glaciolacustrine sediments was deformed. The deformation occurs generally within a 6-m (20-ft) wide zone straddling the bedrock fault (Figure 2.5-34). The numerous small-scale deformational features in the sediments consist of fluidized flow and sediment flow structures, as well as small faults. The most prominent of these faults exhibits maximum displacements of several inches. Structural analysis[94] indicates that the deformation was accomplished by relative vertical movements consisting of broad arching and monoclinal flexuring of the sediments resulting from reverse-slip displacements on the bedrock fault. In Pit 1, the Cooling Tower Trench, Trench 4, and Trench 5 (Figure 2.5-28), the vertical separation, caused by arching and generally accompanied by compensatory normal faults in the sediments, is estimated to range from 0.15 to 0.30 m (0.5 to 1 ft) (Figures 2.5-31 and 2.5-32). In Trench 3 the separation that resulted from monoclinal flexuring is 0.82 to 0.99 m (2.70 to 3.25 ft) (Figure 2.5-34). As a result of the flexuring at this location, many small-scale normal, thrust, and high-angle reverse faults developed in the sediments, especially in the area of the short limb of the monocline (Figure 2.5-34).

The subsurface character of displacements on the Cooling Tower fault was also the subject of a detailed investigation[94]. The following discussion summarizes the results of that investigation; however, more detailed discussion may be found in the referenced report.

Figures 2.5-41 and 2.5-42 depict the configuration of the Cooling Tower fault in the subsurface. Relationships between stratigraphic displacement and depth at both drilling sites are illustrated on Figures 2.5-52 and 2.5-53. In the upper part of the section, the structure resembles a kink band or monocline, consistent with the surficial exposures. The reverse stratigraphic separation of about 1.7 m (5.5 ft) is accomplished mainly by southward rotation of bedding by up to 50 deg within a bedrock sliver bounded by two shear planes. Deeper in the section the separation has resulted from shear directly on the bounding planes, as well as rotation of the bedrock sliver. The bounding planes of the sliver merge at a depth of approximately 43 m (140 ft). Below this depth, the reverse stratigraphic separation results from direct shear displacement along a single shear plane. The magnitude of reverse displacement decreases progressively from the surface to nearly zero at a depth of approximately 60 m (200 ft). Below this depth, a normal displacement of 0.15 to 0.30 (0.5 to 1 ft) occurs (Figure 2.5-53).

As noted previously, the displacement adjacent to the fault (0 to 0.9 m, 3 ft) is less than the displacement measured away from the structure (1.5 to 1.8 m, 5 to 6 ft) near the bedrock surface (Figures 2.5-52 and 2.5-53). In contrast, at the base of the Oswego Sandstone and below, the measured amount of displacement

is significantly less away from the structure than it is adjacent to the fault. On opposite sides of the structure it is always possible to find two points on the same stratigraphic horizon that do not appear to be displaced. The horizontal distance between two such points becomes progressively smaller with depth. Hence, the reverse dislocation of strata occurred only within a narrow zone contained almost entirely in the hanging wall block. In cross section, this zone has roughly the shape of a right triangle with the Cooling Tower fault forming the hypotenuse. The vertical leg of the triangle is approximately 60 m (200 ft) long; the horizontal leg is at least 18 m (60 ft) long.

Figure 2.5-54 is a schematic diagram illustrating the effect of these unusual changes in the displacement. It shows the positions of three marker horizons on each side of the structure to illustrate the actual displacements. Prior to the development of the structure, the stratigraphic thickness must have been equal on each side. However, the present stratigraphic thickness of the section between Markers 1 and 3 on the hanging wall is about 1.8 m (6 ft) greater than on the footwall. Notably, the magnitude of this difference in thickness (dilation) is nearly equal to the overall reverse stratigraphic displacement. The amounts of both the displacement and the dilation decrease progressively downward and do not appear to be present below a depth of approximately 60 m (200 ft). Hence, the reverse displacement at any point along the structure is approximately equal to the amount of dilation of the section on the hanging wall. It should be noted that this is true not only for the zone of flexural shear displacement but also for the direct shear displacement observed below this zone (Figure 2.5-55).

Analysis of the information from the investigation of the Drainage Ditch and Cooling Tower faults provides the basis for understanding the relative age of the reverse and bedding-plane-slip displacements. It must be noted that the principal mechanism for the deformation was buckling[94]. The dilation of the bedrock and slip of the strata toward the crest of the buckle resulted in the observed reverse displacements.

Buckling along the north dipping high-angle faults postdated the strike-slip and normal fault deformations, as indicated by several structural relationships. Structural fabrics resulting from both strike-slip and normal fault episodes were deformed by bedding-plane-slip related to the buckling (Figures 2.5-45 through 2.5-47). Conjugate strike-slip and normal fracture sets have been rotated in conjunction with the limbs of the buckle[94]. The buckling deformation and absence of associated mineralization indicates the deformation occurred near the surface, whereas the homogenization temperatures of mineralization associated with the episodes of strike-slip and normal faulting indicate that deformation occurred at considerable depths of burial[94]. For additional information and further clarification regarding the absence of mineralization during buckling and the possibility

that it was caused by creep movements in a compressional regime, refer to the response to NRC Question F231.7 in Appendix 2N.

The reverse-slip deformation occurred during more than one phase of movement. Field data from precise surveys show that some of the bedrock deformation occurred prior to deposition of the Late Wisconsinan and Holocene sediments above the Cooling Tower fault. Specifically, it has been demonstrated[94] that in the Cooling Tower Piping Trench and Pit 1 exposures (Figure 2.5-28), bedding on the south side of the fault dips 7 to 9 deg southward marking the tilted limb of a monocline. Varved lacustrine sediments spanning this limb were not structurally affected by this rotation. However, there is obvious evidence of deformation of the sediment as described above. Hence, a second phase of deformation must have occurred later than that of the monocline development, and later than 12,000 yr B.P., the approximate age of the layered sediment.

It is uncertain how many deformational events affected the overburden sediments. The effects of arching and monoclinal flexuring, with associated small-scale reverse and normal faults, all may be the result of one deformation. The elements of deformation are emphasized by local well-defined shear planes where the sediments have been faulted because of arching or flexuring. However, there are present in several exposures a number of diapiric structures caused by fluidization. In Pit 1 and Trench 3, these features are clearly deformed by the small faults formed during arching of the sediments (Figures 2.5-32 and 2.5-34). Thus, they predate the formation of small-scale normal faults. It has been observed that, although some diapiric structures do occur in sediments away from the fault, they are mostly concentrated in the area of the fault zone. It is possible that the formation of these structures was related to fluid pressure changes as a result of water level fluctuations in the Ontario Basin, as discussed later.

Mechanism of Reverse-Slip Deformation (Pre-Wisconsinan) The expansion of the stratigraphic section on the north limb of the structure is an important aspect of the deformation that was detected in the subsurface. The cross-sectional area affected can be approximated as a right triangle equal to at least 557 sq m (6,000 sq ft). Unidirectional dilation resulted in an increase of area in this zone that is equal to a minimum of 15 sq m (165 sq ft)[94]. The amount of expansion on the hanging wall then is approximately 2 to 3 percent. It is also likely that expansion affects the strata of the footwall or south limb, particularly for the strata above the apex of the two shear planes within the zone of flexural shear displacement (Figure 2.5-55). However, uncertainties in the amount of stratigraphic separation due to the direct shear displacement and in the geometric arrangement of strata on the south limb do not permit an estimate of the amount of expansion that might exist there.

Dilation of the bedrock is associated with the presence of voids or partings along bedding planes, as corroborated by observations in both the surface and subsurface investigations. These have been observed in the excavated bedrock slots[94]. Similarly dilated strata have also been noticed in the subsurface. They commonly occur as voids into which the drilling rods suddenly dropped and water circulation was lost. The presence of these voids was further substantiated by a downhole impression packer survey[94].

Although the cumulative effect of the bedding plane separations appears to account for most of the expansion of 1.5 or 1.8 m (5 or 6 ft), it is believed that time-dependent deformation of the rock also contributed to the total displacement. Disharmonic, concentric drag folds that occur outside the bedrock sliver and adjacent to the shear fractures bounding the sliver attest to this[94]. Structural shortening of these folds is approximately 20 to 30 percent, based on that observed in Trench 3 (Figure 2.5-51).

As discussed in Section 2.5.4.1.4, a rock specimen can undergo an increase in its original dimensions during and after removal from the in situ stress field. This increase is elastic or instantaneous as well as time dependent. It has been shown by unconfined swelling tests that the rock specimen expands when removed from the bedrock and placed in an environment of constant temperature and high humidity. The rate of this expansion varies with the lithology of the specimen and is greatest in the direction perpendicular to bedding. The swelling process is caused by the development of very high negative pore pressures in response to the elastic strain relief. This results in the flow of water to the available dilated pore species of the rock. The swelling, and concomitant flow of water, can be time dependent as a function of gradual changes in permeability of the rock mass related to time-dependent change in the stored strain energy. Considering these results, it appears reasonable to assume that the perturbation of the stress field at the Cooling Tower fault did cause swelling in the bedrock. Therefore, the total expansion very likely may be the combined result of both factors: the separation along bedding planes and the internal volumetric changes. This internal dilation is relatively small or nonexistent on the footwall (excluding the rotated sliver), but may be considerable on the hanging wall. Hence, these postulated volumetric changes may be differential and could have enhanced and prolonged the deformation process by providing an additional source of incremental, distortional, strain energy.

Considering the observations and relationships provided above, buckling is the mechanism of reverse-slip deformation on the Cooling Tower fault. Buckling is also related to the bedding-plane-slip distortion of the normal fault observed in the excavations. It is important to note, therefore, that the direct shear displacement (Figure 2.5-55) is only a secondary effect of the buckling instability.

The entire structure can actually be accurately described as a sequence of full-wave length (λ) buckles in the upper part and a sequence of half-wave length ($\lambda/2$) buckles in the lower part. These buckles are superimposed upon a preexisting normal fault that dips 60 deg northward. The amplitudes and wavelengths of each type of buckle diminish progressively with depth. Concomitantly, 1) the reverse-slip displacements decrease to zero with depth, 2) the displacement decreases to zero in a lateral direction away from the structure, and 3) the zone of dilation on the hanging wall narrows with depth (Figure 2.5-55).

<u>Mechanical Theory (Stages of Development)</u> To evaluate the possibility of reactivation, the mechanics of buckling along the Cooling Tower fault have been treated by applying the theory of folding of stratified viscoelastic media, as developed by Biot[156-158], by reconstructing the buckling process across the fracture plane. This treatment is summarized herein. A more in-depth treatment is contained in the 1978 report of the Cooling Tower Fault Investigation[94].

The geologic data indicate that the pre-Wisconsinan buckling proceeded through four states: the deflection, amplification, rotation, and stabilization stages (Figure 2.5-56). During the initial stage, a deflection of small amplitude occurred in one or a sequence of layers on the hanging wall of the north-dipping fault (Figure 2.5-56, A and B). The geometry of this feature indicates that the initial deflection did not occur at the bedrock surface, but instead at the base of the massive sandstone or else within the thinner beds of the Transition Zone. The initial wavelength was approximately 18 m (60 ft) (Figures 2.5-41 and 2.5-42)[94]. The possibility that the initial deflection is drag effects of the earlier normal fault displacement on the high-angle fault is addressed in the response to NRC Question F231.10 in Appendix 2N.

During the second (amplification) stage of buckling, the amplitude of the initial deflection increased as a function of time and the acting compressive stress[156]. The result of this stage was the formation of a monocline in the massive sandstone, such as that seen in the initial Cooling Tower Trench exposure of the fault. Consequently, the more thinly bedded strata beneath the sandstone began to deflect upward along the discontinuity because of the reduced vertical load. This process (development of 2 buckles) affected successively deeper strata, thereby propagating the buckling instability downward (Figure 2.5-56, C).

The third (rotation) stage was characterized by the development of a second (lower) shear plane (Figure 2.5-56, D) across the strata. This second shear occurred when progressive buckling, accompanied with reverse-slip along the preexisting discontinuity, induced an incremental bending moment caused by the shear force acting in the plane of the fault. However, the

second shear plane formed only below the depth of the initial deflection (Figure 2.5-55), and converged with the old, normal fault plane (at a depth of 43 m, 140 ft) where buckle amplification was insufficient in magnitude to generate a critical bending moment. Despite a certain geometric similarity, it should be noted that the Cooling Tower and Drainage Ditch faults are not kink structures[94]. For additional information and further clarification regarding development of a new fracture, refer to the response to NRC Question F231.10 in Appendix 2N.

The stabilization stage (Figure 2.5-56, E) is characterized by minor deformation that followed the rotation of the bedrock sliver of the footwall. Evidence from the subsurface indicates that the rotation of the bedrock sliver on the footwall did not terminate the development of the structure. Further strain is indicated by the formation of concentric drag folds in the strata adjacent to the lower and upper shear planes based on interpretation of bedding dip versus depth in boreholes drilled through the fault[94]. It is possible that the rotation of the southern limb occurred prior to the attainment of the final depth by the buckling process. The evident continuation of the development of the structure may reflect the deformation that was extending the buckling instability to that depth. This was accompanied with further amplification of the individual $\lambda/2$ deflections below the zone of rotation. As the south limb had become locked after it was rotated to the critical angle with the shear plane[94], the strain resulting from the continuation of buckling had to be accommodated by a mechanism of deformation other than rotation, such as the bedrock swelling associated with the perturbations of the stress field in proximity to the fault zone. Consequently, the bedrock continued to experience differential (different magnitudes in different parts of the rock mass) and time-dependent internal expansion. This process introduced additional incremental quantities of distortional strain energy into the buckling system. This thereby increased the time required for its stabilization by partly compensating for energy loss due to the rotation of the bedrock sliver between the shear planes.

Mechanical Analysis A detailed treatment of the analysis of buckling mechanics on the Cooling Tower fault may be found in a previous report to the NRC[94]. The analysis was performed to determine whether or not: 1) the buckling can propagate to greater depth, and 2) the feature is seismogenic. The major points of the analysis are summarized herein.

Buckling in the site area was noted to occur on only two of the three high-angle faults (Figure 2.5-57). These two faults dip northward (Cooling Tower and Drainage Ditch faults), whereas the third fault dips southward (Barge Slip fault). Ironically, however, the south-dipping fault could be inferred to have the greatest potential for developing this type of deformation,

assuming that the stresses onsite prior to buckling were horizontal and uniform.

A possible explanation for the absence of reverse-slip deformation and associated buckling on the Barge Slip fault is likely related to the direction of its dip. However, for this factor to be significant, the trajectory of the greatest principal stress must be inclined with respect to the earth's surface. The occurrence of the $\lambda/2$ sequence of buckles on the hanging wall of the northward dipping faults and the sense of bedding-plane-slip indicate that if the stress trajectory were inclined during development of these buckles, it would have to be inclined toward the south. In this situation neither the hanging wall nor the footwall of the Barge Slip fault was susceptible to the reverse-slip deformation because the sense of shear stress acting in a plane parallel to the fault was incompatible with the sense of shear stress in a plane parallel to bedding (Figure 2.5-57).

Thus, the southward inclination of the stress trajectory is a factor that controlled the selective development of the deformation. Changes of this inclination also probably influenced the stability of the northward dipping faults. Such changes have been shown to be related to glacially-induced downwarping associated with the growth of the continental ice sheet known to have affected the site area (Section 2.5.1.1.4)[94]. Hence, the tendency of the plunge of the σ_1 trajectory to increase southward ceased a few thousand years ago.

For additional information and further clarification regarding other possible causes of the absence of buckling on the Barge Slip fault, refer to the response to NRC Question F231.11 in Appendix 2N.

During glacially-induced crustal downwarping, as the plunge of the σ_1 trajectory increased progressively southward, changes in the magnitudes of the stresses occurred and affected the stability of the bedrock. These changes are related to the boundary conditions of the bedrock onsite as depicted schematically on Figure 2.5-58. The site strata crop out to the north on the floor of Lake Ontario or on its north shore. This boundary appears to be a free (or deformable) boundary. By contrast, the southern boundary is assumed to be a nondeformable boundary because the average structural gradient of 9.5 m/km (50 ft/mi) to the south provides a confined boundary condition. Furthermore, the ability of bedding to accommodate shear stress without developing shear failure increases with depth (Figure 2.5-58). Therefore, the amount of shear stress on bedding is too small to cause instability. Figure 2.5-58, B illustrates the inferred relationship of the layer-parallel shear stress with depth as related to the regional dip of the strata. The difference between the layer-parallel shear stress and the strength of a bedding plane decreases southward as the attains a

greater depth. Thus, the excess shear stress developing in the shallower part of the layer is not being relieved by bedding-plane-slip. This results in additional shortening of the layer above the point for which the difference between the shear stress and shear strength is equal to zero. In this situation the normal stress parallel to layering attains progressively greater values in time. When this stress reaches a certain critical value, it will facilitate stress relief by a mechanism other than bedding-plane-slip, such as buckling.

From the foregoing discussion, it is evident that slip of a given layer southward toward the fault relative to another layer beneath it is required to accomplish the buckling along the discontinuity. The question of the stresses necessary to initiate buckling has been considered utilizing a simple model[94]. In general, it may be shown that the layer-parallel normal stress must be large enough to overcome the frictional resistance to sliding of a layer along the inclined fault, the body weight of the layer, and the critical buckling stress of the layer. Moreover, layer-parallel shear stress must exceed the bedding plane shear strength to initiate frictional sliding between layers.

The buckle amplitudes and wavelengths decrease progressively to a depth of 60 m (200 ft) where they die out (Figure 2.5-59). For the stress levels extant prior to glaciation, as inferred from stress measurements at the Unit 2 site (Section 2.5.4.1) which are believed to be less than 210 kg/sq cm (3,000 psi), the analysis indicates that, at the site, typical strata with an average thickness of 0.3 m (1 ft) would buckle with length-to-thickness ratios (L/t) less than 200:1 (Figure 2.5-60). At the level where the smallest L/t occurs, the magnitude of the layer-parallel normal stress must have been much greater than 210 kg/sq cm (3,000 psi) at the time of buckling.

The foregoing information is relevant to the assessment of the possibility that the amplification process may propagate to a greater depth than where presently detected (60 m, 200 ft). This depends upon the magnitudes of two controlling factors referred to as the potential depth of amplification (D_a), and the potential depth of reverse-slip deflection (D_r). The relationships between the subhorizontal compressive stress, the layer-parallel shear stress, and the slenderness ratio (L/t) can be used to evaluate the present D_r for the Cooling Tower fault, a quantity is necessary for one to determine if the structure is capable of propagating to greater depth. The characteristic relationship displayed by the wavelength of individual buckles to the depth of their occurrence is an important factor in this determination.

The buckle wavelengths progressively diminish with depth (Figure 2.5-59), and this corresponds to a similar decrease in the slenderness ratio because the thicknesses of the strata are relatively constant throughout the section in which the structure

is developed. The layer-parallel stress required to initiate adeflection is least if the body weight of only a single stratum is considered, ignoring that of the overlying strata. However, this is true only if a layer beneath the given stratum is deflected with a smaller wavelength (slenderness ratio) than that of the overlying stratum. Thus, it follows that D_r is governed by the wavelength of the uppermost deflections, and is directly dependent upon the magnitude of the layer-parallel stress.

Considering this relationship, it can be shown that, for the Cooling Tower fault, D_r today is smaller than the depth to which the structure developed in the past.

Prior to the glacial advance, the σ_1 trajectory had probably been parallel to bedding. Hence, the layer-parallel shear stress was zero. Under these conditions, as soon as downwarping of the land surface was initiated, the layer-parallel shear stress attained a value greater than zero. Consequently, the strata of the Transition Zone were susceptible to folding. As the downwarping continued, the layer-parallel shear stress increased progressively so that deflections could form at a greater depth. Therefore, the maximum potential depth of development of the reverse-slip deflections occurs when the amount of distortional strain energy in the bedrock is greatest, which occurs concomitantly at the time of maximum crustal downwarping. During glacioisostatic rebound, the amount of distortional strain energy gradually diminishes and D_r, therefore, decays with time accordingly. Hence, it can be justifiably concluded that D_r is presently much smaller than it was during the development of the structure at the time of crustal downwarping.

Biot[159] has shown that the amplification of folding in layered, viscoelastic media will be restricted to a finite depth termed herein the potential depth of amplification (D_a). D_a is dependent upon the lateral compressive stress and is approximated by:

$$\frac{\sigma}{9\rho.g} \leq D_a \qquad (2.5\text{-}1)$$

Where:

σ = Compressive stress

$\rho.g$ = Unit body weight

The in situ stress may exceed 140 kg/sq cm (2,000 psi) at the Unit 2 site (Section 2.5.4.1.4). Hence, D_a may presently exceed 60 m (200 ft), and amplification can theoretically occur below this depth. However, D_a actually depends on the magnitude of D_r. If D_r is less than D_a, the D_a is effectively equal to D_r, which is currently less than 60 m (200 ft).

The considerations of the rates of deformation, and resultant accumulation of distortional strain energy, which are believed to have been extant during the progressive development of the Cooling Tower fault[94], suggest that the folding process may be seismogenic only during the rotation stage which is a stage of unstable growth (formation of the rotated bedrock sliver). Hence, the present ability of the structure to generate vibratory ground motion of noticeable intensity appears to be dependent upon whether or not the structure can reenter the rotation stage. This renewed folding cannot be accommodated on those parts of the fault that have already experienced this phase, because the structure has become locked and the rotation is completed. Thus, the folding would have to occur in the area where only the $\lambda/2$ stack of buckles is developed, that is, below a depth of 43 m (140 ft).

To generate a seismic shock, the renewed folding would require a volume of rock mass sufficient to create another rotated limb or bedrock sliver in this zone. This would require that the depth at which the original amplification was terminated be extended downward to a minimum depth of 85 m (280 ft), or approximately twice the thickness of the stratigraphic section involved in the original folding during unstable growth. However, an amplification depth of 85 m (280 ft) would be realized only if both D_a and D_r equal or exceed 85 m (280 ft), which is not the case. Hence, the recurrence of unstable growth folding on the Cooling Tower and Drainage Ditch faults with a concomitant seismic event is a very unlikely possibility, and there is no rational basis to consider the structure to be a fault presently capable of generating vibratory ground motion.

<u>Mechanism of Postglacial Deformation</u> In light of the foregoing discussion, it is very possible that the disturbance of the overburden sediments reflects movements along the fault caused by changes of fluid pressure in the bedrock. These changes may have been caused by the fluctuation of the water level in the Ontario Basin following glacial retreat. It seems very unlikely that the deformation of the overburden sediments indicates that the Cooling Tower fault buckles were propagating downward within the past 10,000 yr. The reasons for this are twofold. First, the postglacial movements occurred when the differential crustal downwarping was reduced due to glacioisostatic uplift or rebound[94]. Hence, the amount of distortional strain energy stored in the bedrock was reduced from its former value. Consequently, the potential depth of development of the structure at this time was smaller than that to which the structure had already developed. Second, the dilated openings in the bedrock would prevent propagation of any movement to the surface from a depth of more than 43 m (140 ft). Hence, even if these movements had occurred at depth, one would not expect to find them expressed at the surface.

The maximum Lake Iroquois water level was approximately 91 m (300 ft) above the land surface at the Nine Mile Point site. As the ice sheet receded to the north and opened the lowest lake outlet in the Thousand Islands region, Lake Iroquois was drained through the St. Lawrence Valley. The water level subsequently assumed a low stand commonly referred to as the Admiralty Stage. The water level of Lake Iroquois dropped approximately 137 m (450 ft) to reach this latter stage. The bedrock in the site area is covered by a thin, but relatively impervious veneer of unconsolidated sediments which would prevent a rapid flow of fluids to the surface or into the lake from the bedrock. Hence, it may be inferred that, at a time after deposition of the deformed lacustrine sediments, the fluid pressure in the bedrock could be much greater than the water pressure exerted by the lake. It may be further inferred that after the deposition of the unlithified sediments (including the upper sequence of Sandy Creek), this excess fluid pressure, ΔP, underwent incremental changes as suggested on Figure 2.5-61. Consequently, the effective stress normal to bedding (σ_3-P) was modified accordingly.

Such changes in the effective normal stress influenced the stability of the Cooling Tower feature by temporarily reducing the effective shear strength of bedding[94]. The problem has been analyzed using a model showing a bedrock stratum with thickness (h) that is situated above a shear zone and that can move along this zone as a solid block. With the assumption that there was no change of in situ stress during the time between the completion of the preglacial movements on the Cooling Tower fault and the high stand of Lake Iroquois, the model presents the equilibrium conditions for sliding on bedding planes in the vicinity of the fault prior to the buildup of fluid pressure. The stratum modeled corresponds to a layer or sequence of layers forming one of the two limbs of the buckle feature. When the buckle feature was formed, stress-drop normal to the fault occurred. The modeled layer attained equilibrium for a particular value of the effective stress normal to bedding, and a point near the axial plane region of the fold was thereby displaced toward the region of greatest stress relief. If there was no change in ΔP until the end of the high stand of Lake Iroquois, then the equilibrium would endure until that time, and the displacement would remain constant. At the end of the high stand of Lake Iroquois, ΔP was progressively increasing, thereby causing an incremental reduction of the effective normal stress. This, in turn, caused an incremental reduction of the effective shear strength of bedding. Consequently, the equilibrium of the limb could not be maintained and additional incremental translations must have occurred toward the region of low stress, that is, toward the axial plane. The translation was greatest near the axial plane, and gradually decreased away from it.

One notable exception to this occurred in Trench 3 exposure where the north limb is unopposed by the south limb which was removed by glacial erosion (Figures 2.5-47 and 2.5-51). The north limb

migrated southward, displacing the older fault plane about 1.2 m (4 ft) leading to the folding of the unlithified sediments with ancillary faults[94]. Conversely, if the margin of one limb was equally opposed by another, the lateral translations of this limb would be restricted, and most effectively so, if the axial plane (fault plane) separating the limbs were vertical. As indicated on Figure 2.5-44, this situation existed during deformation of the overburden sediments at the Cooling Tower Piping Trench and Pit 1 (and other locations along the trace of the fault). Thus, at these locations, the lateral displacements at the margins of each limb could not be freely accomplished, and thereby resulted in an incremental buildup of layer-parallel normal stress in the strata contiguous with the fault. When this stress attained a certain value in conjunction with the high buildup of fluid pressure, both limbs of the structure buckled further, and formed a gentle arch with the fault along the crest.

Arching and compensatory normal faults developed in the overlying sediments in response to arching of the bedrock.

The disturbance of the unlithified sediments very likely occurred as the result of a buildup of fluid pressure in the bedrock which equaled or exceeded the pressure due to the body weight of the rocks. The deformation does not, therefore, indicate that the Cooling Tower buckle was propagating downward during recent glacioisostatic uplift. The present maximum possible value of the fluid pressure in the bedrock is equal to approximately 40 percent of the pressure due to the body weight of the rock. Hence, it can be concluded that movements with a similar origin to those which caused the postglacial deformation are not likely to occur at the present time.

<u>Conclusions</u> On the basis of the analysis of the deformation process and its origin, the following principal conclusions are drawn with respect to the Cooling Tower fault:

1. The development of the pre-Wisconsinan, reverse-slip deformation is attributed to the combined effects of three factors:

    a. In situ bedrock stress field.

    b. Changes in the stress field induced by the crustal downwarping caused by glacial loading.

    c. Pronounced anisotropy of the bedrock at the site.

    In the present tectonic environment of the site area, it is not possible for the structure to propagate downward below its original depth of development. On this basis, the structure is considered to be presently incapable of generating vibratory ground motion.

2. The disturbance of the glaciolacustrine sediments (Lake Iroquois and Sandy Creek Stage of Lake Iroquois) is attributed to the excessive buildup of fluid pressure in the bedrock caused by postglacial fluctuations of the water level in the Ontario Basin. The present value of the fluid pressure has no significant influence on the stability of bedding adjoining the Cooling Tower fault. Hence, movements of similar origin are unlikely to occur during the next 40 yr.

3. Based on the present understanding of the site conditions and the mechanism of deformation, it cannot be ruled out that minor, subsurface adjustments may occur within the zone of buckling along the Cooling Tower fault. If these minor adjustments occur, they will involve a relatively low strain rate and only limited volumes of the rock mass, and thus should be considered to be inconsequential in terms of vibratory ground motion. These adjustments are expected to be restricted to the rock mass within the depth interval defined by the location of the Transition Zone (depth of approximately 15 m, 50 ft) and a depth of 60 m (200 ft). Furthermore, it is believed that the adjustments will not be expressed at the bedrock surface because of the presence of voids between layers that first must be closed. For additional information and further clarification, refer to the response to NRC Question F231.12 in Appendix 2N.

Thrust Faults

A series of shallowly dipping faults, referred to as thrust structures, are exposed in the bedrock excavations at the site. They appear to be interrelated because of striking similarities in structural style, mechanism of deformation, and relative age. A series of detailed structural, mineralogic, isotopic, and palynologic studies were conducted to investigate the nature of these thrust structures. The specific data and analyses concerning these structures are presented in a previous report[154].

The faults are predominantly developed in the area of the erosional valley in the bedrock surface (Figure 2.5-28). All field evidence suggests that they are confined between the Cooling Tower fault and the Drainage Ditch fault (Section 2.5.1.2.4). The Radwaste fault was traced to the east with borings (Figure 2.5-62). The results indicate that the thrust structure extends in its dip direction a minimum distance of 30 m (100 ft) to the east of its exposure in the North Radwaste Trench[154].

The position of the thrust structures appears related to the presence of three prominent lithologic interfaces within the stratigraphic section penetrated by the site excavations (Figure

2.5-36). The Radwaste thrust structure was exposed in several shallow excavations at the site from the North Radwaste Trench to the Circulating Water Piping Trench (Figure 2.5-28 and Appendix 2H). This structure is situated between the lower Oswego Sandstone and the upper portion of Unit A of the Pulaski Formation (Figures 2.5-63 and 2.5-64). The Unit B slip zone was exposed in the walls of the reactor containment excavation, and in rock cores from borings drilled in the vicinity of the reactor complex[154]. It is located near the interface between Units B and C of the Pulaski Formation. The thrust faults in the tunnels are exposed at the base of the intake shaft and in both lake water tunnels. Stratigraphically, these structures are located within the upper portion of Unit C of the Pulaski Formation (Figure 2.5-36 and Appendix 2H).

The thrust faults are all similar in overall appearance and display common structural characteristics. The structures consist of small zones of brecciation and folding that are at most several feet wide. The dips of these zones range from nearly horizontal to approximately 30 deg. The configuration of the thrust structures resembles a stack of tabular bedrock elements displaced to the west along bedding planes. These structures appear to be confined to specific stratigraphic units. They generally occur as planes of slip, parallel to bedding, with short intervals where the discontinuity transects the layering at a low angle and then merges with bedding at a higher stratigraphic level (Figures 2.5-64 and 2.5-65). The most intense deformation occurs along the inclined portions of the discontinuities. Locally, there may be no shear dislocation of the beds along the fault (Figures 2.5-63 and 2.5-64). The mode of displacement typically consists of:

1. Discontinuous shear dislocation of individual beds or groups of beds.

2. Rigid body rotation of beds to form small folds.

3. Broad arching of the hanging wall strata.

4. Dilation of the bedrock along bedding planes and variously oriented fractures (Figures 2.5-63 through 2.5-65).

Numerous indicators of dilation within the bedrock mass, such as voids, open fractures of various attitudes, and zones of loose bedrock rubble, were encountered during the investigation of the thrust structures[154]. This is possibly the most important characteristic of the structures and is significant because it clearly indicates that an environment of low confining pressure was necessary for their development.

The sense of dislocation on these structures is generally similar to the upper strata typically displaced to the west. Locally, slip down the dip has been noted for some of the ancillary shears

in lake water tunnel no. 1 (Appendix 2H). Curiously, reverse-slip was observed on different parts of the same ancillary shear with normal slip in some places. The principal stratigraphic dislocation in the tunnels, however, is reverse-slip toward the west. The slip direction of all the thrust structures appears to vary from west-southwest to west-northwest progressively from the cooling tower area to the tunnel exposure (Figure 2.5-28). This variation apparently depends either on the location of a particular exposure and/or depth of development, that is, upon the stratigraphic position of the faults. The variation of slip direction attests to the heterogeneous nature of the strain along these structures. It is possible that this has resulted from a progressive change in the stress trajectories either laterally or with depth.

The magnitude of displacement also varies along both the strike and dip of the Radwaste fault. For example, 1.5 to 2.1 m (5 to 7 ft) of displacement is present near the upper limit of the ramp of the structure in the North Radwaste Trench (Figures 2.5-63 and 2.5-64); however, in the deepest part of the excavation, an apparent dip separation of 1.4 m (4.5 ft) was noted. The faults in the lake water tunnels (Appendix 2H) exhibit small displacements typically less than several inches. The magnitude of displacement along the Unit B slip zone has not been established. However, one may infer that the displacement is at least equivalent to that of the tunnel faults because it occurs at a higher level in the stratigraphic section.

Secondary materials commonly occur within the thrust structures. These materials consist of several types of calcite mineralization and glaciolacustrine sediment. The calcite occurs along dilated bedding planes and fractures, as well as along shear surfaces[154]. On shear planes, the calcite commonly cements breccia fragments and displays dip-slip slickensides. Calcite within dilated fractures occurs as concretionary nodules or patches of euhedral, drusy crystals. In one case, laminated clays deposited within a dilated bedding plane are cemented by the calcite.

Two types of unlithified sediment fill openings in the bedrock near the thrust structures: a gray to tan laminated silty clay, and a gray massive plastic clay. The latter type was most commonly present in zones of intensely shattered bedrock along the ramp portions of the faults. The laminated clay typically fills dilated bedding planes. Similar occurrences were noted to depths as great as 82 m (270 ft) in rock cores recovered from borings[154]. Within the North Radwaste Trench, laminated clay was found mixed with the breccia along bedding planes where it appeared to be contorted. Another example of deformation of the laminated clays within the Radwaste fault occurs in the exposure of a small monocline in the North Radwaste Trench (Figure 2.5-66). At this location, clay layers 0.6 to 2.5 cm (0.25 to 1 in) thick dip from 20 to 70 deg, parallel to the steep limb of the monocline.

The age of development of the Radwaste fault (and other thrusts) was evaluated by considering the age and origin of the secondary filling materials. Specimens of calcite minerals were analyzed using geothermometry, isotope ratio analysis, and radiometric dating. The details of the samples collected and specific results obtained are presented in a previous report to the NRC[154]. These results were compared with similar data obtained from analyses of mineralization occurring in proximity to the high-angle faults. The paragenetic study of the calcite found in association with the thrust structures strongly suggests that the four principal varieties identified may be different facies of the same depositional stage. Fluid inclusion analysis of this calcite indicated that it crystallized very near the ground surface at temperatures only slightly greater than present ambient temperatures, that is, 10° to 30°C (50° to 86°F). This calcite contrasts with the epigenetic calcite along the high-angle faults whose formation temperatures ranged from 70° to 160°C (158° to 320°F), indicating an origin at greater depth, and thus geologically older. Isotopic analysis confirmed the fresh water origin of the low temperature calcite and revealed that some of it (the calcitic breccias) probably formed below the groundwater table, whereas some (the travertine) apparently formed in the vadose zone. Radiometric dating of the former, using the Thorium-230/Uranium-234 technique[154], indicated only that the material is younger than 300,000 yr old. However, radiocarbon dating of the travertine yielded a C-14 age of 14,180 ±550 radiocarbon yr B.P. These radiometric dates confirm the interpretation that the low-temperature calcite is younger than the high-temperature calcite and is likely Quaternary in age.

The interstitial sediment within openings in the bedrock is similar in appearance to surficial glaciolacustrine sediments of Pleistocene age (Section 2.5.1.2.2). Therefore, it was suspected to be of similar origin. Specimens of the interstitial sediment were collected for grain size analysis, compositional and heavy mineral analysis, and pollen analysis. These results were compared with similar analyses of specimens from the surficial sediments. The data, results, and conclusions drawn from the analyses are presented in a previous report to the NRC[154]. The conclusion of principal importance is that the tan, laminated sediment is derived from material of glaciolacustrine origin. This material contained pollen and spores (Figure 2.5-67) similar to those found in portions of the Late-Pleistocene age surficial sediments (Figure 2.5-33). A Late-Pleistocene age of the interstitial sediment was confirmed by a C-14 date of 11,060 ±360 radiocarbon yr B.P. for organic material in the sediment collected from within the Radwaste fault[154]. The age of similar laminated clay encountered at a depth of 76 m (250 ft) in the borings is uncertain. However, palynologic analysis of this material (Figure 2.5-67) also suggests a late-Quaternary age.

A minimum age of approximately 11,000 yr B.P. can be interpreted for the development of the Radwaste fault on the basis of the

radiocarbon date of laminated clays encountered within the zone of deformation. Dilation of the bedrock associated with development of the structure clearly had to have occurred prior to emplacement of the clays. Nevertheless, the exact age of latest deformation is uncertain because the time of deformation of the laminated clay is unclear. Based on his observations in the North Radwaste Trench, Dr. L. Sirkin[154] concluded that the lacustrine sediments were deposited in the bedrock openings and were subsequently deformed. In contrast, the prevailing opinion is that of Drs. T. L. Pewe, R. H. Jahns, and S. S. Philbrick[154], namely that the deformation in the bedrock had occurred prior to the deposition of the clays on the basis of observations of the clay overlying the hinge of a fold.

<u>Mechanics of Thrust Structure Development</u> Many factors indicate that the thrust structures developed near the ground surface under similar conditions extant at the time of development of buckling along the north-dipping faults. Also, the thrust structures are only recognized within the bedrock block bounded by the high-angle faults (Figure 2.5-28). The possible age and spatial relationships between these two groups of structures must be inferred because the absolute age of formation of the thrusts is not known, and there are no data which allow an interpretation of their relative ages. Analysis of the possible relationships between these deformations led to the inference of a mechanism of development for the thrust structures (response to NRC Geologic Information Request Q361.28).

The thrust structures appear to have resulted from the relief of stored strain energy which is assumed to be remanent gravitational and/or tectonic in origin. Development of the structures was triggered by an environmental change that reduced the ability of the bedrock block to retain the stored strain energy. This ability is controlled by the resistances to shearing along the boundaries and the lateral restraining forces. Hence, reduction of the restraints precipitated release of stored strain energy resulting in the development of the thrust structures.

The postulated boundary conditions for the bedrock block are shown on Figure 2.5-68. Two factors, consisting of lateral restraining force and shear resistance along the block boundaries, controlled the initial strain energy stored in the bedrock. The lateral restraining force was provided by the bedrock which has since been removed during formation of the erosional valley shown on Figure 2.5-28. The shear resistance along the block boundaries (Figure 2.5-68) consisted of shear resistance along the Drainage Ditch fault (Boundary 1), shear resistance along the Cooling Tower fault (Boundary 2), and shear resistance along the base of the block (Boundary 3).

In the instance of Boundary 3, the shear resistance is the sum of the integrated shear strength on the boundary and the shear stress acting along bedding planes.

Significant changes in the values of normal stress (σ_{n1} and σ_{n2}, Figure 2.5-68) and shear stress parallel to bedding must have occurred in response to gravitational loading of the lithosphere by glaciation(s) and other related phenomena. Changes in the value of shear stress parallel to bedding resulted from shear straining of the lithosphere caused by glacially-induced differential vertical movements. Reduction in the value of normal stress perpendicular to bedding may have accompanied changes in the water level of ice-marginal lakes.

Considering the foregoing, the following three-part scenario for development of the Radwaste fault is inferred. Initially, downwarping of the crust in relation to glacial advance created conditions favorable for buckling of the hanging wall blocks of northward dipping high-angle faults (as discussed previously). This reduced normal stresses acting across the high-angle faults. Hence, the shear resistances along the Drainage Ditch and Cooling Tower faults were reduced. Next, formation of the erosional valley reduced the lateral restraining force. Finally, removal of the load imposed by the continental ice sheet reduced normal stress perpendicular to bedding, and thus reduced shear resistance along the base of the block (Boundary 3, Figure 2.5-68). Further reduction of these restraints may have occurred during the drainage of Lake Iroquois as a result of the development of high fluid pressure in the bedrock.

The mechanics of deformation of the thrust structures in the intake shaft and lake water tunnels are inferred to be similar to the Radwaste fault. Studies[154] have suggested that these structures consist of a principal fault with numerous nearby ancillary splays or subparallel shears. The attitude, curviplanar nature, and lateral extent of all the shears are similar. Slickensides indicate instances of oblique slip (Section 2.5.4.3, Appendix 2H). All exhibit some occurrences of calcite mineralization of a type shown to have precipitated from groundwater at ambient surface temperature[154].

As noted above, some instances of slip down the dip occur on these shears. Most of these occur structurally beneath the principal thrust fault exposed in the shaft. They can be generically explained in either of two ways:

1. They could be developed as the result of, and in proximity to, a differential, vertical displacement, for example, low-angle "normal" faults that are formed in response to development of a high-angle reverse fault.

2. It is also possible that they represent a secondary response to bedding-slip thrusting deformation.

The slip down the dip of low-angle faults in the westernmost tunnel can best be explained by the latter possibility because: 1) the strain along the thrust faults is heterogeneous (Figure 2.5-28), 2) this manner of strain supports the possibility of rotational slip on the thrusts where reverse-slip at one location is accompanied by normal-slip at another location, and 3) the spatial relation of shears with normal-slip below the main structure and reverse-slip above it. Furthermore, no differential vertical displacement, as in the explanation 1 above, is present.

In summary, detailed studies of the site conditions strongly suggest that the thrust faults do not cut the Cooling Tower and Drainage Ditch faults. It appears that the thrust faults together with the high-angle faults form an integrated, but sequential, system of bedrock deformation.

The postulated equilibrium conditions of the thrust structures imply that the development of buckling (the pre-Wisconsinan phase of the reverse-slip deformation) along the Cooling Tower and Drainage Ditch faults contributed to the instability of the intervening bedrock block. Hence, it can also be inferred that the development of the thrust faults postdates the first phase of reverse-slip movement along the Cooling Tower fault.

<u>Conclusions</u> The results of detailed geologic investigation of the Radwaste fault resulted in the following conclusions regarding the nature and origin of the thrust structures. The following conclusions and results are supported by a panel of consultants[154]:

1. Movements along the Radwaste fault (and similar thrust faults) have been recurrent.

2. The initial development of the structure is believed to be associated with crustal loading and unloading during episodes of glaciation[154]. This suggests that the thrust was initiated sometime between 12,000 and 2,000,000 yr ago. Based on experience with similar structures, Drs. Jahns and Philbrick[154] believe that the age of initial formation can be narrowed to between 150,000 and 400,000 yr B.P.

3. The minimum age of development of the Radwaste fault is approximately 11,000 yr B.P. However, the exact age of the latest deformation is uncertain[154] because the relationship of the lacustrine clays to the bedrock deformation does not provide certain resolution of the age of latest deformation.

4. The thrust faulting results from the release of stored strain energy caused by the reduction in vertical confining pressure by erosion. The faulting occurs on the flanks of the small bedrock valley. It is

postulated that formation of the valley disturbed equilibrium conditions and removed the lateral restraint that had prevented the expansion of the strata on either side of the valley. Furthermore, the development of the thrust faults was facilitated by buckling across the lateral boundaries of the thrust sheet (i.e., Cooling Tower and Drainage Ditch faults). The buckling resulted in a significant reduction of the normal stress acting perpendicular to these boundaries, thus lowering the resistance to frictional sliding of the thrust sheet relative to the surrounding bedrock.

5. As stated in the consultant report[154], "the faulting is not related to current tectonic processes that could introduce additional amounts of strain energy." In addition, it is stated that "it can be concluded that no increase in the amount of stored strain energy will occur during the coming centuries."

6. Based on observations of analogous geologic structures, Drs. Philbrick and Jahns conclude that because of the inability of the structure to build up significant amounts of strain energy, the Radwaste structure is so nearly dead at present levels of exposure that its participation in such future movements would amount to no more than a small fraction of an inch.

Drs. Jahns and Philbrick also conclude that any future movements should not exceed 0.64 cm (0.25 in), based on their experience. Based on studies including mathematical modeling, it has been concluded that 2.54 cm (1 in) is a conservative allowance for future maximum credible movement. Thus, an allowance of 1 in is used for design purposes.

2.5.1.2.4 Broad Low-Amplitude Folds and Associated Normal Faults (Demster Structural Zone)

Introduction

The Demster Structural Zone trends northeast. Associated with this zone of locally intense fracturing and faulting is a sequence of gently southwest-plunging, broad, asymmetric anticlines and synclines. This zone of complex deformation is in the Late Ordovician Oswego Sandstone, the youngest site area rock unit in outcrop and subcrop. Post-Ordovician deformation was identified during subregional and site subsurface mapping investigations at the New York State Electric & Gas Corporation (NYSEG) proposed New Haven nuclear site approximately 8 km (5 mi) southeast of Nine Mile Point (Appendix 2I, Figure 1-0).

Subsurface mapping defined and delimited the major bedrock structures within a 5-mi radius of the New Haven site. Recently acquired deep well data[160] for the region east of the New Haven

site substantiate subsurface interpretations reported in the 1979 investigations[81].

Field investigations, data synthesis, and conclusions of the New Haven site study are reported in the NYSEG PSAR[81]. Appendix 2I to this Final Safety Analysis Report (FSAR) supplements the New Haven PSAR data base and also integrates results from more recent geology and geophysics literature. This section is a summary of Appendix 2I and concentrates primarily on the structural characteristics of the Demster Structural Zone.

Studies associated with the exploration of the Demster Structural Zone are shown in Appendix 2I, Figure 1-0, and included 29 diamond drill core borings; site area geologic mapping at a scale of 1:24,000 with supplemental reconnaissance mapping in and adjacent to the Adirondack Mountains; geologic mapping of a 240-ft long trench excavated across the Demster Zone; geophysical surveys including natural gamma logging of boreholes, offshore seismic refraction and reflection surveys, land refraction surveys and land magnetic surveys; review and update of regional gravity and aeromagnetic data; mineralogical, petrographic, isotope, and radiometric age analyses of representative samples obtained from the Demster fault zone; and analysis interpretation of available subsurface data in central and northern New York.

In summary, the interpretation and evaluation of the combined geologic and geophysical data support the following conclusions:

1. The Demster Structural Zone is not capable.

2. Broad folding, reverse faulting, and normal faulting associated with the Demster Zone developed sequentially through a series of three events or phases that occurred in Middle to Late Paleozoic and, possibly, Mesozoic time.

3. Ordovician strata in the site area are folded into a series of essentially parallel, southwestward-plunging anticlines and synclines. The Demster Beach Anticline is intensely deformed and faulted within part of the eastern oversteepened limb designated the Demster Structural Zone. Stratigraphic offset is due primarily to folding, but steeply northwest-dipping small faults and fold axial fractures account for the intense brittle deformation.

4. Assuming ambient depositional conditions, fluid inclusion data are indicative of calcite mineralization emplaced at temperatures greater than 100°C (212°F). Paragenetic and structural element correlation demonstrate the deposition of calcite after bacteriological reduction of sulfides, in part contemporaneous with and soon after the deformation. Early calcite mineralization indicates deposition prior

to completion of structural development and late calcite is undeformed.

Geologic Setting of the Demster Zone

Stratigraphic Setting The area adjacent to and including the Demster Zone is underlain at a relatively shallow depth by Grenville crystalline rocks of the Precambrian basement. The basement complex is apparently similar to lithologically and genetically equivalent strata cropping out in the Canadian Shield and Adirondack Dome (Section 2.5.1.1.2).

The Precambrian basement is overlain by approximately 1,500 to 1,800 ft of Cambrian-Ordovician strata which, in the site area, from oldest to youngest, are the Theresa Sandstone, Black River Group, Trenton Group, Utica Shale, Whetstone Gulf Shale, Pulaski Shale, and the Oswego Sandstone (Section 2.5.1.1.2). The Late Ordovician Queenston Formation, a sequence of red beds overlying the Oswego to the south of the site area, completes the progradational character of this sedimentary succession from limestone to shale to sandstone. This sedimentary sequence rests unconformably on a basement surface that slopes southward at 50 ft/mi[52] and results in a south-dipping homoclinal sequence of Paleozoic strata that thicken to the south and southwest. The site area, except for infrequent exposures, is overlain by several types of glacial deposits that include till, undifferentiated ice-contact stratified drift, glaciolacustrine, and peat deposits. The stratigraphic column for the New Haven site area is shown on Appendix 2I, Figure 2-1.

Subdivision of the near surface stratigraphic succession at both the Nine Mile Point and New Haven sites is nearly identical, although the nomenclature differs somewhat. The nomenclature and a comparison of the correlative units are discussed in Section 2.5.1.2.2 and are shown on Figures 2.5-36 and 2-2 of Appendix 2I. The stratigraphy of the Demster Structural Zone, both locally and regionally, is discussed in Appendix 2I.

Subsurface Structure Subsurface stratigraphic investigations in the New Haven site area were carried out in order to divide the section into mappable units to define the subsurface structure. The section represents a continuum of marine deposition in which unit boundaries are assumed to have been essentially horizontal as deposited, except on a very local scale, and therefore are considered reliable horizons. Structure contour maps of the unit boundaries, or key horizons, were constructed and examined for evidence of structural trends. The Pulaski-Oswego boundary was selected as the primary horizon because of its mappability resulting from lithologic differences between the Oswego and Pulaski. The boundaries of five mappable subzones within the Oswego Sandstone were also utilized in the structural analysis.

Structurally, the top of the Pulaski Shale is a gently sloping surface consistent with the marine conditions of its deposition,

as modified by subsequent regional tilting. Within the areal limits of stratigraphic control, from Boring R-6 on the east to Nine Mile Point on the west (Appendix 2I, Figure 2-3), the Pulaski appears to strike west-northwest and dips to the south-southwest approximately 11 m/km (60 ft/mi). Both the New Haven and Nine Mile Point sites overlie a gently sloping, mildly downwarped rock sequence whose south-southwest dip reflects the regional homoclinal structure.

Based on closely spaced Pulaski control points, the contour patterns southeast of the Nine Mile site (Appendix 2I, Figure 2-4) are indicative of abrupt changes in the strike and dip of the Pulaski-Oswego boundary. These changes, together with the pronounced linearity and the steep contour gradient, are suggestive of faulting. Inclined boreholes in the zone of suspected faulting traversed a crushed zone several tens of feet wide, including intervals of gouge and breccia, confirming the occurrence of a fault zone. Deep exploration data east of the New Haven site confirm the Mexico Anticline and suggest associated deformation on the eastern limb of this fold.

The contour pattern of the formation boundary and boring data define the position and orientation of a northeast-trending fault zone and associated folding. Figure 2-5 of Appendix 2I indicates the effects of tectonism on the Pulaski-Oswego boundary on the eastern limb of the Demster Beach Anticline, herein designated the Demster Structural Zone. Spacing and alignment of the contour pattern are indicative of folding, rather than faulting, as the dominant process in the formation of the Demster Structural Zone, as further explained in the response to NRC Question F231.18a in Appendix 2N. The fold is markedly asymmetrical with little net displacement on the fault. These structural relationships are illustrated on Figures 3-3 and 3-5 of Appendix 2I and on regional cross sections C-C' (Figure 2-6) and D-D' (Figure 2-7). Southward deflections of the contour pattern occur west-northwest and east-southeast of the New Haven site. To reestablish the regional strike and correlate with stratigraphic control at Nine Mile Point (Borings 314, L-1, L-4, and L-8), the structural contours must return to a northerly trend (Figure 2-4). Stratigraphic control west of the New Haven site indicates a repeated pattern, similar to the southwest-trending zone delineated in Figure 2-4. The contour pattern is undulatory along regional strike. To the west of Nine Mile Point, the continuity of the pattern is uncertain.

Examination of the individual structure contour maps indicates clearly the marked compression and linearity of contours for all mapped horizons in the vicinity of the Demster Zone (Figure 2-9). This anomalous contour pattern as well as the site area pattern indicate that the Upper Ordovician age strata are folded into a series of broad, low amplitude, southwest-plunging folds designated the Demster Beach Anticline and Mexico Anticline and an unnamed inferred syncline at Nine Mile Point and the New Haven Syncline.

Structural contours of all horizons examined indicated that a linear zone of deformation separates the Demster Anticline fromthe New Haven Syncline. This was shown to be a relatively narrow zone of flexure characterized by considerable stratigraphic displacement, brittle deformation, and calcite mineralization. The pattern of structure contours relative to the Demster Fault Zone is further discussed in the response to NRC Question F231.18e in Appendix 2N.

The configuration, location, trend, and extent of the three folds named above are shown on Figure 2-3; they trend N45E, plunge southwest, and extend a minimum of several miles. The amount of stratigraphic offset due to broad folding between any two control points as well as the distribution of folded units is shown on structure contour maps and sections. Initial limits of the Demster Zone were reconstructed from data solely derived from the R and P series borings. Dip angle and direction of the fault zone were defined through analysis of sedimentary and tectonic structures in core. Excavation of a 73-m (240-ft) long trench (Trench II) exposed the bedrock across the most intensely deformed zone. Detailed studies were made of the type of deformation, the amount of stratigraphic displacement due to faulting, the relationship between faulting and broad folding, and the nature and condition of surficial units overlying the bedrock faults.

<u>Detailed Geologic Studies, Trench II and Vicinity</u>

<u>Surface Geology</u> Excavations, detailed geophysical investigations, and drilling enabled an evaluation of mechanism, cause, style, extent, and apparent age of deformation. Bedrock exposure and overlying surficial deposits were mapped in detail and reported in Appendix 2.5I[81]. Trench II was the second bedrock trench excavated during the New Haven site studies. Figure 1-0 of Appendix 2I indicates Trench I location and Appendix 2.5H[81] provides the descriptive geology.

Details of the surficial and bedrock geology in the New Haven area can be found in Appendix 2I. The bedrock/till interface at faults mapped on the trench floor was closely examined for evidence of displacement. The till fabric was random and the bedrock surface smooth over mapped faults. A distinct pair of silt laminae occur continuously near the base of the lake sediments over the fault between Stations 8+85 and 9+80 along the northeast trench wall. The laminae are undisturbed and follow the topography of the lower till upon which they were deposited. These laminae were most likely laid down in proglacial Lake Iroquois, 12,500 to 10,000 yr B.P.

The silt laminae are locally contorted and warped where draped over cobbles or boulders or where rafted material has settled. Faulting and folding associated with the development of the

Demster Structural Zone have not disturbed the overlying Pleistocene deposits exposed in Trench II.

Subsurface Structure Stratigraphic and structural interpretation of data from boring alignments R-2/R-8 and R-5/P-2, combined with Trench II excavations, indicate that major stratigraphic offset in the section explored is caused by the development of the Demster Beach Anticline and not by faulting. As the structure contours indicate, the Oswego and Pulaski folding is not a single fold but a series of folds. Drilling, stratigraphic interpretation, and seismic studies surrounding the Demster Zone (Appendix 2.5I)[81] demonstrate an apparent dying out of faulting to the northeast and southwest along the Demster Zone. Only Boring R-25 (Appendix 2I, Figure 1-0) to the west of the zone intersected minor faults. Details of the tow boring alignments are given in Appendix 2I.

Subsurface data show two styles and phases of deformation, large-scale reverse-slip and small-scale normal-slip. Fracturing, calcite mineralization, and faulting decrease away from the fault zone. Offsets diminish away from the fault zone, as well as to the southwest along the zone. Principal reverse stratigraphic offsets across the zone vary from 40 to 49 m (130 to 160 ft). Within the zone of intense deformation 3 to 4.6 m (10 to 15 ft) of normal displacement was found.

At the fault zone proper, the exact amount of offset is uncertain due to complex folding, fracturing, and the necessary extrapolation of data. As depicted on Figure 3-5 in Appendix 2I, the vertical component of normal faulting is suspected to be no more than approximately 3 m (10 ft). This 3 m (10 ft) normal throw is in agreement with boring alignment data.

Structure Trench II Geological details of the trench floor and rock pits are shown on Figures 3-1, 3-7, and 3-8 of Appendix 2I. Detailed bedrock mapping covered the entire trench subgrade from Stations 8+00 to 10+40. Additional subsurface control subsequent to bedrock mapping was provided by Borings R-27, R-28, and R-29.

Bedrock along the entire 240-ft exposure of bedrock in the trench is affected by the two phases of deformation mentioned above. Resultant bedrock deformation, in and adjacent to this exposed zone, is principally due to areal folding, not faulting.

The observed gentle bedding dips (2 to 10 deg southeast) reflect the fold structural dip and not the regional homoclinal dip. Dips in the trench areas average 2 to 10 deg southeast and represent the southeast limb of a southwest-plunging asymmetric anticline (Appendix 2I, Figure 2-4).

Faulting exposed in the trench is not a single structural break, but a zone of variable deformation approximately 70 ft wide. Detailed mapping indicates the bedrock structures exposed in Trench II can be subdivided into three small-scale structural

domains for description and analysis. These domains are delineated on the basis of deformation style and structural elements. The continuity of these domains along the entire length of the fault is uncertain. However, similar domains are inferred for the R-5/P-2 boring alignment. The southeast domain, Stations 9+48 to 10+40, is characterized by relatively steep southeast-dipping strata (locally up to 50 deg). No faults or folds are observed in the southeast domain. Joints and minor bedding plane shears are the only structural elements recognized.

The central domain rocks, bounded at Stations 9+48 and 8+78 by faults with normal movement, are intensely fractured, faulted, and folded. This domain contains the greatest amount of deformation exposed in the trench and characteristically exhibits bedding plane gouge, flexural slip folding, and high-angle faulting.

The northwest domain, Stations 8+78 to 8+00, consists of gentle, southeast-dipping, Zone 1 strata. Small-scale reverse faults and joints are the predominant structural elements. Shallow bedding dips recorded in this structural domain reflect the limb of the Demster Beach Anticline; this dip appears in core boring data northwestward to approximately Boring R-2.

The detailed descriptions of Trench II are contained in Appendix 2I.

Rock Pit I

Rock Pit I, excavated 20 ft to the south of the Trench II centerline (Appendix 2I, Figure 3-9), provided a three-dimensional evaluation of the fold/fault deformation and allowed sampling of geological materials for age analysis and observation of any crosscutting mineralization. The excavated limits of Rock Pit I are primarily the central structural domain with limited vertical exposures of the other two structural domains in Trench II. The strata exposed in Rock Pit I are essentially upper Zone I with minor amounts of Zone 2 strata (Figure 2-1). Detailed geologic sections and floor maps of Rock Pit I (Stations 8+40 to 9+55) are shown on Figures 3-7 and 3-12 of Appendix 2I.

The principal brittle structural features exposed in Rock Pit I are faults, folds, and fractures. Detailed descriptions are provided in Appendix 2I.

Rock Pit II

Rock Pit II (Appendix 2I, Figure 3-9) was excavated along the toe of the northeast trench wall from Stations 9+48 to 9+15 to aid in evaluation of three-dimensional aspects of the deformation and to explore for crosscutting mineralization. This rock pit is in the central structural domain of Trench II and primarily exposes flexural slip folds and normal faults. Drag of beds associated

with the normal faulting is prominent on both walls at Station 9+25. The dragged Zone 2 strata (Appendix 2I, Figure 2-1) show minor small-scale thrusts with flexural and bedding slip (Stations 9+40 to 9+47). Details of the geology and structure of Rock Pit II are shown on Figure 3-9 of Appendix 2I.

Joints

Six joint sets were identified in the vicinity of the Demster Structural Zone with orientations as follows, in order of abundance:

	Strike	Dip
Set I	N74E	High-angle
Set II	N44E	High-angle
Set III	N44W	High-angle
Set IV	N13E	High-angle
Set V	N38E	Low-angle
Set VI	N69W	High-angle

Detailed descriptions of the nature and occurrence of these joint sets are given in Appendix 2I, as well as their relationship to regional joint sets. Fractures in the immediate vicinity of the Demster Structural Zone exhibit pervasive calcite and minor sulfide mineralization. Calcite mineralized joints decrease in abundance away from the Demster Structural Zone.

Analysis of the joint trends suggests a relationship between folding, faulting, and jointing of the Demster area. Folds identified from analysis of borehole data (Figures 2-3, 2-4, 2-6, and 2-9 of Appendix 2I) trend approximately N45E. Joint Sets II and III are essentially parallel and perpendicular, respectively, to the fold axis and are apparently tensional in origin. Joint Sets I and IV occur at approximately 30-deg angles to the N45E fold trend and apparently originated due to shear.

Set V is mainly confined to the Demster Structural Zone and appears to be associated with flexuring and bedding-plane-slip. These joints are probably contemporaneous with reverse faulting (Appendix 2I). Joint Set VI may be related to the folding.

Reverse fault movement appears to accentuate the dip of Set II in the upturned beds of the southeast domain. Also, faults coinciding with the trend of Set I reflect the reverse displacement observed throughout the northwest section of Trench II (Stations 8+11 and 8+52). Thus it appears that Joint Sets I and II developed prior to reverse faulting and are related to folding. Joint Sets I and II also served as planes of weakness during the normal phase of deformation. Within Trench II, these trends coincide with those of faulting located at Stations 8+78, 9+24, and 9+48.

Based on structural evidence from areas investigated, Joint Sets I, II, III, and IV appear to be contemporaneous with the regional northeast folding. These four sets were further accentuated during the subsequent reverse faulting phase, and Set V, localized joints, may have developed at this time. Within the central structural domain, a readjustment of Joint Sets I and II occurred at the time of normal faulting, the second phase of deformation.

Mineralization

Epigenetic mineralization in the trench proper and adjacent borings is primarily calcite with varying amounts of sulfides. The petrological and mineralogical aspects of this mineralization are described in Reference 81. Epigenetic calcite and sulfide assemblages are well developed in breccia zones, joints, and faults. This mineral assemblage is predominantly associated with sandstones and, to a lesser extent, siltstones. Gouge and shales are barren of visible calcite veins but are, themselves, calcareous.

Sulfide assemblages are essentially undeformed and generally predate calcite (Appendix 2I, Figure 3-17). Recognized sulfides are pyrite, marcasite, sphalerite, and chalcopyrite. Sulfur isotope analysis[81] indicates that these sulfides were derived primarily by bacteriological reduction of sulfate in the sedimentary environment. Thus, isotope data preclude a hydrothermal source for these sulfides.

Fluid inclusion studies on vein calcite[81] indicate a range of temperatures from 75° to 180°C (167° to 356°F). Diagenetic temperatures of the Oswego Sandstone range from 174° to 176°C (345° to 349°F) based on the studies of the Nine Mile Point Unit 2 site (Section 2.5.1.2.6)[94]. Fluid inclusion data indicate that the vein calcite was deposited at temperatures similar to those during diagenesis.

Petrologic studies indicate a definite paragenetic sequence for the calcite mineralization (Appendix 2I, Figure 3-17). Field data concerning fractures and breccias and the paragenetic sequence indicate that deformation occurred after sediment lithification and prior to last stage of calcite mineralization. The paragenesis of the vein calcite demonstrates two minor deformation events, but the last stage of calcite mineralization is postdeformation. Further evidence for this is recorded at Station 9+42 in Rock Pit I where a small vein of calcite intrudes the main gouge zone[81] and is not offset. Petrologic and fluid inclusion data indicate that the postlithification deformation and mineralization have not been disturbed since the formation of late-stage calcite.

Mineralogical Studies Mineralogical studies were undertaken to determine the type, origin, and possible age(s) of minerals associated with folding and faulting in the Demster Structural

Zone. Several techniques and investigations were used to identify the mineral assemblages and distinct mineralogical episodes and to determine the possible age(s) of faulting. These detailed results are reported in Reference 81, Attachments 1 through 5 of Appendix 2.5I.

The studies consisted of two separate approaches: one examined the formation and nature of the vein minerals, and the other examined the gouge minerals for suitable material to be dated by the K-Ar method. Investigation of the vein minerals included: microscopic examination in transmitted and reflected light; inspection of the cathodoluminescence of the calcites; study of the fluid inclusions in the calcites; and an analysis of the sulfur isotope ratios from the sulfides. Investigation of the gouge minerals included X-ray diffraction and radiometric age determination by the K-Ar method. Detailed methodology, identification of participants, and resultant conclusions are contained in Attachments to Reference 81, Appendix 2.5I; summaries and conclusions of these studies are also presented in the Appendix to this FSAR. Sample locations and studies performed are summarized in Table 1 in Appendix 2I.

An exact age of faulting and last movement cannot be assigned based on the mineralogical studies; yet, the cumulative evidence does demonstrate reasonable consistency. Fluid inclusion studies indicate that the calcite formed at depth, with an overlying rock column of approximately 2 km (1.2 mi) or more. Sulfur isotope data indicate very high δ^{34}S values, and most of the sulfide was produced by bacterial reduction of limited sulfate. Sulfur isotope data eliminate the possibility of an igneous mass as the source of the mineralizing fluid for the sulfides and calcite. Explanation of the fluid inclusion temperatures involving unknown magmatic activity must be precluded, because only nonmagnetic sulfides are present in the veins. Detailed petrographic studies of the vein minerals agree with this hypothesis.

All deformational features in the calcite are minor. Deformation occurs in the middle of the mineral sequence. Furthermore, deformation apparently was not sufficiently pervasive to open new fractures in the preexisting mineralized areas. The last stages of the mineral sequence are not deformed. Detritus (Appendix 2I, Figure 3-17) deposited during this sequence may be related to the stress relaxation interval of the structures.

Potassium-Argon (K-Ar) age determinations yield an age of approximately 400 million yr for samples of clays. However, the similarities of the clay mineralogy of the gouge samples to control samples, and the probability of partial resetting of argon in the analyzed clays, prevent a conclusive quantitative determination of the age of minerals and time of last movement of the Demster Structural Zone.

Structural Synthesis of Demster Zone Structural data substantiated by the stratigraphic sequence in the trench vicinity indicate two phases of folding and faulting for the Demster Structural Zone. These multiple deformation events have produced three separate, small-scale deformation zones. Each deformation zone in part exhibits the effects of the overall fold/fault deformation and no movement has been identified since late calcite mineralization.

Sequentially, the structural deformation appears to be of two stages or phases. The first stage of apparent compression resulted in a series of broad, low-amplitude, eastward-verging, southwest-plunging folds (Demster Beach and Mexico Anticlines and New Haven Syncline) which account for the main stratigraphic offset. This stage is manifested by a gentle southeast dip at the extremities of Trench II. With continuing compression, the steep limb of the Demster Beach Anticline was faulted in a reverse sense rather than in a left-lateral strike-slip sense, as further explained in the response to NRC Question F231.18b in Appendix 2N. Associated with the reverse faulting are small-scale, eastward-verging, northeast-plunging folds. This folding style is recognized only in the intensely deformed strata of the central structural domain of Trench II and may not have developed along the entire length of the Demster Structural Zone. The exact stratigraphic displacement due to reverse faulting could not be ascertained at the trench exposure because the second-stage structural deformation, normal faulting, modified the offset due to reverse faulting (Appendix 2I, Figure 3-16).

Normal faulting resulting from apparent extension, the final deformational event, truncated the limbs of the small-scale folds and displaced the main reverse fault at Trench II Station 9+47. This relaxation of the compressional forces resulted in outliers of Zone 2 strata (Appendix 2I, Figure 3-1) in the central structural domain.

Based on petrologic evidence and bedrock mapping of the structural features, the last stage of epigenetic calcite mineralization was emplaced after the normal fault movement. However, the earliest phases of mineralization may have occurred prior to the end of the deformation, as shown by the twinning, crushing, and detritus events identified in the paragenetic sequence (Appendix 2I, Figure 3-17). Fracturing associated with the folding and faulting provided channelways for the calcite mineralization.

Subsurface data along the R-5/P-2 boring alignment reveal the same structural style as that exposed in Trench II, and stratigraphic offset due to faulting is also apparently similar. Normal faulting appears to die out several thousand feet to the southwest, and the main stratigraphic offset there is caused by folding. Geophysical studies along the projected deformation trace indicate a lack of continuity of fracturing[81].

Structural Contour Anomalies and Geophysical Correlations

Structure Contours Stratigraphic and sedimentologic studies infer an early-Paleozoic northeast-trending subsurface structure. Rickard[52], using selected deep well data, studied the subsurface stratigraphy and structure of the Cambrian and Ordovician carbonates of New York. Structure contours were drawn on the tops of the Precambrian basement, the Knox unconformity, and the Trenton Group. As pointed out by Rickard[52], subsurface data in many areas are sparse; however, his structure contour maps demonstrate apparent north-trending subsurface faulting and folding and may show only a small portion of those structures actually present. Figures 4-2, 4-3, and 4-4 of Appendix 2I show those various contoured surfaces.

To confirm these structure contour anomalies and to contour horizons higher than the Trenton top and possibly relate these anomalies to the Demster Zone, deep exploration borehole data from Kreidler et al[11] and Hartnagel[161] were contoured. The tops of the Ordovician Trenton, Ordovician Queenston, Silurian Lockport, and Devonian Onondaga were chosen for contouring and are shown on Figures 4-5, 4-6, 4-7, and 4-8 of Appendix 2I, respectively.

Subsurface data demonstrate three salient results: 1) Ordovician through Devonian strata are deformed by folding with or without faulting; however, the true style and nature of these structure contour flexures are indeterminant, 2) the apparent north-trending anomalies of Rickard[52] are more north-northeast to northeast in orientation, and 3) the Clarendon-Linden structure, although not included on these maps, is expressed in all horizons up to Mid-Devonian where the structure apparently becomes a monoclinal element[70,76].

Interpretation of the subsurface data not only verified Rickard's[52] Trenton anomalies but extended the contours higher into the stratigraphic section. The northeasterly trend is coincident with regional geophysical and basement anomalies. Whether structure contour anomalies are due to faulting or folding or both is uncertain. Many could be interpreted as faults, and indeed drilling data, supported by geophysical data, infer basement involvement. Basement deformation is inferred particularly where retrogressive metamorphism of amphibolite grade Grenville basement is coincident with both structure contour and gravity anomalies. The deformation style of this apparent basement involvement on the overlying Paleozoics is uncertain and may include compaction structures, growth faulting, folding, and faulting.

Geophysical Correlations The gravity and aeromagnetic data covering the region surrounding the Nine Mile Point site are mutually consistent in defining a northeast-trending structural fabric in the crystalline basement. Previously, Rickard[52] had identified essentially north-south basement trends based on

limited boring data. The northeast trend is similar to the structural fabric of the Central Metasedimentary Belt north of the site and in the region west of the site[8,70].

The geophysical data consist of gravity and aeromagnetics. The gravity data are responsive to density changes, whereas magnetic data are responsive to susceptibility changes in the basement rock. Although it is an oversimplification, the gravity anomalies in this area can be attributed, at least in part, to basement rock topography whereas aeromagnetic data are more related to basement lithologies. In some instances, gravity highs are coincident with magnetic lows, whereas in other locales gravity lows are located in the same area as magnetic highs. However, this apparent inconsistency is resolved with borehole data and regional geologic information. Several of the broad and "simple" gravity anomalies are characterized by a complex series of magnetic anomalies. The geophysical data indicate that the crystalline basement in this portion of New York State is composed of a complex assemblage of rock types within a dominantly northeast-trending structural fabric.

The gravity and magnetic data for central New York provide confirmation of inferred structures and indicate a northeast fabric for the region (Figures 2.5-8 and 2.5-9 and Figures 5-1 through 5-3 of Appendix 2I). The geophysical data support the interpretation that the probable faulting in the Cross Lake area is related to basement uplift and probable alteration of the basement rocks along a northeast trend. The structural high inferred in the Camden area is supported by a gravity high at the same locality. The Demster Zone proper does not have a distinctive geophysical signature. This would suggest either limited or no direct basement control; however, it could have resulted as an indirect consequence of basement deformation.

Summary

Structural and stratigraphic relationships in the Demster Structural Zone show that the New Haven area has been deformed by two sequences of paleotectonic activity: initial broad folding culminating in reverse faulting and later normal faulting. No other tectonic activity is documented at the Demster Structural Zone. Calcite paragenesis indicates no deformation subsequent to the youngest sequence of minerals, and Pleistocene surficial sediments overlying the fault zone are not deformed.

The K-Ar data may suggest a Middle-Paleozoic (Silurian) time of deformation for the Demster Structural Zone. The reconstructed geologic column, associated geologic history, and other interpretations of data suggest a Middle- to Late-Paleozoic age. A younger Late-Jurassic age[107] cannot be ruled out, although the sulfur isotope data do not strongly support this age. The uncertainty of the timing of alkaline emplacement based on the geochemical data, plus lack of documented high-angle Late-Mesozoic faulting, place constraints on this time interval.

Consequently, a Middle- to Late-Paleozoic age is inferred for the final development of the Demster Structural Zone.

The deformation style, the northeast trend of the structural elements, regional stratigraphy, and analytical data are in agreement that the Ordovician strata in northern Oswego County, and conceivably the underlying Cambrian and Ordovician strata in central New York, have undergone broad areal folding with variable reverse and normal faulting. Combined geologic and geophysical data indicate that the Demster Structure is not capable.

2.5.1.2.5 Relationship of Site Structures to Near-Site Structures

The important near-site structure, the Demster Structural Zone, is described in Reference 81, Volume 12, Part I, Appendix 2.5I (Section 2.5.1.2.4 and Appendix 2I). This detailed description is included herein by reference.

The Demster Structural Zone of folding and faulting trends northeastward. Its surface trace is approximately 4,570 m (15,000 ft) southeast of Nine Mile Point. This zone is characterized by tight to broad, eastward-verging, asymmetric, locally-overturned folds; flexural slip; reverse faulting; normal faulting; and associated drag folding. The deformation resulted from at least two phases of essentially contemporaneous movement, an initial stage of folding and reverse faulting followed by a stage of relaxation and normal faulting.

The faults, both normal and reverse, strike northeasterly and dip steeply to the northwest; maximum throws are no more than a few feet. They occur in an elongate domain of closely jointed and highly broken rock that is transected by several zones of breccia-free gouge with trends both parallel and normal to the strike of bedding and the strike of the faults. In this ground there is no evidence of dilation such as that observed at Nine Mile Point, where the gouge and breccia are packed tightly along the nearly vertical faults. None of the faults offset the surface of bedrock or cut the overlying Quaternary glacial and lacustrine sediments. These faults are probably Paleozoic (possibly Alleghanian) in age, and certainly are no younger than Mesozoic in age. They are not capable faults within the meaning of 10CFR100, Appendix A (Section 2.5.1.2.4).

The geologic structure at the Unit 2 site (Section 2.5.1.2.3) is expressed by two steeply dipping, northwesterly striking normal faults that bound a block of gently dipping sedimentary rocks that are cut at shallow depths by a series of subparallel thrust faults grossly concordant with the host-rock bedding. The uppermost thrust fault dips southeastward at low angles across the bedrock strata where it is exposed in the excavation for the radwaste building. There the leading portion of the faulted rock is crumpled and dilated in very close similarity to the

valley-bottom faults of the Upper Ohio River valley that were produced as a result of erosion of the bedrock in the river valley and the consequent reduction of the least principal stress in the bedrock (σ_3). Ferguson[97] discusses these faults and their origin. The relationships observed along the thrust zone also are similar to those associated with shallow-seated breaks noted in many New England granite quarries, where small thrust movements and numerous expressions of dilatancy also represent geologically young stress relief related to the ground surface[154].

The openwork along the Radwaste fault contains in-fillings of lacustrine sediments containing pollen, which indicates a late-Wisconsinan age (10,000 to 13,500 yr B.P.) for those sediments. The faulting appears to have been geologically young, with movements that probably occurred during Pleistocene time in response to episodes of glacial loading and unloading. Holocene (post-Pleistocene) movements have been small if they have occurred at all. It cannot be demonstrated that no Holocene movements have occurred, as no dated in-filling sediments or other reference features extend entirely across the zones of disturbance.

Displacement of late-Pleistocene lake sediments has occurred along the southerly bounding fault, the Cooling Tower fault. Both of the normal faults are marked by thin zones of gouge, and both have displacements of a few feet or less.

The strata at the Unit 2 site are essentially undisturbed except right at the faults, in contrast to the broken and highly disturbed strata in the Demster Zone. The faulting in the Demster Zone may well have been of direct tectonic origin, whereas the much younger faulting at the Unit 2 site, and especially that along the Radwaste thrust, is readily explainable as a result of local stress relief unrelated to major or contemporary tectonic activity. Such relief, as widely expressed in this and other regions, derives through failure, within a highly anisotropic stress field, of rocks containing a combination of residual strain energy and strain energy inherited from earlier tectonic activity.

As discussed in Section 2.5.1.2.3, reduction of the vertical load (σ_3) as a result of Pleistocene glacial erosion facilitated westward slip of the radwaste structure's hanging wall block by reducing the shear resistance of the block to stresses acting subhorizontally (σ_1). This is similar to the mechanics of formation of the valley-bottom faults in the Upper Ohio River basin and of sheet structure and associated exfoliation phenomena in more massive rocks of the northeastern United States.

Formation of the bounding normal faults at Nine Mile Point probably resulted from adjustments during late stages of the Appalachian geosyncline as the bottom ceased to subside and

oxidizing processes began. Minor Quaternary movements could be expected in the form of much younger readjustments associated with erosional unloading of the bedrock section.

The Demster folds and faults and the Radwaste thrust faulting are quite different geologic structures in terms of respective sizes, extents, attitudes, degrees of brecciation, origins, and ages. The Demster Zone is tectonic in origin, whereas the Radwaste structure is a result of much younger unloading by accelerated erosion. In both occurrences the causative processes are no longer active, and the faults are not capable.

The steeply dipping normal faults, striking northeasterly at New Haven and northwesterly at Nine Mile Point, are related in origin to late-stage processes in the evolution of the Appalachian geosyncline, and perhaps to the extensional forces that opened near-vertical channels for emplacement of ultrabasic dikes in the adjacent Finger Lakes and Syracuse region during Mesozoic time. Deformation of the Quaternary lake sediments along the Cooling Tower fault at Nine Mile Point evidently resulted from localized buckling related to removal of the ice load following recession of the Wisconsinan ice sheet. These normal faults are not tectonically capable (Section 2.5.1.2.3).

2.5.1.2.6 Geologic History

The current understanding of the site geologic history is derived from several sources: regional geologic history (Section 2.5.1.1), detailed mineralogic studies[94,154], and various structural relationships observed at and near the site (Sections 2.5.1.2.3 through 2.5.1.2.5). The site is located within a region that has been subjected to nonorogenic tectonic deformation and glacioisostatic adjustments, both of which are characterized by vertical crustal movements. These broad-scale movements, diagrammatically shown on Figure 2.5-28, precipitated a geologically simple, yet mechanically complex, sequence of events that compose the history of this site.

During Late-Precambrian time, a long interval of crustal uplift and concomitant erosion occurred. As a result, the highly deformed and metamorphosed basement rocks that are similar to lithologies currently exposed in the Adirondack Massif and Canadian Shield (Section 2.5.1.1) were exposed across the site area.

During early Paleozoic time the previous trend toward uplift was reversed and subsidence began. This downwarping of the crust signaled the initial development of the Appalachian Orogen. Throughout most of early to middle Paleozoic time, the site was situated in the midst of a vast subsiding sedimentary basin. Within the basin a westward thickening wedge of predominantly clastic lithologies were being deposited as a result of the intense orogenesis occurring to the east. Approximately 600 m (2,000 ft) of this sedimentary sequence accumulated prior to the

deposition of the site strata. These strata, which are currently exposed at the site, represent a shallow water phase of deposition in the basin history coincident with the culmination of the Late Ordovician Taconic Orogeny in the Valley and Ridge Appalachians Province. Subsidence and deposition continued, and it is estimated that 2,440 to 3,050 m (8,000 to 10,000 ft) (Section 2.5.1.1) of younger sedimentary rocks buried the site strata before deposition within the basin ceased (Figure 2.5-70). This maximum depth of burial is interpreted on the basis of regional relationships (Section 2.5.1.1) and the homogenization temperatures (176° to 147°C, 349° to 297°F) of fluid inclusions within grains of the host rock and secondary fracture-filling minerals (Figures 2.5-71 through 2.5-73).

Subsidence and deposition ended during middle- to late-Paleozoic time and reflect the cessation of the regional orogenesis (Figure 2.5-70). The first deformation (D_1-D_2, Figures 2.5-69 and 2.5-73) of the site rocks is interpreted to have occurred in late-Paleozoic time. The conjugate fractures and strike-slip faults (Section 2.5.1.2.3) within relatively brittle lithologic layers developed at this time. The homogenization temperatures of mineralization associated with these deformations range from 160° to 120°C (320° to 248°F) (Figures 2.5-73 and 2.5-74). Mineralization within the Demster Structural Zone displayed similar temperatures; hence it is interpreted that this structure was also active at the time (Section 2.5.1.2.4).

The history of the site during the Mesozoic and Cenozoic eras was characterized by uplift and erosion (Figures 2.5-69 and 2.5-70). Differential uplift resulted in a southward regional tilt of 9.5 m/km (50 ft/mi) (0.5 deg). Erosion accompanying uplift removed approximately 3,050 m (10,000 ft) of Paleozoic rocks which had previously overlain the site strata. These eroded sediments were likely deposited in the Triassic Basins and the Coastal Plain to the southeast[130].

A second phase of deformation (D_3-D_4 on Figures 2.5-69 and 2.5-73) affected the site vicinity probably during early- to late-Mesozoic time. This episode of deformation resulted in normal faulting along the strike-slip faults (Section 2.5.1.2.3), extending them into previously unaffected strata. Structural relationships described in Section 2.5.1.2.3, as well as mineralization associated with normal faulting with homogenization temperatures of 116° to 73°C (241° to 163°F) (Figures 2.5-73 and 2.5-74), confirm that this deformation postdated the strike-slip faulting.

At Nine Mile Point there is a definite relationship between the structural fabric associated with the normal faults, and evidence of a major geochemical change during crystallization of the calcite minerals on the faults. This relationship is exhibited by the transformation from sulfide minerals to goethite at decreasing temperatures which represents a change from reducing conditions to oxidizing conditions caused by the influx of

convective, air-saturated ground waters[94]. It is believed that the change, more importantly than the distribution of homogenization temperatures, must be regarded as time-line of regional extent because it is also recorded in the mineral data from the New Haven site[81] (Section 2.5.1.2.4).

Apparently, the regional environmental change to oxidizing conditions corresponds to the cessation of the tendency for the northern Appalachian Basin to subside. The history of sedimentation ceased in conjunction with the late stages of mountain building processes in the Appalachian geosyncline in the site area during late-Paleozoic time. The literature documents that the region around the site was subjected to extension in Mesozoic time as evidenced by the occurrence of ultramafic dikes that intruded the crust in the area of the Finger Lakes and Syracuse (Section 2.5.1.1.2). Hence, deformational events D_3 and D_4, representing normal faulting at Nine Mile Point, are interpreted to be of late-Paleozoic to Mesozoic age[94]. Furthermore, the data from both the structural fabric and mineralization studies from the Demster Structural Zone (Section 2.5.1.2.4) are in accordance with those from Nine Mile Point, and imply a similar age for the normal faulting at the New Haven site.

Uplift and erosion of the site vicinity continued through the Tertiary and Quaternary Periods. During the Pleistocene Epoch the site underwent repeated glaciations[24]. The crustal depression and rebound associated with each glaciation were superimposed on the continuous tectonic uplift. Glacioisostatic movements and accompanying environmental effects played an important role in the buckling along the preexisting north-dipping, high-angle faults, and in the development of the low-angle thrust structures (Section 2.5.1.2.3). The development of these structures appears to be interrelated. It is certain that both developed at relatively shallow levels in the crust because of the character of deformation. In particular, the study of the Radwaste and Intake Shaft faults[154] has shown that calcite minerals are present along shear planes and on some open vertical fractures within the zone of deformation. These minerals are deformed (D_5-D_6 on Figure 2.5-75). Studies of this mineralization indicate that all of the calcite occurring on the low-angle structures is younger than the latest stage of epigenetic (high-temperature) calcite reported from the site.

Initial buckling on the high-angle faults had occurred prior to the Wisconsinan glaciation (Section 2.5.1.2.3). This initial buckling, together with the erosion of the Wisconsinan or older bedrock valley, have been interpreted to be necessary for the development of the thrust structures (Section 2.5.1.2.3 and response to NRC Question Q361.28). Drs. Philbrick and Jahns[154] (NRC Question Q361.1) have concluded that:

1. The structure was initially developed in pre-Holocene time and in the Illinoian time interval between 500,000

and 140,000 yr B.P. with glacial erosion of rock and consequent reduction of vertical confining pressure. That at least most of its movement occurred in pre-Holocene time is indicated by the partial filling of structurally-formed openings by silts that are about 11,000 yr old.

2. Initial formation of the structure probably was abrupt, with displacements at a given place relatively large at first and attenuating with time.

3. Movements along the structure probably occurred during Pleistocene time, as prompted by episodes of glacial loading and unloading.

Fluid pressure changes accompanying the draining of Late-Wisconsinan proglacial Lake Iroquois augmented the second stage of buckling on the high-angle faults. This stage postdates lacustrine sediment, dated to be 12,200 to 10,400 yr old[154].

The Holocene history of the site is predominated by crustal uplift principally related to glacial rebound. Holocene movements have been small, if they have occurred at all.

2.5.1.2.7 Engineering Geology

All safety-related engineering structures at the Unit 2 site are founded on bedrock. Detailed evaluations of the engineering geology aspects of the geologic features at the site indicate that conditions of in situ stress, bedrock lithology, rock mass anisotropy, and groundwater fluctuations are important elements that had to be considered in the design. In addition to the above, the dynamic behavior of the site during earthquakes is discussed in Sections 2.5.2.5 and 2.5.4.2.2. The purpose of this section is to summarize the pertinent engineering geology aspects of the site. Detailed geologic maps of the excavations and specific discussion of the subsurface materials are provided in Section 2.5.4.

There are several zones of bedrock deformation that intersect the site excavations. The cooling tower (Figure 2.5-28) was relocated to avoid being founded above a fault along which Quaternary buckling had occurred (Section 2.5.1.2.3). Several Quaternary, low-angle thrust faults intersect the main site excavations (Figure 2.5-26) and are described in Section 2.5.1.2.3. These faults were judged by a panel of experts[154] to have a negligible impact on the site engineering structures.

In situ stresses in the bedrock, ranging from low-magnitude tensile stresses to compressive stresses of more than 141 kg/sq cm (2,000 psi), were measured at the site[94,154]. The referenced reports contain detailed interpretations of the wide range of stresses with respect to the geologic structures at the site.

Section 2.5.4.1.4 discusses these results with regard to the engineering geology of the site.

The anisotropic character of the bedrock is related to the variation of physical properties of the rock resulting from compositional variations. Static properties have been determined and are presented in Section 2.5.4.2. Dynamic properties were determined from geophysical surveys, and the results are contained in Section 2.5.4.4.

Large-scale fluctuation of the groundwater table is a factor which, because of the stress conditions at the site, can influence the stability of planes of structural weakness in the bedrock mass. This aspect is discussed with respect to historical geology of the site in Section 2.5.1.1. Further information on groundwater conditions is presented in Sections 2.4.13 and 2.5.4.6.

Section 2.5.4.13 outlines the program of subsurface instrumentation at the Unit 2 site. Monitoring of various instruments for information about possible strain in the bedrock and in critical engineered structures is planned for the period ending January 1985.

Geological hazards and hazards related to man's activities within the site region are described in Section 2.5.1.1.5. The hazards discussed are not considered to have an adverse impact on the site. However, several phenomena are considered to be potentially capable of resulting in small movements within the site bedrock. The potential impact of these phenomena has been considered in the design of the site foundations.

The design criteria developed in consideration of various aspects of the site engineering geology are discussed in Section 2.5.4.

2.5.2 Vibratory Ground Motion

2.5.2.1 Seismicity

The history of seismic activity in the northeastern United States and adjacent Canada is typical of intraplate tectonic regimes, showing only a few shocks that can be classified as major over the approximately two to three centuries of historical record. These significant events, reported mostly in the earlier record, can be associated with what has since been identified as zones of relatively more intense earthquake activity, as discussed in Section 2.5.2.2. Only two of the recognized zones (Attica, NY, and Cornwall-Massena) occur within 320 km (200 mi) of the site, and the site locale itself may be characterized as virtually aseismic within a 145-km (90-mi) radius.

The reliability of the historical record, particularly the early portion, is open to question, being highly subject to bias introduced by population distribution, construction materials and

techniques, inaccurate or exaggerated reporting, and misinterpretation of man-made or natural events such as explosions and storms. To derive a more homogeneous total record, pertinent agencies have reviewed the original historical data, and have reevaluated portions of the record in terms of current perspective and methodology. The Geological Survey of New York (New York State Museum and Science Service) has conducted a detailed review of the New York State seismicity utilizing the original sources. Similarly, the Dominion Observatory of Canada has reevaluated the Canadian record. Several private organizations have also reinterpreted various specific events for reactor siting studies in the Northeast. The results of these reviews are used in this report.

Table 2.5-2 is a listing of all earthquakes classified as greater than Modified Mercalli Intensity IV (MMI) and/or Richter magnitude (m) 3.0 within a 320-km (200-mi) radius of the site. Also included are all events, regardless of size, reported within 80 km (50 mi) of the site. These are listed in order of increasing distance from Unit 2, and are superimposed on the tectonic provinces of the site region on Figure 2.5-12. Significant shocks outside the 320-km (200-mi) radius are not listed or shown, but their effects on the site and influence on seismic design for the Unit 2 installation is taken into consideration and discussed in subsequent sections. The most significant events within the 320-km study area are discussed below.

The Cornwall-Massena earthquake of September 5, 1944, (MMI VIII) occurred at 12:39 a.m. Eastern War Time. The epicenter was at about 45.0°N, 74.7°W, about 200 km (124 mi) northeast of the site. The shock was felt in an area of about 453,250 sq km (175,000 sq mi) in the United States. The epicenter was about midway between Cornwall, Ontario and Massena, NY. The damage loss resulting from the earthquake amounted to about 2 million dollars. At Massena, the shock destroyed or damaged 90 percent of the chimneys. Masonry, plumbing, and house foundations were also damaged and windows were broken. Many structures were rendered unsafe for occupancy until repaired. A large number of wells in St. Lawrence County, NY, went dry; water levels were affected in streams and wells as far away as Westchester County and Long Island, NY. Fourteen aftershocks were noted by November 1. The strongest, on September 9 at 7:25 p.m., was nearly equal to the intensity of the main shock. It is significant that structures founded on rock or compact till soils in the epicentral areas did not sustain any appreciable damage. The greatest damage in the epicentral area was suffered by structures founded on outwash sand, silts, and clays[162]. Deposits at the Unit 2 site would have undergone intensities no greater than V, as shown on Figure 2.5-81.

The Attica earthquake of August 12, 1929, occurred at 6:25 a.m. EST, about 165 km (103 mi) west of the site. The epicenter was at about 42.9°N, 78.4°W. The total affected area was about

129,500 sq km (50,000 sq mi) including parts of Canada. The shock was felt most strongly in the eastern part of the city of Attica and the region lying immediately to the east. In the region immediately to the south there was less effect on structures, but changes in groundwater conditions were noted. The shock was accompanied by sounds compared to thunder. It was reported that 251 house chimneys collapsed in Attica. Recent investigations in Attica indicate that many of these occurrences probably consisted of bricks falling from the tops of still-standing chimneys. Walls were cracked and instances of fallen plaster were common. Objects were thrown from shelves, monuments in cemeteries toppled, and a number of wells went dry. The degree of damage to structures generally could be related to the type of design and construction. As an example, three churches and a school within one square block area experienced different degrees of damage. The Methodist Church, considered of poor design, was badly damaged; the Presbyterian Church, a simpler structure, was less damaged; the Baptist Church, a still simpler structure, was scarcely damaged at all. The Attica High School, a new three-story building considered to be of good construction had damage limited to minor plaster cracking. Damage occurred to portions of the Westinghouse factory southeast of the city. The building is of brick construction. Damage was greatest in the east-west wing of the T-shaped building due to the collapse of a brick chimney. Based on a review of damage effects, an MMI of VII and a magnitude of 5.2 has been assigned to this event[163].

Shocks smaller than the preceding Attica event have been reported fairly frequently in the Attica area. Such shocks occurred in December 1929, 1939, and 1955; July and August 1965; January 1966; and June 1967. The largest of these were the two most recent, each reported as being MMI VI in the epicentral area. These shocks probably resulted in intensities at the site of less than III[164]. The shock of January 1, 1966, (MMI VI) occurred at 8:24 a.m. EST with two similar foreshocks reported at 5:30 a.m. and 6:30 a.m. The shock was accompanied by thunderous sounds and was felt by many. A few chimneys were damaged slightly, cellar walls and foundations cracked, bricks tumbled from houses, and plaster fell at the state prison. The shocks were felt over approximately 9,065 sq km (3,500 sq mi) of western New York, northwestern Pennsylvania, and southern Ontario, Canada. The shock was most strongly felt at Varysburg. The shock of June 13, 1967, (MMI VI) occurred at 2:09 p.m. EST and was accompanied by loud noises compared to a sonic boom. The shock was felt over an area of about 7,770 sq km (3,000 sq mi) in western New York. Slight damage to plaster and ceilings was sustained at Attica and Alabama, NY, where the shock was felt by many. Some chimneys cracked at Attica.

The Buffalo-Lockport (NY) earthquake of October 23, 1857, occurred at 3:15 p.m. EST. The epicenter was about 43.2°N, 78.6°W, and the shock affected a total area of about 46,620 sq km (18,000 sq mi). The MMI of VI was felt in an area about 120 km

(75 mi) long from north-northeast to south-southwest, and about 97 km (60 mi) east-west. It was felt at Hamilton, Petersborough, and Port Hope, Ontario; Rochester, NY; Warren, PA; and also reported felt at Dayton, OH. In Buffalo, a man seated on a chair was thrown to the ground, walls vibrated and surged, bells rang, and crocks fell from shelves. At Lockport, rumbling noises were heard for a full minute. The shock was felt by many, but caused no serious damage.

In addition to the 1857 shock, several smaller events were experienced in the Buffalo-Hamilton, Ontario area in 1879, 1944, 1946, and 1962. The 1962 shock is typical. It occurred at 1:37 p.m. EST. The epicenter was located near Niagara Falls, NY. The shock was accompanied by a loud noise and was felt by many. Beds and chairs were displaced, houses shook, and dishes rattled. The shock was a maximum of MMI V and was felt over a relatively small area.

Effects at the Unit 2 site from the large, distant shocks of record, such as the Charleston, SC, event of 1886, the New Madrid, MO, event(s) of 1811-12, and the more significant (MMI IX-X) events in the upper St. Lawrence Valley would have been minimal. Using empirical relationships[165] to correlate MMI with acceleration and considering the effects of attenuation, it is conservatively concluded that accelerations resulting from any historical events in the entire region have been no more than 0.05 g at the Unit 2 site. Intensity-acceleration relationships are discussed in more detail in Section 2.5.2.3.

2.5.2.2 Geologic Structures and Tectonic Activity

2.5.2.2.1 Introduction

The Unit 2 site is in a region characterized by rocks at the surface which, although very old, have not been subjected to large-scale orogenic processes (Section 2.5.1.1). In consequence, few major structural features are known within 161 km (100 mi) of the Unit 2 site.

Low-level seismicity is known to occur throughout the northeastern region of the United States, but the distribution of historic and instrumentally detected events appears in most instances to be unrelated to movement on either specific or known geological structures. The site region exhibits very low seismicity. The lack of a well-defined relationship between seismicity and geologic structure leads to a conservative assessment of the design values for vibratory ground motion at the site based upon the delineation of tectonic provinces as required by Appendix A to 10CFR100. Therefore, several approaches or models have been used. The approach used by Hadley and Devine[28] is one of the most conservative available for selection of the safe shutdown earthquake (SSE) and operating basis earthquake (OBE) for the Unit 2 facility; this model,

slightly modified, is used and described in detail in the following sections.

2.5.2.2.2 Central Stable Region Eastern Stable Platform

Eastern Stable Platform Sub-Province

The Unit 2 site is situated within the Eastern Stable Platform sub-province of the Central Stable Region[28] (Figure 2.5-12). The Central Stable Region tectonic province is a vast area represented by the interior portion of the North American craton. The cratonic part of the continent was affected by only convergent, diastrophic processes[166]. The central eastern portion of the craton is represented both by exposed and buried continental crust ranging in age from 1,000 to 1,450 million yr[7]. The exposed basement rocks occur as part of the Grenville (Geologic) Province of the Canadian Shield and the Adirondack Massif (Figures 2.5-3 and 2.5-12). The buried Grenvillian rocks in this tectonic province are covered by Paleozoic sedimentary rock (Cambrian to Permian) deposited in a downwarped basin (Figure 2.5-77). This basin is bounded on the west by a broad structural rise, the Findlay Arch. The strata of this basin extend eastward, beyond the Eastern Stable Platform Sub-Province, until they meet the Hudson Highlands anticlinoria[166], which separate non- or slightly metamorphosed basinal facies from the more intensely altered ones. These strata form a sedimentary wedge, thickening to the southeast, reflecting the asymmetry of the basin floor (Figure 2.5-78). The eastern boundary of this sub-province marks the transition to the region of thin-skinned folding and thrust faulting of the Fold Belt.

West of the Findlay Arch-Algonquin Axis (outside of the Eastern Stable Platform Sub-Province) lie the Michigan and Illinois Basins, contemporaries of the Appalachian Basin. The Anna, OH, area lies at the confluence of three reentrant zones of these contemporary basins: 1) the boundary of the Kankakee and Findlay Arches with the Michigan Basin, 2) the boundary of the Kankakee Arch and Indiana-Ohio Platform (northern portion of the Cincinnati Arch) with the Illinois Basin, and 3) the boundary of the Findlay Arch and the Indiana-Ohio Platform with the Appalachian Basin. Studies by Dames & Moore[167] indicate that, just as in the vicinity of Attica, NY, along the Clarendon-Linden Fault, the basement rocks in the Anna area reflect a strong north-south magnetic anomaly, coincident with the Bowling Green Fault. Likewise, a focal mechanism solution for the March 8, 1937, MMI VII-VIII shock at Anna has a nodal plane (north-south orientation) that is in agreement with postulated basement faulting. On the basis of regional geophysical studies and the results described above, the Anna seismogenic zone was defined as lying between 84° and 84°30' longitude, north of the northwesterly-trending Champaign Fault and south of the northern limit of a band of acidic igneous intrusive rocks which contrast strongly with other basement rocks in the region[167].

The deformational history of the basement rocks is long and complex involving both ductile and brittle episodes; nevertheless, the overlying Paleozoic sedimentary rocks of the Appalachian Basin within the site province reflect a long history of very minor deformation. Broad upwarps like the Findlay Arch-Algonquin Axis are bordered by moderately deep depositional basins containing Paleozoic strata. These upwarps and complementary basins developed gradually throughout Paleozoic time under stress conditions that reflect vertical tectonic movement of the crust[28]. Local stress concentrations developed and resulted in high-angle faulting and mild folding. In this subprovince, the Clarendon-Linden Fault (Section 2.5.1.1.3) and the Bowling Green and Champaign Faults in the Anna, OH, area are examples of the effect of these conditions in Paleozoic time.

Brittle faulting occurred in this subprovince during Mesozoic time also; the association of normal faults, alkaline igneous activity, and the formation of domes indicates that the crust was subjected to extensional strain to great depth[21,37,166].

Since Mesozoic time, the Eastern Stable Platform (and adjoining tectonic provinces) has undergone extensive epeirogenic uplift and concomitant erosion. These processes have resulted in tilting of the basin floor and sedimentary strata southward with a gentle dip of 1/2 to 1 deg (Figure 2.5-78). This structural feature is known in the site region as the Lake Ontario Homocline[80].

The contemporary stress conditions in the site tectonic subprovince (Eastern Stable Platform) are characterized by high (subhorizontal compressive) stress near the upper free surface of the crust[120]. The Unit 2 site occurs within the Mid-Continent stress province[121] (Figure 2.5-79), which is characterized by east-northeast trending, maximum compressive stress, essentially parallel to the absolute motion of the North American Plate.

Approximately 95 km (59 mi) southwest of the site, a deep boring (1,540 m, 5,053 ft) was recently drilled (under sponsorship of the New York State Energy and Resource Development Agency [NYSERDA]) to explore the geothermal potential of basement rocks in the Auburn, NY, area. Preliminary data have been obtained by the U.S. Geological Survey, who performed hydrofracture tests in the boring, and their report on these data is currently in progress.

In 1929, a MMI VII earthquake occurred near Attica, NY, approximately 145 km (90 mi) west of the site. This shock and an accompanying concentration of lesser events has been attributed to movement along the Clarendon-Linden Fault[168,169]. A shock of MMI VI in 1853 near Lowville, NY, about 80 km (50 mi) northeast of the site, may be related to faulting expressed in the Paleozoic strata at the western edge of the Adirondack Massif. The faults are oriented northeast-southwest, downthrown an unknown amount to the southeast.

MMI VII-VIII shocks in the area of Anna, OH, should not be assumed to occur at the site because they have been attributed to movement along known basement faults in the Anna region[167]. Furthermore, this active zone lies west of the boundary (Findlay Arch-Algonquin Axis) of the site subprovince (Section 2.5.2.3).

Ottawa Basin Sub-Province

The Ottawa River Basin is characterized by a west-northwest striking zone of faults called the Ottawa-Bonnechere Graben (Section 2.5.1.1.3). The character of this feature is unlike the other portions of the Central Stable Region tectonic province lying east of the Findlay Arch-Algonquin Axis. Consequently, one can subdivide this portion of the Central Stable Region with another subprovince, namely the Ottawa Basin Sub-Province, which is distinct from the site subprovince.

The Ottawa Basin Sub-Province lies immediately to the north of the site tectonic province. The boundary between them marks the position of the extension of the New England Salient as defined by Cady[170] into the Eastern Stable Platform. The Ottawa Basin is characterized by the development of a deep, Cambro-Ordovician structural basin along an older zone of crustal weakness. This basin was subsequently deformed by a branching system of normal faults: one west-northwesterly branch formed the Ottawa-Bonnechere Graben, and the other the St. Lawrence Rift Valley with northeasterly trending normal faults[37]. The eastern border of the Ottawa Basin corresponds to Logan's Line.

The early record indicates that events of MMI IX and X occurred in the 1600s and 1700s in the St. Lawrence River Region, and a MMI IX event occurred in 1925 near LaMalbaie, Quebec. In 1944 a maximum MMI VIII shock occurred between Massena, NY, and Cornwall, Ontario, a distance of 177 km (110 mi) from the site. These events, if migrated to the boundary of the Ottawa Basin Sub-Province with the site tectonic subprovince (175 km, 109 mi from the site), would subject the site to about MMI VI using the attenuation relationships shown on Figure 2.5-80.

2.5.2.2.3 Northern Appalachian Fold Belt

The Northern Appalachian Fold Belt tectonic province (Figure 2.5-12) adjoins the site province on the south and east. Hadley and Devine[28] define the Central Appalachian Fold Belt tectonic province. The province consists of strongly folded Paleozoic sedimentary strata 3 to 10 km (1.9 to 6.2 mi) thick, broken by low-angle thrust faults (decollement structures) that are largely confined to the sedimentary strata and rarely involve the Precambrian basement rocks. The fold belt changes considerably in width along its length from the southern Appalachians to New England. It is nearly 190 km (118 mi) wide in Pennsylvania and only about 30 km (19 mi) wide in eastern New York State[28]. The western boundary of the province delineates the point where

orogenic folding of the Paleozoic sedimentary strata of the basin dies out. The eastern tectonic province boundary is marked by the abrupt change to basement rocks in the cores of the Hudson Highlands anticlinoria (Figure 2.5-12) and highly alteredmetasediments that characterize the eugeosynclinal deposits of the Appalachian Geosyncline.

Hadley and Devine[28] subdivided this tectonic province into two parts: the faulted and nonfaulted fold belts (Figure 2.5-12). Moreover, they noted that the tectonic style of deformation in the northern portion of this province differs notably from the southern portion. This change of tectonic style is related to the geological evolution of the province, and is manifested by the dominance of folding in the north as compared to overthrust faulting in the south.

The northeast structural trends of the Appalachian Fold Belt are transected by an east-northeast trending belt in southern Pennsylvania known as the Central Appalachian Salient[29,171,172]. This salient was likely developed during late Precambrian time in association with rifting and the concomitant development of the proto-Atlantic Ocean[173]. Whether this feature is related to an aulocogen[29] or to transcurrent or transform faulting[79,174,175] is uncertain. However, it seems to be a fundamental crustal feature with no evidence of post-Cretaceous deformation.

The compressional forces that led to the development of folds and thrust faults in this tectonic province were generated during the continental convergence represented by the Taconian, Acadian, and Alleghanian orogenies during the Paleozoic Era (Section 2.5.1.1). During the late stages of the Alleghanian event, the effects of compressional forces were more pronounced in the southern Appalachians (south of the Central Appalachian Salient) than they were in the northern Appalachians.

During the divergent rifting of the continent (opening of the Atlantic Ocean) in the Mesozoic Era, rocks of the Northern Appalachian Fold Belt tectonic province were subjected to extensional strains that were most pronounced along its eastern boundary and manifest in the development of the Triassic-Jurassic basins of Maryland, Pennsylvania, and New Jersey. Minor igneous activity associated with rifting also affected the province, as scattered diabase dikes of Triassic and Jurassic Age, plus kimberlite and peridotite dikes of late-Jurassic Age that were intruded into upper levels of the crust. Mild folding recognized along the western border of this province[17,44] (Section 2.5.1.1) may have occurred in late-Jurassic time. During this era, the orientation of stresses of the northern Appalachians differed significantly from that of the southern Appalachians[176], as evidenced by the regional change in strikes of Mesozoic diabase dikes.

From Cretaceous time to the present, the province has been subjected to epeirogenic uplift and erosion. The majority of

this province occurs within the Central Lowlands Canadian Shield stress province of Zoback and Zoback[121] (Figure 2.5-79), which they believe is characterized by high subhorizontal, compressive stress, with the maximum stress oriented east-northeast.

Hadley and Devine[28] show that the Northern Appalachian Fold Belt tectonic province, as is true for the Central Stable Region, exhibits both a relatively aseismic area (the nonfaulted fold belt) and a moderately seismic area (the faulted fold belt). Based upon the Southern Appalachian Seismic Zone of Bollinger[177], they further recognize that the southern portion of the faulted fold belt is more seismic than its northern counterpart. Hence, it seems reasonable that the Central Appalachian Salient divides the Fold Belt province with respect to the distribution of seismic events (only the Northern Appalachian Fold Belt is shown on Figure 2.5-12). The largest event known in the Southern Fold Belt tectonic province was the 1897 Giles County, VA, earthquake (MMI VIII). To the north of the salient, no events greater than MMI VI have been recorded within the Northern Appalachian Fold Belt tectonic province. (The February 1954 localized shock near Wilkes Barre, PA, [MMI VII] has been attributed to mine collapse)[178].

2.5.2.2.4 Appalachian Mobile Belt

Hadley and Devine[28] considered the Blue Ridge, Piedmont, and Green Mountain Belt to be part of a single, unnamed tectonic province. The rocks of this province were affected in the geologic past by initial divergent and convergent diastrophic processes in Precambrian and Paleozoic time, as well as the final divergent diastrophism in the Mesozoic Era. For this reason, it is termed the Appalachian Mobile Belt[166] (Figure 2.5-12). The province corresponds to the eugeosynclinal belt and includes the ancient continental margins. The western edge is parallel to and lies west of the eastern edge of the North American continent of Cambro-Ordovician time as defined by Rodgers[179].

The Appalachian Mobile Belt is underlain by both sialic crust of Grenvillian age and by thick, dense, and presumably mafic crust[180]. Additionally, metasedimentary and metavolcanic rocks of Avalonian age (approximately 600 million yr ago) occur in this province in eastern New England. In the southern and western portions of the Mobile Belt, the Avalonian rocks, with mafic intrusives, unconformably overlie Grenvillian basement as an eastward-thickening sequence. The origin of the Avalonian rocks is believed related to the rifting that led to the opening of the proto-Atlantic Ocean[29].

Unconformably above these two sequences of basement rocks are various metamorphosed early- and late-Paleozoic marine sedimentary rocks (mio- and eugeosynclinal) which were deformed during both the Medial Ordovician Taconic Orogeny (450 million yr ago) and the Devonian Acadian Orogeny (300 million yr ago).

Continental clastic and volcanic rocks of late-Paleozoic Age (late-Carboniferous to Permian) were deposited in southern and central-eastern New England within rift basins along the ancient western margin of the Avalon Platform during the translational stage of Paleozoic continental convergence[166].

Lastly, the Mobile Belt contains continental clastic, volcanic, and intrusive mafic rocks in rift basins formed in Triassic to Jurassic time and associated with opening of the Atlantic Ocean[166]. Concomitant with the formation of these basins in southern New England, New York, New Jersey, Pennsylvania, and Maryland, alkaline intrusive rocks invaded the crust in northern New England and Quebec Province, Canada, known as the White Mountain and Monteregian Hills intrusive complexes.

The Cretaceous and Cenozoic clastic wedge of the Atlantic Coastal Plain covers a major portion of this province south of New York City. The majority of these sediments were deposited as a result of episodes of epeirogenic uplift and erosion of the eastern craton and subsidence with deposition as its margin.

The foregoing summary indicates that this tectonic province has undergone major changes in regional stress conditions since late-Precambrian time. The changes have been cyclic, ranging from episodes of crustal extension to major compression. Since the Cretaceous Period, strain developed in the crust has been attributed to motion related to lateral spreading from the Mid-Atlantic Ridge[120] and to vertical crustal uplift and subsidence[94]. Zoback and Zoback[121] (Figure 2.5-79) have interpreted that the Atlantic Coast region, Coastal Plain, and Piedmont represent a contemporary stress province in which the maximum horizontal stress and least horizontal stress are perpendicular and parallel to the coastline, respectively. The western boundary of this stress province is approximately coincident with the peak of the steep Appalachian gravity gradient[40] which separates the highly metamorphosed, plutonized, and upthrust rocks of the core of the orogen on the east (Mobile Belt) from the miogeosynclinal rocks to the west (Fold Belt).

The largest earthquake recorded in the Appalachian Mobile Belt was the 1886 MMI X event near Charleston, SC. This event is not considered relevant to the Unit 2 site, because the earthquake is considered to be associated with structure in the Charleston, SC, area[181]. The MMI VIII Boston-Cape Ann event of 1755 occurred nearer to the site. Ballard and Uchupi[182] suggest that this event is restricted to the northwestern boundary of the Avalon Platform near Boston. Sbar and Sykes[120] associate this event with the Boston-Ottawa seismic trend which includes the Ottawa-Bonnechere Graben area. Delineation of the Ottawa Basin subprovince of the Central Stable Region is consistent with this seismic trend (Figure 2.5-12). The closest approach of the boundary of the Appalachian Mobile Belt Province is 260 km (162 mi) from the site.

Additionally, a number of MMI VII events have occurred in historic and recent time in the Mobile Belt. Some have occurred within the Connecticut Basin, some within the central Piedmont, and some along the Fall Line boundary of the Piedmont and the Atlantic Coastal Plain (Section 2.5.2.1). These events would approach no closer than 260 km (162 mi) to the site.

2.5.2.2.5 Alternative Tectonic Province Models

The distribution of recent and historic seismicity of the eastern United States does not correlate strongly with the occurrence of known or inferred geological structures. Therefore, a "province" approach to select conservative seismic design values for a nuclear facility such as Unit 2 is required. Such an approach is deterministic and all available geological and geophysical data and understanding, to the extent possible, are required as a basis for delineating restrictive boundaries of tectonic or seismic provinces. These boundaries theoretically define the nature and quantity of seismic energy which is released within a given volume of crust.

Dames & Moore[(166)] defined tectonic provinces for the Indian Point nuclear facility in the lower Hudson River region of New York State. This work represents a strict application of the regulation (10CFR100, Appendix A); that is, the structural and tectonic evolution of the Appalachian Orogen is used as the basis for delineating the tectonic provinces. The Stable Interior province of that study corresponds to the site tectonic province (Eastern Stable Platform subprovince) and is bounded on the north by the Ottawa Basin tectonic province[(166)].

Later, the same tectonic provinces as those for the Indian Point facility were also used for the Pilgrim nuclear facility near Boston, MA[(183)]. It was argued that the epicenter of the 1755 Cape Ann earthquake (MMI VIII) is restricted to one of the boundaries of the tectonic province within which the facility was sited, rather than translate that event to the Pilgrim site for consideration as the SSE.

The tectonic provinces selected for the New Haven site[(81)] represented a composite of tectonic and seismotectonic provinces. The two principal differences between the NYSEG model and the one used here are: 1) isolation of the Adirondack Mountains from the Central Stable Region as a separate tectonic province, and 2) identification of the Western Quebec Seismic Zone, a linear northwest-trending zone whose southern boundary is roughly parallel to the Ottawa River, and at its closest point is approximately 161 km (100 mi) northeast of Nine Mile Point. The Western Quebec Seismic Zone corresponds closely to the Boston-Ottawa seismic trend and to the Ottawa Basin subprovince as defined in this report.

With respect to the Nine Mile Point site, none of these other deterministic models differs greatly from nor is, in our opinion,

more conservative than that of the modified version of Hadley and Devine[28]; therefore, the model chosen is believed to be conservative in the selection of design accelerations for the Unit 2 facility.

2.5.2.3 Correlation of Earthquake Activity with Geologic Structures or Tectonic Provinces

2.5.2.3.1 Introduction

Although the record of historic seismicity in the eastern United States is of several hundred years duration, it has been only in the last several decades that systematic evaluation of the association of seismicity to geologic structure has been attempted. On the basis of the geological, seismological, and other geophysical data available, only a limited number of associations of earthquakes to tectonic elements can be made. This section discusses those correlations between earthquakes and geologic structures or tectonic provinces of importance to the Nine Mile Point site.

2.5.2.3.2 Correlations with Structures

The most important earthquake activity in the area can be related to known fault structures and other documented geologic features previously discussed in Sections 2.5.2.2 and 2.5.1.1. Although no earthquake larger than magnitude (duration) M_c = 3.0 has originated in the site vicinity in historic time, there are two localized areas of earthquake activity within 322 km (200 mi) of the site. These are the area near Massena, NY, about 177 km (110 mi) northeast of the site, and the general area of Attica, NY, about 145 km (90 mi) west-southwest of the site.

Massena Area

The 1944 MMI VIII shock in the Massena-Cornwall area (Section 2.5.2.1) is thought to be related to the Gloucester fault[81], a feature (within the Ottawa Basin Sub-Province) that extends northwestward from near Massena, NY, and lies subparallel to the Ottawa-Bonnechere Graben (Section 2.5.1.1). Two recent earthquakes in July of 1981 near Cornwall, Ontario, for which focal mechanisms were computed[184], corroborate this probable relationship. Both mechanisms (Figure 2.5-20) show approximately northwest striking nodal planes with a combination of thrusting and strike-slip motion. Forsyth[8,185] has suggested a relationship between seismic events occurring near Bouguer gravity anomalies in the Central Metasedimentary Belt (Figure 2.5-3), and near the intersection of rift structures in the northern Ottawa-Bonnechere Graben and the structures along the St. Lawrence River. Forsyth also suggests that, whereas the epicenters delineate a northwesterly trend (Figure 2.5-12), closer inspection shows the major activity is associated with northeast-trending geological structures. Such a relationship is not born out, however, with the northwest-trending nodal planes

of the two recent Cornwall, Ontario, events, or the focal mechanism solutions of the July 12, 1975, Maniwaki, Quebec, event (Figure 2.5-20)[186]. Examination of Figure 2.5-20 shows that there is a distinctive difference in focal mechanism solutions of earthquakes to the north and south of the Frontenac Axis (Figure2.5-4). To the north, the solutions display northwesterly-trending nodal planes. To the south, however, the solutions indicate either northerly- or north-northeasterly-trending nodal planes (i.e., Attica or Blue Mt. Lake)[187,188].

There are several possible explanations for this difference. One of these, as Barosh[189] has suggested, is the influence of vertical crustal movements on the occurrence of earthquakes in the eastern United States. In a previous report[94] it was shown that there is considerable variation in the rates of vertical movements of the crust in the northeastern United States. The velocity of these movements varies from one point to another and does not remain constant with time. Certain areas are undergoing more rapid uplift (e.g., Adirondack Massif) than others are (e.g., the site area). Differential movements such as these must cause a reorientation of the regional stress field by inducing shear strains in planes tangential and normal to the earth's surface.

Sanford[190] studied the two-dimensional system of stresses likely to be caused in a lithospheric block by small vertical dislocations, assuming that the initial horizontal stresses at the surface of the block were equal to zero. He found that a vertical step-like dislocation at the base of the lithospheric block causes a differentiation of the stress field across the lithospheric block. In the portion subjected to uplift, the horizontal surface stress is tensile, whereas in the subsiding portion, the horizontal near-surface stress is compressional. The stress trajectories within the boundary zone or transition zone separating these two portions are inclined, indicating high shear stress in a vertical plane. Depending upon the wavelength of uplift or subsidence caused by the applied dislocation, regions of the block with a high density of distortional strain energy vary from the crest of maximum uplift and trough of maximum subsidence (for long wavelength distortions of the crust) to the transition zone between maximum uplift and subsidence (for short wavelength distortions of the crust).

As shown on Figure 2.5-21[94], the trend of the velocity isobases is northwest-southeast, and is related to the pattern of crustal tilting caused by glacial unloading. Locally, relatively high rates of uplift of the Adirondack Massif perturb this general trend. A comparison of Figure 2.5-20 with the velocity isobases shown on Figure 2.5-21 suggests that:

1. The relatively rapid vertical uplift of the Adirondack Massif and the Frontenac Axis may be inducing a short

wavelength type of distortion of the crust especially north and east of the axis of uplift. The frequency of seismic events is much greater in this area than to the south of this axis.

2. The zero velocity isobase occurs generally near the Frontenac Axis and has a northwest-southeast trend. This represents a relative zone of transition between areas of maximum uplift and subsidence.

3. Focal mechanism solutions (Figure 2.5-20) from events north and east of the Frontenac Axis indicate that the intermediate principal stress is subparallel to the isobases. However, the principal stress trajectories are inclined, and the greatest shear strain occurs in planes that are steeply inclined and subparallel to the front of crustal undulation.

4. Geological evidence of northwest-striking zones of weakness in the basement (parallel to the isobases) consists of northwest-striking Precambrian dike swarms near the Frontenac Axis (Figure 2.5-4) and structures of the Ottawa-Bonnechere system (Figures 2.5-4 and 2.5-20). The focal mechanism solutions indicate that the stress field north of the Frontenac Axis is more favorably oriented for slip along these features than along northeast-striking features.

5. The lack of significant seismicity to the southwest of the Frontenac Axis-Adirondack Massif may be a result of:

   a. A relatively lower gradient of vertical crustal movements and a return to the dominant stress field extant during the interglacial state, that is, a stress field that is less influenced by glaciostatic rebound than in the area to the north.

   b. Different patterns of vertical movements so that the isobases have different trends (perhaps of neotectonic origin).

   c. A combination of a and b.

Another possible explanation for the differences in focal mechanism solutions north and south of the Frontenac Axis is offered by Diment et al[40]. They consider that several geophysical (gravity) anomalies, which trend northwesterly and are generally shown on Figure 2.5-12, have a correlation with earthquakes that occur in northern New York State and adjacent parts of Canada. As a basis for this correlation, Diment et al cite the work by Sbar and Sykes[120] that earthquakes in northern New York State and adjacent Canada occur along planes of weakness

oriented north-northwest to northwest, even though the more obvious structural trends are northeasterly in this region.

Attica Area

The moderate historic seismic activity in western New York State near Attica is related to the Clarendon-Linden structure (Section 2.5.1.1), a prominent geologic feature trending roughly north-south, and associated with the Eastern Stable Platform Sub-Province (Section 2.5.2.2). A permanent array of seismic stations, operated by the Lamont-Doherty Geological Observatory of Columbia University, has been monitoring the earthquake activity in the Attica area since early 1971, and is designed to accurately locate events in the Buffalo-Attica region. A number of small (magnitudes generally less than 3.0) naturally occurring events have been reported[169,191]. Also, a swarm of microseismic events occurred during the summer and fall of 1971. The swarm was related in time and space to the injection of water under high pressure in a brine well[169]. The cluster of events was elongated along the strike of the Clarendon-Linden structure, which strongly suggests that the fluid injection triggered the release of tectonic strain energy accumulated on the fault. Much of the natural and induced activity occurs within the triangular fault-bounded block formed by the main Clarendon-Linden structure and its bifurcated limbs[69] in the Attica-Dale area.

Focal mechanism solutions of two Attica earthquakes that occurred in 1966 (m_b = 4.6) and 1967 (m_b = 4.4)[122] and a composite focal mechanism solution of earthquake swarms near Attica[169], indicate a combination of right lateral strike-slip and reverse faulting on a plane parallel to the northerly trend of the Clarendon-Linden system (Figure 2.5-20). Focal depths of the 1966 and 1967 Attica events are computed by Herrmann to occur in the upper 2 to 3 km (1.2 to 1.9 mi) of the crust. The events analyzed by Fletcher and Sykes, which were thought to be related to hydraulic mining of salt deposits at a depth of 500 m (1,640 ft), occur at depths less than 1 km (0.62 mi).

A cursory examination of Figure 2.5-12 indicates a possible alignment of epicenters in an east-west trend. An examination of aerial photographs and subsurface contour maps based on deep well data[52,192,193] discloses no lineations that might suggest the existence of such features. Based on the Applicant's review of existing data, there is no geologic or seismologic basis for projecting the Attica event eastward in closer proximity to the site than the closest approach of the Clarendon-Linden Structure, which is approximately 97 km (60 mi)[70].

Detailed studies by Dames & Moore[167] indicate that the Anna, OH, events of MMI VII-VIII are associated with the Anna (OH) Seismogenic Zone, which is located at the intersection of three reentrants of adjoining Paleozoic basins. To assume that events similar to the March 8, 1937, MMI VII-VIII earthquake could occur closer to the site than the Anna Seismogenic Zone is not warranted (Section 2.5.2.2.2).

Occasional small shocks in the region cannot be related to known geologic structure. Typical of the largest of these are earthquakes in the Buffalo, NY, Hamilton, Ontario, and Lowville, NY, areas (Figure 2.5-12). Even smaller events (MMI IV or less) have been reported near Syracuse, NY, and 48 km (30 mi) west of the site beneath Lake Ontario. The actual cause of these events is not known. They represent examples of numerous earthquakes of minor intensity that occur throughout the eastern United States that are seemingly random.

Since mid-1981, a seismic monitoring network has been operational in the north central region of New York State. Consisting of 12 stations (two of which are three component stations), the network is one of two in New York State sponsored by the Empire State Electrical and Energy Research Corporation (ESEERCO). Several minor earthquakes have occurred and have been located within 80 km (50 mi) of the site. On September 7, 1981, a small event occurred near Lafayette, NY, some 65 km (40 mi) southeast of the site, having a coda magnitude (M_c) of 1.6. The focal mechanism constructed for this event indicates normal-oblique slip faulting along a northwest- or northeast-striking plane[194] (Figure 2.5-20). On September 16, 1981, an event of M_c = 2.7 occurred some 12 km (7.5 mi) south of the site, between Fulton and Oswego, NY. The event was apparently not felt, based on a survey of local police departments. The focal mechanism solution for this event (Figure 2.5-20) indicates normal-oblique slip faulting along a northeast- or northwest-striking, steeply dipping plane. Presently there is no correlation between the Fulton-Oswego event and known structures within the epicentral region[194]. Two small events (M_c = 1.6 and M_c = 1.5) also were recorded in Lake Ontario about 21 km (13 mi) north of the Unit 2 site on November 3, 1981. Because they were located outside the network and were small in size, no focal mechanisms could be constructed.

The Demster Structural Zone (Section 2.5.1.2), also within the Eastern Stable Platform Sub-Province, has not been associated with any historic seismic events.

2.5.2.3.3 Correlations with Tectonic Provinces

Apart from those earthquakes correlative with geologic structure discussed in the previous section, there remain earthquakes that occur within tectonic provinces or subprovinces near and adjoining the site, and that do not have any obvious or suggestive association with geologic structures. The tectonic provinces (including that in which the site is located) within 320 km (200 mi) of the site and the largest historic earthquake(s) associated with each are:

1. Central Stable Region - Eastern Stable Platform Sub-Province (site province) - MMI VI.

2. Ottawa Basin Sub-Province - MMI IX.

3. Northern Appalachian Fold Belt - MMI VI.

4. Appalachian Mobile Belt - MMI VII.

Section 2.5.2.4 contains a discussion of the effect of each maximum earthquake associated with either geologic structure or tectonic provinces at the site.

2.5.2.4 Maximum Earthquake Potential

As discussed in the previous section, there are two structurally restrictive areas of repeated earthquake activity that are significant to the seismic assessment of the Unit 2 site. These are: 1) the concentration of activity near Massena, NY, which generated the maximum event of MMI VIII in 1944, and 2) the seismicity associated with the Clarendon-Linden fault near Attica, NY, the largest of which was the MMI VII shock of 1929; justification for downgrading the maximum intensity of this event from MMI VIII, as originally reported, to MMI VII is provided in the response to NRC Question F230.5 in Appendix 2N. The only additional consideration for seismic design is the random activity in the site region which cannot be associated with specific geologic structures. Other seismicity from any province, structure, or zone is not of concern for vibratory ground motion at the site.

To ascertain the ground motion to be expected at the site on the basis of the postulated recurrences of the types of earthquake activity discussed above, the attenuation characteristics of the site region have been reviewed and are illustrated on Figure 2.5-80. In this figure, the attenuation function calculated for the eastern province of the U.S.[195] is used to derive ground motion intensity at the site from a recurrence of the design events. For purposes of comparison, the curves for the Attica and Massena events (1929 and 1944, respectively) have been derived by averaging the isoseismal radii using the isoseismal maps of Figures 2.5-81 and 2.5-82.

A recurrence of the Massena MMI VIII shock would not occur closer to the site than the boundary of the Ottawa Basin Sub-Province which is coincident with the southwesternmost extent of its associated faulting, a distance of 175 km (109 mi). Such a recurrence would impose a MMI V at the site, a level corroborated by the actual experience of felt intensities shown on Figure 2.5-81. Similarly, a MMI IX event, equivalent to the largest of the St. Lawrence Valley earthquakes, would be attenuated to slightly more than MMI VI if the shock were migrated to the closest point of the Ottawa Basin Sub-Province to the site (175 km, 109 mi).

Using this same attenuation relationship, a recurrence of an event similar to the Attica MMI VII event would be felt onsite as MMI V if translated along the associated structure to its closest

approach of 100 km (67 mi). The interpretation of feltintensities shown on Figure 2.5-82 suggests a site intensity of MMI IV. Nevertheless, the site vibratory ground motion and its impact on seismic design have been evaluated assuming MMI VIII at the epicenter and are presented in the response to NRC Question F230.5 in Appendix 2N.

Within the site tectonic province, the historical occurrences of shocks of MMI VI or less (similar to the Lowville, NY, or Hamilton, Ontario, events) have not been reliably associated with known seismogenic structure. Therefore, the maximum earthquake potential at the site must be represented by the random occurrence of a MMI VI shock adjacent to the site, a level of ground motion which is larger than other cases previously discussed.

2.5.2.5 Seismic Wave Transmission Characteristics of the Site

Properties of the material underlying the site have been determined and are described in Section 2.5.4. The competent rocks of the site foundations show no physical properties or geometric characteristics that would contribute to amplification, wave-guide effects, or other adverse alteration of impinging seismic energy. Therefore, the wave transmission characteristics of the materials underlying the Unit 2 site are adequately taken into consideration in the general attenuation and acceleration/intensity correlations used to identify the maximum earthquake and the SSE in Sections 2.5.2.4 and 2.5.2.6.

2.5.2.6 Safe Shutdown Earthquake

The maximum potential earthquake was described in Section 2.5.2.4 as a random MMI VI event adjacent to the Unit 2 site. Correlations between MMI and peak horizontal acceleration have been presented by many authors. The conservative relationship developed by Trifunac and Brady[165] was used to quantify the ground motion level for the SSE.

This relationship ($\log A_{(H)} = 0.3\ I + 0.014$) calculates a peak horizontal acceleration value of 0.07 g. However, to be very conservative, horizontal and vertical acceleration levels of 0.15 g have been adopted in the design response spectra shown on Figures 2.5-83 and 2.5-84. The design spectra with a zero period value of 0.15 g should effectively envelop structural response resulting from the occurrence of the design event near the site or response from long-duration, low-amplitude motion generated by large, distant events such as those in the St. Lawrence Valley.

A further discussion regarding the adequacy of the SSE design response spectrum at Unit 2 is provided in the response to NRC Question F230.3 in Appendix 2N.

2.5.2.7 Operating Basis Earthquake

In accordance with Appendix A to 10CFR100, the OBE is herein defined as an earthquake that would produce a horizontal acceleration at the site of 0.075 g. This level is conservative in that it is one-half the SSE design level, and exceeds the ground motion expected from the deterministic assessment of the maximum potential earthquake (worst case) discussed in the preceding section. Therefore, the 0.075 g level for the OBE is adequately conservative and is used in the response spectra shown on Figures 2.5-85 and 2.5-86. For additional information and justification regarding the probability of exceeding the OBE, refer to the response to NRC Question F230.2 in Appendix 2N.

2.5.3 Surface Faulting

The potential for surface faulting has been given careful consideration at the Unit 2 site. Faults of small displacement are present on the site proper, and a large structural zone, known as the Demster Structural Zone, occurs within 8 km (5 mi) of the Unit 2 site (Figure 2.5-25). Investigations of the origin and history of movement of these features have been performed within the context of Appendix A to 10CFR100. The detailed results of the investigations of the Cooling Tower fault and Radwaste thrust at the Unit 2 site were presented in previous reports[154] and summarized in Section 2.5.1.2. Formal responses to requests from the NRC for additional information (Questions 361.1 through 361.35) regarding the 1978 NMPC report were submitted in 1979 and 1980. It was concluded in each case that the potential for surface faulting at the site proper is negligible (Section 2.5.1.2.3). Investigations of a proposed nuclear power plant site at New Haven, NY, 11.2 km (7 mi) east of the Unit 2 site, disclosed the existence of broad, low-amplitude folds and associated faults (Section 2.5.1.2.4)[81]. Further detailed investigation of the Demster Structural Zone has led to the conclusion that the faults within the zone are not capable, and that no potential for surface faulting is associated with these structures.

2.5.3.1 Geologic Conditions of the Site

The lithologic, stratigraphic, and structural conditions at the site are discussed in Section 2.5.1.2. The relationships of the site and regional geologic conditions to site safety have been addressed in detail in previous reports[94] (responses to NRC questions) and are summarized in Sections 2.5.1.2 and 2.5.1.1, respectively.

2.5.3.2 Evidence of Fault Offset

Detailed documentation of the investigations of observed fault offsets near the site is contained in previously documented reports[94]. This information is also summarized in Section 2.5.1.2.

2.5.3.3 Earthquakes Associated with Capable Faults

The discussion of the historical seismicity in the region around the Unit 2 site is contained in Section 2.5.2.1. None of these earthquakes have been associated with faults within 8 km (5 mi) of the site (Section 2.5.2.3).

2.5.3.4 Investigation of Capable Faults

Faults at the site are not capable within the intent of Appendix A to 10CFR100[94] (responses to NRC questions). Also, investigations regarding the Demster Structural Zone[81] concluded that this feature is not a capable fault.

2.5.3.5 Correlation of Epicenters with Capable Faults

Structural and genetic relationships between site area faulting and the regional tectonic framework are discussed in Section 2.5.1.1.3. The relationship between regional geologic structures and earthquake epicenters is described in Section 2.5.2.

2.5.3.6 Description of Capable Faults

There are no known capable faults within 8 km (5 mi) of the site (Section 2.5.3.4).

2.5.3.7 Zone Requiring Detailed Faulting Investigation

There are no known capable faults within 8 km (5 mi) of the site (Section 2.5.3.4).

2.5.3.8 Results of Faulting Investigation

There are no known capable faults within 8 km (5 mi) of the site (Section 2.5.3.4).

2.5.4 Stability of Subsurface Materials and Foundations

This section defines the subsurface conditions and engineering properties of the bedrock supporting the nuclear power plant foundations. No safety-related structures are founded on natural soils. The main stack and underground exhaust tunnels are major Category I structures which are founded on sound Oswego sandstone, as discussed in Section 2.5.4.10. Site geologic conditions are described in detail in Section 2.5.1.2 and summarized in Section 2.5.4.1. A detailed summary of laboratory testing procedures and results for determination of rock properties of the site bedrock are presented in Section 2.5.4.2. Section 2.5.4.3 provides information pertaining to geologic exploration and investigations at the site. Geophysical surveys to establish the in situ dynamic properties of the rock beneath the site are discussed in Section 2.5.4.4. Excavations and backfill information are discussed in Section 2.5.4.5. Detailed information is provided on the extent (horizontal and vertical) of all Category I excavations, fills, and slopes; the dewatering

and excavation methods used; and the sources and quantities of backfill and borrow. Analysis of groundwater at the site is discussed in Section 2.5.4.6. Dynamic and static loading conditions for rock at the site are discussed in Sections 2.5.4.7 and 2.5.4.8. Earthquake design basis information containing a summary of the derivation of the OBE and SSE is provided in Section 2.5.4.9.

2.5.4.1 Geologic Features

Basic geologic and seismic information is presented in Section 2.5.1. Conditions at the site associated with subsidence, uplift, and collapse are discussed in Section 2.5.4.1.1. Loading history (Section 2.5.4.1.2) pertains only to bedrock at the Unit 2 site because no safety-related structures are founded on natural soils. The geologic history of the site with respect to erosion and deposition, groundwater levels, and glacial loading and unloading, is discussed in Section 2.5.4.1.2. Characteristics associated with the bedrock, such as weathering, zones of alteration, jointing pattern, and zones of structural weakness, are presented in Section 2.5.4.1.3. More detailed discussion regarding the bedrock fractures is provided in Section 2.5.1.2.3.

2.5.4.1.1 Collapse, Subsidence, and Uplift

No large-scale features indicative of collapse or subsidence occur in the vicinity of Nine Mile Point. The bedrock underlying the site is not subject to solution. The only uplift occurring in the area is of regional extent (Section 2.5.1.1) and, as such, is not liable to have a direct impact on the stability of the site foundations. Open dilations were encountered within the rock mass at a number of locations in the vicinity of the thrust structures and high-angle faults discussed in Section 2.5.1.2.3. These openings are up to several inches wide. The possibility of settlement of the engineering structures related to the closure of bedrock dilations under design foundation loads has been evaluated and accounted for in the design.

2.5.4.1.2 Loading History

No safety-related structures are founded on natural soils; therefore, the loading history of the overburden materials is not pertinent to this discussion.

A complete geologic history of the site is presented in Section 2.5.1.2.6[94]. This section provides a synopsis of the bedrock loading history, focusing on its impact on the stability of the foundation materials.

The site strata were deposited in Late Ordovician time (450 M.Y.). Continual regional subsidence until late in the Paleozoic era (250 to 300 M.Y.) resulted in the burial of these units by 2,438 to 3,048 m (8,000 to 10,000 ft) of younger sediments.

Throughout this period of subsidence, these sediments were subjected to diagenesis resulting in their induration and lithification. In Late Paleozoic time, the more brittle siliceous strata were deformed by strike-slip faulting (Section 2.5.1.2.6). The process of regional uplift began in Late-Paleozoic time (200 M.Y.) and continues today. Late in the Cretaceous period, uplift resulted in the development of normal faults along the discontinuities resulting from the earlier deformation. Younger stratigraphic units were eroded from the site in association with regional uplift. By Late-Tertiary time much of the thick cover of sediments overlying the site was no longer present. During the Pleistocene Epoch, a series of glaciations dominated the active erosional and depositional processes in the site area.

Four stages of glaciation have been identified in North America (Section 2.5.1.1). The last stage (Wisconsinan) removed any evidence of prior glaciation in the site area. Therefore, it is not known how the first three stages of glaciation affected the site. It is certain that each glaciation resulted in a depression of the crust as an isostatic response to the load imposed by the ice. After the removal of the ice sheet during interglacial stages, the crust responded, at least partially, in an opposite manner by glacio-isostatic rebound. The thickness of the Wisconsinan ice sheet at the site is estimated to have been about 1,980 m (6,500 ft), approximately 18,000 yr B.P. The depression of the crust from this load is estimated to have been between 457 to 549 m (1,500 to 1,800 ft)[94].

Downward flexing of the strata from glacial loading contributed to the deformation of the site rock mass as demonstrated by:

1. Breccia zones along bedding (Section 2.5.1.2.3).

2. Buckling associated with the north-dipping, high-angle faults (Sections 2.5.1.2.3 and 2.5.1.2.6).

3. Shallow thrust structures (Sections 2.5.1.2.3 and 2.5.1.2.6).

During Wisconsinan deglaciation, the site area was occupied by proglacial Lake Iroquois. It is estimated that the depth of water at the site was approximately 90 m (300 ft). From studies of the history of Lake Iroquois[94] it is known that 11,000 yr B.P., the lake level decreased to a level below the present Lake Ontario level in only a few hundred years (Figure 2.5-24). A tendency for increasing uplift pressure might have developed, because the bedrock surface was covered at that time by a veneer of impermeable glacial sediments which prevented the fluid pressure in the rock mass from dissipating as the lake level was decreasing. This condition would have contributed to additional bedding-plane-slip where stress gradients were steep, such as at the Cooling Tower fault (Section 2.5.1.2.3).

2.5.4.1.3 Weathering, Alteration, and Bedrock Structure

Weathering and Alteration

Any major zones of surficial weathering that may have been present at the site were removed by erosion during the Wisconsinan glaciation. The bedrock surface is very clean and unweathered. Minor oxidation of near surface joints is present in some locations. There are no zones of alteration or irregular weathering within the rock mass. On exposed rock surfaces, the strata composed of largely argillaceous material tend to disintegrate (Section 2.5.4.2). However, in newly excavated exposures these strata are unweathered.

Bedrock Structures

Except where faults are present, the bedrock at Nine Mile Point is only slightly to moderately fractured. A set of systematic fractures in the bedrock, which are consistent with regional trends, is discussed in Section 2.5.1.2.3. These conjugate shear fractures commonly contain calcite and sulfide minerals, and are occasionally healed by this mineralization. The systematic fractures are most prevalent in the relatively brittle siliceous beds within the stratigraphic section. Mineralization studies[94] suggest that these fractures are older than Late-Cretaceous age.

Fracture density increases markedly in the vicinity of the faults at the site (Figure 2.5-26). The nature of the fracturing at these locations is summarized in Section 2.5.1.2.3 and discussed in detail in prior reports [94,154]. These fractures resulted from the development of the faults and associated folds. Some of these fractures are mineralized; others are either open or filled with unconsolidated sediments depending on their age of development.

At the site, fractures along bedding planes are common indicators that slip has occurred (Section 2.5.1.2.3). They are sometimes filled with thin breccia and are frequently slickensided as a result of bedding-plane-slip. In the vicinity of the thrust structures these subhorizontal fractures were often dilated and frequently filled with unconsolidated silt and clay, presumably of lacustrine origin. Bedding plane fractures filled with laminated silt and clay were detected within the rock mass to depths as great as 82 m (270 ft)[154].

Several high-angle faults and a series of thrust faults (Figure 2.5-26) are present at the site and are described in Section 2.5.1.2.3. Information pertaining to the discussion of these deformational features is detailed in Section 2.5.1.2.3. Northeast-trending broad, low-amplitude folds, inferred to be of more regional extent, are discussed in Section 2.5.1.2.4.

For additional information regarding the bedding dip angle, as well as the ranges and means of core recovery and RQD, refer to the response to NRC Question F241.2 in Appendix 2N.

2.5.4.1.4 Unrelieved Residual Stresses

Regional Stress Conditions

A review of stress determinations in the Lake Ontario region was undertaken in 1978[94] to assist in evaluating the stress conditions at Nine Mile Point. Five groups of data were considered with the intention of identifying regional characteristics of rock stress. Of these five groups, only the results of overcoring and hydrofracturing are field measurements of in situ stress, and even these measurements are based on simple models that result in approximations of the in situ stress. Additionally, analyses of earthquake seismic records, surface strain-relief observations and trends of postglacial buckles were compiled and assessed. These measurements and indicators of stress provide a profile of stress conditions from the surface and near surface (surface strain relief, postglacial buckles, and overcoring) through intermediate depths (hydrofracturing) to depths of up to 15,240 m (50,000 ft) (earthquake focal mechanisms).

The available hydrofracture test data in western New York State are presented in Volume III of Reference 94. Maximum horizontal stresses of 105 to 160 kg/sq cm (1,495 to 2,275 psi) are reported from tests ranging from 152 to 528 m (500 to 1,700 ft) below the ground surface. The maximum principal stresses, as determined by hydrofracturing, generally trend east-northeast. Additionally, hydrofracturing tests were completed by the United States Geological Survey (USGS) in a well near Auburn, NY, in the spring of 1982. These tests, conducted as part of a NYSERDA and ESEERCO-sponsored geothermal project, were performed in the Paleozoic cover rock. Results of these tests show that the maximum horizontal stress is oriented approximately N85°E with a magnitude that varies from 141 $kg/cm^2 \pm 10$ kg/cm^2 at 593 m to 499 $kg/cm^2 \pm 10$ kg/cm^2 at 1482 m depth. These results agree in orientation and relative magnitude with other hydrofracturing results in western New York State.

Table 2.5-3 is an updated summary of near-surface overcore test sites and results in the region. An outstanding characteristic of the data is the variability of both the maximum and minimum horizontal stress magnitudes, stress differences, and stress orientations. The reported magnitude of maximum horizontal stress at depths of less than 26 m (85 ft) varies from -49 to 302 kg/sq cm (-700 to 4,300 psi) and averages approximately 84 kg/sq cm (1,200 psi). Despite the variability of stress magnitude and orientation, the regional data indicate a horizontal stress that is higher than the value that could be attributed to a simple gravitational loading by the present overburden.

Focal mechanism solutions of earthquakes that have occurred in the region are discussed in Section 2.5.2.3 and illustrated on Figure 2.5-20. Although indications of stresses at depth provided by earthquake focal plane analysis are limited in number, the results of these analyses support the conclusion of the presence of high horizontal stress. They indicate that the inferred maximum principal stress is subhorizontal and predominantly associated with thrust faulting. Furthermore, the average orientation of the inferred maximum principal stress is east-northeast to northeast, which corresponds well with the average orientation determined by hydrofracture stress measurements.

Surface measurements of in situ strain are discussed in Volume III of Reference 94. Plate 2-4 and Table 2-4 of the referenced report present the results of surface measurements of strain, and although these results are indicators of regional stress, it is expected that they are strongly influenced by near-surface effects.

Postglacial deformational features in the region are discussed in Section 2.5.1.1.3 and illustrated on Figure 2.5-17. The rose diagram of postglacial folds on Figure 2.5-17 shows a roughly northwest orientation of the axial trend of these folds. This suggests that the contemporary maximum horizontal stresses responsible for the formation of the folds are oriented northeast (perpendicular to the axial trends of the folds). The development of these postglacial buckles is a consequence of high horizontal stress. However, the maximum stress orientation inferred from these structures is considered to provide only an approximation of horizontal stress direction.

In conclusion, a review of the regional stress conditions indicates that the area is characterized by high horizontal rock stresses. The maximum measured horizontal stresses range from -49 to 302 kg/sq cm (-700 to 4,300 psi), and within the upper 60 m (200 ft) of bedrock, horizontal stresses can be expected to reach 70 kg/sq cm (1,000 psi) in the region.

Focal mechanism solutions and the results of hydrofracture testing indicate that the orientation of the maximum horizontal stress is east-northeast. Surface strain measurements and the general orientation of postglacial folds tend to confirm this orientation; however, overcore measurements reveal a widespread variation of the orientation of the maximum horizontal stress.

Evolution of Regional Stresses

The evolution of regional stresses has been detailed as Appendix III-L of the 1978 Geologic Investigation[94]. The principal features of this interpretation are that the relatively high stresses in the region are remnant gravitational and tectonic stresses resulting from burial in the Appalachian basin and subsequent uplift (Section 2.5.1.2.6). The stress distribution

is modified locally by geologic structures such as the Drainage Ditch and Cooling Tower faults. Glacial loading introduced additional stresses into the bedrock, with the shear stresses in the plane of bedding increasing with increasing flexure of the strata during glacial loading of the crust. These shear stresses decreased with crustal rebound.

These effects (basin subsidence, normal faulting, glacial loading, etc.) are likely to be common to large areas of the northeastern part of the Appalachian basin, and similar stresses to those measured at the Nine Mile Point site exist at other locations in this sedimentary basin. Therefore, although the stresses measured at Nine Mile Point are relatively high, they are not anomalous for the region and are significantly less than the strength of the bedrock.

Site Stress Conditions

As a result of the review of regional stress conditions it became evident that the rock mass in the region is characterized by high horizontal stresses. This fact, coupled with the observations of bedrock displacements at the site[154], led to the decision to use overcoring as a method to determine the stress conditions at the site.

Stress measurements were previously made at Nine Mile Point as part of the geologic investigation of the James A. FitzPatrick Nuclear Power Plant site. However, these measurements were made near the excavation, and the low stress values recorded are not considered to be representative of the general site conditions.

During the geologic investigations at Nine Mile Point, three series of overcoring tests to determine in situ stress conditions and one series of tests to evaluate residual strains by undercoring were conducted. The locations of these test borings are shown on Figure 2.5-87.

In 1977, 105 stress measurements were made in nine different boreholes (OC-series). Testing penetrated a stratigraphic sequence (Figure 2.5-8) of approximately 60 m (200 ft) in four vertical boreholes from the ground surface and three horizontal boreholes and one vertical borehole from the base of the cooling water intake shaft. In all the overcoring tests on site, the determination of in situ stress was accomplished by overcoring the U.S. Bureau of Mines (USBM) deformation gauge, recording the borehole deformation, determining the modulus of elasticity of the test cores in the laboratory, and computing the stresses using the theory of linear elasticity. Specifics regarding the procedures used for determining the in situ stress by overcoring are presented as Appendix III-B of Reference 94. Results of the OC-series testing are presented and discussed in Section 3.0 of the referenced report.

In the winter of 1977-78 a single borehole (OC-100) was drilled at the south end of the radwaste excavation between the reactor excavation and the bedrock slot west of the 12-line wall (Figure 2.5-87). Strain-relief measurements were made to a depth of 19 m (63 ft). The purpose of these stress determinations was to provide an understanding of the stress levels and distribution in the volume of rock bounded by the rock slot and 12-line wall (Figure 2.5-88). Results of the six completed tests are presented on Table 2.5-4.

In April through July 1980, strain-relief measurements were made in four vertical boreholes to assist in the evaluation of the Radwaste thrust fault. All four borings (RS-1, RS-2, RS-3, and RS-4) were sited within the fault block bounded by the Drainage Ditch fault and the Cooling Tower fault (Figure 2.5-87). Results of the RS-series of in situ stress measurements are provided and discussed in Reference 154, previously submitted as part of this docket.

In an attempt to evaluate the potential contribution to the displacements observed during overcoring by relief of residual strains, five cores were undercored in 1977. These 5- to 6-in diameter cores were monitored for associated strain changes with electrical strain gauges. The details of the procedures and the results are presented in Chapter 4.0 of the 1978 Geologic Investigation[94].

As a result of the various stress measurements conducted, a number of conclusions have been drawn regarding the nature of in situ stress conditions at the Nine Mile Point site. Generally, measured stress magnitudes and orientations differ between measurements taken outside the fault block bounded by the Cooling Tower fault and the Drainage Ditch fault and within the fault block. Outside the fault block, the stresses measured in OC-1 are believed to be typical of regional stress conditions at these depths. The direction of maximum normal stress is approximately northeast and the average magnitude of the maximum normal stress, as determined in OC-1, is greater than 120 kg/sq cm (1,700 psi). Stress measurements taken between the Cooling Tower fault and the Drainage Ditch fault indicate that the fault block has experienced extensional relief parallel and perpendicular to the strike of the bounding faults. This deformation is expressed by changes in both orientation and magnitude of stress relative to the predeformational stresses. This is manifest in a change in stress value and orientation in the hanging wall block of the Radwaste thrust structure relative to the foot wall block. The orientation of the maximum stress in the hanging wall block is northeast and the magnitude is low, whereas in the foot wall block the orientation is east-west and the magnitude is 84 to 91 kg/sq cm (1,200 to 1,300 psi). Additionally, stress measurements within the Radwaste thrust block reveal the existence of stress gradients normal and parallel to the Cooling Tower fault. These gradients are postulated to have resulted from buckling in the vicinity of the Cooling Tower fault and fault-parallel

translation associated with westward thrusting of the Radwaste fault. In conjunction with other geologic evidence, the stress measurements indicate that the Cooling Tower fault is the southern boundary of the thrust sheet, the northern boundary is inferred to be either the Drainage Ditch fault or the outcrop of the strata in the lake, the western boundary coincides with the bedrock depression in the vicinity of the heater bay, and the eastern boundary is assumed to be coincident with the axial plane of a syncline in the vicinity of RS-1. The base of the Radwaste thrust structure, as defined by an increase in stress magnitude and a change in the orientation of the maximum stress, lies approximately coincident with the illite layer in Unit A of the Pulaski at least in the northern part of the fault block.

The magnitude of stress measured in the vicinity of the reactor excavation and the bedrock slot (OC-100) are low, not exceeding approximately 28 kg/sq cm (400 psi). The low recorded values are as expected because of the stress relief of the excavation and the close proximity of the western boundary of the Radwaste thrust structure.

The undercoring tests revealed that the strains measured by undercoring are influenced by swelling and shrinkage of the rock and that any residual strains that may be relieved during in situ measurements by overcoring do not significantly affect the assumption that the measured stresses are applied boundary tractions. The contribution, if any, of residual strain relief to the values of in situ stress determined by overcoring is believed to be less than ±7 kg/sq cm (±100 psi).

Rock Squeeze

Rock squeeze is a term that originally was used to describe the deformation of rock that caused distress, in various degrees, to structures built in rock[196]. The use of the term also has been expanded to describe any time-dependent displacements that lead to closure in rock excavations regardless of whether or not there has been structural damage. Although various causes have been attributed to these time-dependent displacements, the term “rock squeeze” does not imply a singular cause and effect relationship. Instead, the term has included long-term movements of excavation walls as a result of stress relief and the viscoelasticity of the rock, and volume changes caused by variations in available moisture (rock swell). These factors can operate separately or together and have been considered in the geotechnical studies of the Nine Mile Point site. This section summarizes the regional and site-specific rock squeeze observations and discusses the various analyses completed. The design criteria developed from the various investigations are presented in Section 2.5.4.11.

Regional Considerations Rock squeeze was recognized in the southern Ontario-western New York area as early as the beginning of this century. In 1903, engineers began to measure wall closure of the wheelpits of the Canadian Niagara and Toronto

Power Plants in Niagara Falls[197,198]. Since then, numerous cases of time-dependent rock displacements have been identified. Some of these have caused distress in engineering structures, such as pressure tunnels, bridges, and canals, while others are simply observations in open excavations or quarries. Most of the observations have been made in the area from Hamilton, Ontario, to Rochester, NY, particularly in the vicinity of Niagara Falls. No specific problems or distress in engineering structures have been attributed to rock squeeze in the Oswego, NY, area.

In the instances where regular measurements of closure were taken at close enough time intervals (such as at the Wheelpit of the Canadian Niagara Power Plant), cyclic, seasonal reversals of strain have been recorded. These seasonal variations have produced a peak-to-peak fluctuation of 0.15 to 0.30 cm (0.06 to 0.12 in) at the Canadian Niagara Wheelpit[196]. This fluctuation is superimposed upon a continuing closure with rates of deformation greatest at the beginning of the measurement period and decreasing with time. At the Wheelpit, slightly more than 2 in of closure were recorded in a period of approximately 40 yr.

A study of rock squeeze movements in western New York and southern Ontario has indicated that rock squeeze problems commonly occur in the Paleozoic sedimentary rocks (particularly shales) that are subjected to high horizontal stresses and are susceptible to swelling. Free-swell laboratory testing of some of the shales exposed in the Canadian Niagara Wheelpit indicate that the rate of horizontal free swell is approximately 0.1 percent per log cycle of time (cycle 1 is taken as 0.1 to 1 hr; cycle 2 as 1 to 10 hr, etc.).

<u>Site Considerations</u> Based on a review of rock squeeze in the region and the occurrences of possible indicators of high horizontal stresses at the site (offset drill holes and small buckle folds), it was decided to investigate the possibility of rock squeeze affecting the proposed structures at the site. Stress measurements by overcoring were made (Section 2.5.4.1.4) and free swell tests were conducted on rock cores (Section 2.5.4.2.2)[94]. Based on these tests, it was concluded that the Nine Mile Point site did possess those characteristics (high in situ stress and rock units susceptible to swelling) that elsewhere in the region have caused time-dependent displacements in excavations. Because of this it was decided to isolate Category 1 structures from the effects of rock squeeze. At that time a single wall (12-line wall) had been poured against the rock. This wall was analyzed for the effects of rock squeeze and subsequently redesigned to accommodate conservative estimates of potential rock movements.

<u>Site Analysis</u> Rock squeeze studies at Nine Mile Point were directed at analyzing the stability of the 12-line wall and defining the amount of vertical and horizontal movement that may occur around Category 1 excavations so that adequate space could be provided and piping systems designed accordingly.

12-Line Wall Analysis

Initially, because the 12-line wall had been poured directly against the bedrock, a relief slot was drilled into the bedrock approximately 9 m (30 ft) west of 12-line wall[154]. Additionally, an analysis that considered the swell behavior of the rock between the slot and 12-line wall was undertaken. Two models were analyzed: a plane strain model that encompassed the western wall of the reactor excavation, and a model that analyzed the specific area around the 12-line wall in both plane strain and plane stress conditions. Both were finite element models. The behavior of the rock was modeled as linear, elastic, and transversely-anisotropic. Bedding planes were modeled by nonlinear spring elements that allowed relative movement between two adjacent points, and swell properties were defined by free swell and oedometer ring swell tests specifically conducted for this analysis (Section 2.5.4.2.2).

An incremental strain approach in a finite element model was utilized to account for the geometric and material nonlinearity which resulted from the actual construction sequencing, bedding plane slippage, and time/stress dependency of the swell strain.

The in situ stress field was simulated by an equivalent anisotropic thermal expansion and gravity "turn-on" in the initial elastic increment. For the 12-line wall analysis, a uniform stress pattern was assumed throughout the depth of the reactor excavation.

The time-dependent behavior of the site rock caused by swelling was also simulated. Laboratory test results on rock cores indicate that the swell/shrinkage phenomenon is primarily a result of hydration (Section 2.5.4.2.2). Results of free swell and oedometer ring swell tests indicate creep deformation as well. The laboratory tests indicate that the time-dependent behavior of the finer-grained rocks is responsive to the availability of water at the site and the release of in situ stresses by excavation. Depending on the stress state used for each increment in the model, the swell strain for the next increment was calculated and used as input to determine the time-dependent movement in that increment.

For purposes of the model, the elastic properties of the rock were reduced from the values of intact cores to account for the presence of bedding planes and other discontinuities. To define the strength properties of intact and broken bedding planes, core samples were tested in direct shear (Section 2.5.4.2.2). Using these data, the finite element models were used to predict the time-dependent vertical and horizontal rock movements behind 12-line wall.

Horizontal displacements of the 12-line wall were calculated for a 40-yr period and the resultant bending moments along the wall

were determined. These predicted displacements and bending moments were then incorporated into an analysis of the stability of the wall. Design changes were made to accommodate the conservative predictions of rock displacement.

Analyses of Vertical and Horizontal Strains

For the analysis of vertical and horizontal strains in proximity to structures, a monitoring program was established around the cooling water intake shaft (Section 2.5.4.13). Extensometers, inclinometers, and stressmeters were installed in the bedrock at distances of 1.5 to 44 m (5 to 145 ft) from the walls of the shaft. The instruments were installed in late 1979 while the groundwater level in the shaft was at el 62 m (205 ft). Shortly after the instruments were installed and baseline reading established, the shaft was dewatered (mid-December 1979). This allowed for a monitoring of the vertical and horizontal changes in the rock mass because of dewatering. Monitoring of the shaft will continue until after the shaft is rewatered.

Vertical Strains Based on the results of the monitoring of the effects of dewatering, two principal categories of vertical strain resulting from shrinking or swelling of the rock were recognized. One category represents the shrinkage associated with groundwater drawdown. The second category of vertical strain is the swelling or shrinkage resulting from near-face changes associated with evaporation, condensation, and other environmental factors affecting moisture availability at the free face of an excavation. The depth of influence of this seasonal effect is approximately 3 m (10 ft). Because the engineered structures in the excavations shield the excavation faces from these seasonal climatic fluctuations, the influence will be negligible. Therefore, the only significant vertical strain to be expected and accounted for in the design criteria is the vertical strain related to allowing the phreatic surface to return to el 78 m (255 ft) from its current lower elevation.

Based on calculations of the shrinkage experienced in extensometers EX-2 and EX-3, it can be shown that the 25 m (83 ft) of dewatered rock column experienced a vertical shrinkage of 0.015 in/1,000 in or 0.0015 percent strain. EX-4, located near the northwest corner of the shaft, experienced 0.005 percent strain. However, the total vertical strain in EX-4 is a combination of vertical displacement as a result of stress relief because of the excavation of the tunnels and vertical displacement as a result of dewatering. Using a Young's modulus of 140,600 kg/sq cm (2 x 10^6 psi) for the rock, a stress of 141 kg/sq cm (2,000 psi) in the corner of the shaft, a Poisson's ratio of 0.25 and a tunnel height of 4 m (14 ft), it can be shown that approximately 0.0042 percent strain can be attributed to stress relief as a result of the tunneling. Therefore, the 0.0015 percent strain calculated from readings in EX-2 and EX-3 appears to be an accurate representation of the vertical strain to be expected from dewatering. A similar, or slightly larger

strain would be expected from rewatering. Therefore, to accommodate possible increases in the volume of rock because of swelling, a conservative design value of 0.005 percent strain was chosen.

Horizontal Strain Because of the similarity of rock properties and stresses between the Niagara Falls area and the Nine Mile Point site, it was decided to utilize the long record of deformation at Niagara Falls to predict the future horizontal displacements at Nine Mile Point. The most fully documented example of rock swell in the Niagara Falls region is that of the wheelpit of the Canadian Niagara Power Company's Generating Station close to Horseshoe Falls. Closure across the 5-m (18-ft) width of the 174-m (571-ft) long by 50-m (165-ft) deep wheelpit exceeded 5 cm (2 in) in the more than 50-yr period since the excavation was completed in 1903 until the situation was disturbed by other excavations in 1954. Comparison between laboratory results of ring swell, triaxial swell, and free swell tests (Section 2.5.4.2.2) on the rocks at Nine Mile Point with free swell data obtained from tests on laboratory specimens on Rochester Shale and Gasport Shale formations indicate that they exhibit similar free swell behavior. Swell in each case is approximately 0.1 percent per log cycle of time (cycle 1: 0.1 to 1 hr; cycle 2: 1 to 10 hr; etc.). This suggests that the corresponding rock types may be expected to exhibit similar rock swell behavior on the large scale and that it is possible to estimate the long-term time-dependent closure around the Unit 2 reactor from the long-term closure around the Niagara Wheelpit. Thus, assuming the two rock formations (Rochester Shale and Pulaski Shale) exhibit the same creep or swelling characteristics, the difference in the magnitude of the creep will be the result of the difference between the regional stress fields in the two locations and the difference in geometry between the two excavations.

To compute the predicted strains, a boundary element program (an elastic analysis that permits modeling of the actual shape of the excavations) was utilized. Because the stresses around the excavations are known to be less than the levels required for nonlinear deformation of the rock, it is possible to invoke the correspondence principle[199]. This case consists of a pair of two-dimensional viscoelastic problems in which the material behavior is identical for each; the boundary conditions in each case involve a traction-free internal boundary and a biaxial stress field at infinity. Knowing the time-dependent displacement of a point in one of the problems (Niagara Wheelpit), therefore, allows for the computation of the general time-dependent behavior of the strain and displacement fields at the other (Nine Mile Point).

The elastic deformation field and distribution of stresses around each excavation were determined by using a boundary element program. The computations were completed for displacements in both the north-south and east-west directions around the reactor

excavation. The results indicate a maximum inward movement of the excavation walls of slightly more than 1.8 cm (0.7 in) in the east-west direction 50 yr after completion of the reactor excavation. The maximum horizontal extensional strain of the rock was calculated to be approximately 0.02 percent at the face, and the maximum shear strain in the corners of the excavation should not exceed 0.04 percent.

2.5.4.1.5 Hazardous Soil and Rock Conditions

No safety-related structures are founded on the natural soils at Nine Mile Point. Therefore, evaluations of the consolidation, water content, and other characteristics of the soils are not pertinent to this study.

The rock units exposed in the excavations at the site are fully lithified, unweathered, and are not subject to solutioning. Several minor buckle folds were observed in the excavation floors during excavation[154]. The development of these buckles can be attributed to the thinly bedded nature of the rock units, the high horizontal stresses, and the removal of the overlying confining pressure. Because structures are placed in these excavations, thereby increasing the vertical confining pressure, the potential for further development of the buckles or the initiation of new buckles is unlikely.

The argillaceous rock units at the site are susceptible to volumetric changes (Section 2.5.4.1.4) depending upon the availability of moisture. Several analyses have been completed that calculate the horizontal and vertical swelling capacities of the rock (Section 2.5.4.1.4). Based on these calculations, design criteria have been established and implemented (Section 2.5.4.11).

2.5.4.2 Properties of Subsurface Materials

This section describes the static and dynamic engineering properties of the rock mass at the Nine Mile Point site. The glacial deposits overlying the bedrock at the site were removed and do not constitute the foundation materials for any of the power plant structures.

The methods used to determine the rock properties are referenced in each of the sections, and Figure 2.5-89 presents a compilation of all the property tests in relation to the composite stratigraphic section. Summary tables cataloging the important test results are presented and referenced in the appropriate sections. Testing procedures, where different from referenced standards, are detailed in Appendix 2J.

2.5.4.2.1 Index Properties

<u>Density</u>

Fifty wet density and 11 dry density laboratory determinations were made on rock core samples from the site according to procedures outlined by the International Society of Rock Mechanics[200]. Table 2.5-5 tabulates the density determinations according to rock types. All tested samples were 15-cm (6-in) diameter cores from borings RT-1, RT-2, and GP-1. All tests were completed in the winter of 1977-78 and were performed as part of the time-dependent displacement evaluation program and the program of geophysical evaluation of rock properties.

Generally, the wet density values for the various rock types are very similar. Average density values range from 2.65 g/cc (0.096 lb/cu in) with a standard deviation of 0.08 (0.003) for sandstone to 2.68 g/cc (0.097 lb/cu in) with a standard deviation of 0.05 (0.002) for shale. The sandstone density determinations indicate that the sandstones in the lower part of the site stratigraphic column (Whetstone Gulf) are more dense than the sandstones from the upper part of the column (Oswego Sandstone). In fact, the samples tested from the Oswego Sandstone have an average density of 2.59 g/cc (0.094 lb/cu in) with a standard deviation of 0.04 (0.001).

The tested shale samples proved to be, on the average, the most dense rock type at the Nine Mile Point site with an average value of 2.67 g/cc (0.097 lb/cu in) and a standard deviation of 0.02 (0.001). Because the shale units are more numerous with depth, the rock mass tends to be more dense with depth.

Because of the low porosity values of the rock units, the dry density values (Table 2.5-5) are slightly lower but similar to the wet density values. The average dry density value of all shale samples tested is 2.63 g/cc (0.097 lb/cu in) with a standard deviation of 0.02 (0.001) or 98 percent of the average wet density value for shale. During the course of the geophysical crosshole testing to evaluate the in situ dynamic properties of the rock mass, down-hole density determinations were obtained using a borehole compensated density tool. This device contains a gamma-ray source that obtains representative density values by averaging the measurement made at the recording depth with values obtained 1 ft above and below the recording depth. Values thus obtained are more representative of average rock mass values rather than specific rock types. Density values obtained from the down-hole density survey are presented as Tables 2.5-6 through 2.5-8. Values range from a low of 2.63 g/cc (0.095 lb/cu in) in the Oswego Sandstone to a high of 2.74 g/cc (0.099 lb/cu in) in the Whetstone Gulf Unit B. These values are in very close accord with those obtained on individual rock samples.

<u>Porosity</u>

Eleven laboratory determinations of porosity were completed during the testing of the time-dependent displacement evaluation program in the winter of 1977-78. The samples tested,

predominantly shale, were the same as those on which dry density values were determined. Procedures used for porosity determinations are specified in Part 1-3 of the International Society for Rock Mechanics (ISRM) Document No. 2[200]. The results, presented as Table 2.5-9, indicate that the rocks have a low porosity, typically in the range of 3 to 5 percent.

Water Content

Thirty-five rock core samples were tested for water content (Table 2.5-9) according to the specifications of the ISRM[200]. These rock cores were taken from below the water table in borings GP-1 and RT-2. The water content of the rock samples is generally less than 2 percent. The sandstone demonstrated the lowest water content, with an average for all samples tested of approximately 1 percent. The finer-grained rock types (shale and siltstone) contain a greater percentage of water, but never exceed 2.7 percent. Comparison of the water content with the dry density and porosity indicates that the samples were generally fully saturated at the time of testing.

Mineralogical Analyses

In 1977, six samples from boring TD-1 were chosen for petrographic and mineralogic examination. Two samples each of sandstone, graywacke, and shale were examined in thin section and subjected to X-ray diffraction analysis. Heavy mineral and carbonate content determinations were also made. Table 2.5-10 lists the same depth, rock type, and petrographic composition of the six samples, and Table 2.5-11 presents the results of the modal analysis by X-ray diffraction. The clay proportions of three additional samples of shale from boring T-3-1 were subjected to X-ray diffraction analysis.

Sandstone Two sandstone samples were composed primarily of quartz and rock fragments and were classified as submature lithic arenites. The low porosity of the sandstones is probably a result of the secondary silica cementation. The rock matrix has a low percentage of clay minerals, and the bedding appears to be weakly developed. Scattered grains of calcite and of opaque minerals (possibly pyrite) were also observed, and heavy mineral analysis revealed traces of magnetite.

Graywacke The two graywacke samples were judged to be lithic graywackes by examination of thin sections. Determinations of silica content both by thin section inspection and X-ray diffraction analysis yield a common value of approximately 50 percent, the majority of which is quartz. The clay mineral content represents approximately 35 percent of each sample.

Secondary silica cementation appears to be locally developed, but it is evident that a connected network of cemented grains is not present in these samples. A higher proportion of the matrix is composed of argillaceous material than in the sandstones.

Shale Clay minerals compose approximately 55 percent of both shale samples from boring TD-1, with kaolinite dominating the clay mineral assemblage. A characteristic of these shales which clearly differentiates them from the sandstone and graywackes is the predominance of a clay matrix. The sand and silt size grains are entirely separated by argillaceous material. However, in common with the graywacke and sandstone samples, grains of opaque materials, possibly pyrite, are scattered throughout the rock. In contrast to the graywackes and sandstones, traces of organic material were detected in the shale by both X-ray and optical techniques. X-ray diffraction analyses of three samples from boring T-3-1 indicate that the shale samples are 60 to 65 percent illite with the remainder of the sample being silica and chlorite.

Mineralization Diagenetic and epigenetic minerals were encountered in the rock mass at Nine Mile Point. The diagenetic minerals, calcite and sulfides, are found on bedding planes throughout the rock mass, and their temperatures of formation, as determined by fluid inclusion analysis, correspond well with the temperature estimated for the stability field of the host rock mineralogical assemblage (180°C to 150°C, 356°F to 302°F)[94].

The epigenetic mineralization is characteristically a vein mineralization deposited on fractures, faults, and bedding planes. Microscopic examination of many samples of this mineralization has revealed a paragenetic sequence that shows the relative temporal succession of the minerals and their relationships to temperatures and deformational events (Section 2.5.1.2.6). The epigenetic minerals, also predominantly calcite and sulfides, are most apparent along the fractures associated with the Cooling Tower fault. A complete description of the mineralogy and paragenesis is presented in previous reports[94,154].

With the exception of the direct association of the epigenetic mineralization with the high-angle fractures of the Cooling Tower and Drainage Ditch faults, the mineralization itself does not affect the physical properties of the rock mass. The mineralization is most typically found as a small cluster of crystals or a thin film on fracture surfaces that only locally effectively heals the fracture.

2.5.4.2.2 Deformational Properties

Elastic Modulus

As part of the site investigations for the three power plant sites in the Nine Mile Point vicinity, a number of laboratory and field tests were performed from which elastic moduli can be calculated. Ranges in moduli values from the various tests are compared on Figures 2.5-76 and 2.5-90, and brief descriptions of the sources of data follow this introduction.

A geophysical refraction survey detailed in the 1964 report[201] provided data allowing dynamic moduli to be calculated. The dynamic Young's modulus above el 32 m (105 ft) was 3.8 x 10^5 kg/sq cm (5.4 x 10^6 psi), and the dynamic Young's modulus below el 32 m (105 ft) was 6.1 x 10^5 kg/sq cm (8.6 x 10^6 psi). A 1968 downhole 3-D survey was done using boreholes 205, 206, and 217 of the James A. FitzPatrick plant site[93]. Values of elastic moduli were calculated from measured compressional wave velocities between the depths of 6 and 44 m (20 and 145 ft) below the ground surface. The value of dynamic Young's modulus from the 3-D survey has a distinct change at the 24 m (80 ft) depth. Between the depths of 9 and 24 m (30 and 80 ft), the mean dynamic Young's modulus is 3.4 x 10^5 kg/sq cm (4.8 x 10^6 psi) with a standard deviation of 2,800 kg/sq cm (0.4 x 10^6 psi). Below the depth of 24 m (80 ft), the mean decreases to 2.7 x 10^5 kg/sq cm (3.8 x 10^6 psi) with a standard deviation of 35,200 kg/sq cm (0.5 x 10^6 psi).

The results of an uphole and crosshole survey in 1972[202] allowed the calculation of a dynamic Young's modulus. For the uphole survey, the average modulus down to a depth of 30 m (100 ft) was 3.9 x 10^5 kg/sq cm (5.6 x 10^6 psi). For the crosshole survey, the dynamic moduli based on compressional wave and shear wave velocities were 3.8 x 10^5 kg/sq cm (5.4 x 10^6 psi) and 3.7 x 10^5 kg/sq cm (5.2 x 10^6 psi), respectively. The modulus values calculated from geophysical surveys inevitably reflect the characteristics of the stiffer rocks and should be considered as upper bound values. Also, the dramatic increase in modulus at el 32 m (105 ft) in the 1964 report is inconsistent with static modulus results and site stratigraphy. Details of testing procedures for geophysical surveys are provided in Section 2.5.4.4.1.

In 1978, a series of sonic velocity tests was conducted to determine dynamic elastic moduli of samples from borehole GP-1. Results from the tests are presented in Table 2.5-12. The tests were performed in accordance with ASTM Designation D2845-69, Laboratory Determination of Pulse Velocities and Ultrasonic Elastic Constants of Rock. Young's modulus values measured ranged from 1.8 to 8.4 x 10^5 kg/sq cm (2.5 to 12.0 x 10^6 psi) with a mean value of 4.4 x 10^5 (6.2 x 10^6), and a standard deviation of 1.8 x 10^5 (2.5 x 10^6).

Ten unconfined compression tests were performed on rock cores from the 400 series borings as part of the PSAR studies for Unit 2[202]. The modulus of elasticity of each of these samples is reported as part of Table A-4 of the referenced report. The modulus values reported range from 3.1 to 6.3 x 10^5 kg/sq cm (4.4 to 9.0 x 10^6 psi) with a mean of 4.4 x 10^5 (6.2 x 10^6), and a standard deviation of 98,400 (1.4 x 10^6). The elastic moduli were calculated according to ASTM Designation D3148-72.

During the geologic investigations for Unit 2, a series of triaxial tests was performed on rock cores to evaluate the compressive strength of the different rock types and to determine the relationship of the inclination of the bedding to the compressive strength. Details of the testing are provided in Section 2.5.4.2.3; the modulus values that resulted from this testing follow.

In 1977, nine triaxial tests were performed on 1/2-in diameter cores from three samples from borehole T-3-3. Samples of siltstone, shale and interbedded siltstone, and shale were tested at confining pressures of 14 to 28 kg/sq cm (200 to 400 psi). The testing of the samples resulted in an average vertical modulus (perpendicular to bedding) of 2.7 x 10^5 kg/sq cm (3.8 x 10^6 psi) with a standard deviation of 703 (0.01 x 10^6) for shale; 2.7 x 10^5 kg/sq cm (6.7 x 10^6 psi) (standard deviation of 28,100 (0.4 x 10^6) for siltstone; and 4.7 x 10^5 kg/sq cm (4.6 x 10^6 psi) with a standard deviation of 2,810 (0.04 x 10^6) for interbedded siltstone and shale. A summary of the test results is presented as Table 2.5-13.

Also in 1977, 13 samples of interbedded siltstone and shale were tested with the bedding oriented 30 deg to the specimen axis. Resultant modulus values averaged approximately 7.03 x 10^5 kg/sq cm (1.0 x 10^6 psi) with a standard deviation of 14,060 (0.2 x 10^6). A summary of these test results is presented in Table 2.5-14.

In 1978, as part of the time-dependent displacement evaluation of the rock mass, 30 rock samples from borings RT-1 and RT-2 were subjected to triaxial compressive strength tests. The purpose of these tests was to determine the compressive strength of different rock types for different bedding inclinations. Confining pressures of 3.5, 53, and 105 kg/sq cm (50, 750, and 1,500 psi) were used on 1/2-in diameter, 1-in long test specimens that were oriented perpendicular (90 deg), parallel (0 deg), and 45 deg to the bedding. Details of the testing procedure can be found in Section 2.5.4.2.3 and Appendix 2J, Section 2J.4. Young's modulus values, calculated as secant moduli from the stress-strain curves, are presented as Table 2.5-15. As can be seen from the results, the moduli values are unaffected by confining pressures within the range tested, but differ according to the inclination of the bedding planes.

A series of unconfined compression tests was done on samples from borings 7-3-1 and MPX-1, and results are presented as Table III-L-1 in the 1978 Geologic Investigation[94]. Values of moduli range from 2.2 to 4 x 10^5 kg/sq cm (3.1 to 5.7 x 10^6 psi) with an average of 3 x 10^5 (4.3 x 10^6) and a standard deviation of 3 x 10^5 (0.9 x 10^6).

Several uniaxial modulus tests were conducted in 1978. The object of this set of tests was to determine values for the five independent elastic parameters that characterize a transversely

isotropic material. The parameters to be evaluated were: Young's modulus and Poisson's ratio for a specimen loaded uniaxially in a direction perpendicular to the bedding, and Young's modulus and Poisson's ratio for a specimen loaded uniaxially in a direction parallel to the bedding and the shear modulus of the rock.

The uniaxial compression test with strain measurements for elastic parameter determination has been covered by the ISRM[203]. These suggested methods apply to isotropic rock rather than to rock with anisotropic characteristics. However, the suggested test methods and equipment are applicable to measurements on anisotropic rocks and have been applied with suitable extensions and adaptations as discussed in Appendix 2J, Section 2J.4.

Values for Young's modulus and Poisson's ratio were computed and are listed in Table 2.5-16. These elastic parameters are chord (secant) values and appear to be consistent with those that one would expect for rocks of this nature. The calculated moduli averaged 2.8×10^5 kg/sq cm (4.0×10^6 psi) with a standard deviation of 1.4×10^5 (2.0×10^6) for samples oriented 90 deg to bedding, 2.1×10^5 kg/sq cm (3.0×10^6 psi) with a standard deviation of 7×10^5 (1.0×10^6) for samples oriented 45 deg to bedding, and 3.5×10^5 kg/sq cm (5.0×10^6 psi) with a standard deviation of 7×10^5 (1.0×10^6) for samples oriented parallel to bedding.

The general behavior of the rocks agrees with the standard behavior of sedimentary rocks. The test records show an initial concave curve of axial stress-strain resulting from pore closure, followed by linear elastic behavior through most of the range. As the sample approaches failure, a rapid radial expansion results from the development of axial cracking. Most specimens displayed this or similar behavior. Values obtained for the elastic parameters suggest that differences between rock types are relatively minor. The degree of elastic anisotropy also appears less marked than is often the case. The rocks in general may be described as moderately anisotropic.

During 1977 and 1978, a series of overcore stress measurements was performed at the Unit 2 site (Section 2.5.4.1.4). Part of the procedure is a determination of the modulus of elasticity by use of a biaxial loading chamber. The recovered overcorespecimen and the borehole deformation gauge are placed in a steel chamber which is designed to provide a radial stress of 140 kg/sq cm (2,000 psi). The specimen is loaded and axial deformations are measured via the borehole deformation gauge, and the elastic modulus is calculated. Modulus values calculated in this manner are summarized in Table 2.5-17. The values ranged from 1.1 to 6×10^5 kg/sq cm (1.6 to 8.5×10^6 psi) with a mean value of 3×10^5 (4.3×10^6) and a standard deviation of 9×10^4 (1.3×10^6).

<u>Dynamic Rock Properties</u>

During the period from 1964 to 1978, several laboratory and field tests were performed for the purpose of determining dynamic deformational properties of subsurface materials at the Unit 2 site. These tests included several field scale geophysical surveys in addition to shock tests on small-scale samples. The results of the surveys and laboratory tests produced values of shear wave velocities, compressional wave velocities, dynamic Young's modulus, dynamic shear modulus, and Poisson's ratio. Values of dynamic Young's modulus and Poisson's ratio for the rock units on the site are also presented in Section 2.5.4.2.2.

A 1964 seismic refraction survey was included as part of the PSAR[82]. Results from the survey report compressional wave velocities of 4,300 m/sec (14,000 ft/sec) for materials above the 30 m (100 ft) elevation and a compressional wave velocity of 4,880 m/sec (16,000 ft/sec) below the 100-ft elevation. However, the apparent rise in compressional wave velocities at the 100-ft elevation is questionable because of the limitations in the state of the art when the survey was completed.

A geophysical survey was done in 1968 for the James A. FitzPatrick[93] site. This information was also reported as part of the PSAR for Unit 2[82]. Reported compressional wave velocities range from 3,367 to 4,600 m/sec (11,046 to 15,093 ft/sec). Shear wave velocities range in value from 1,694 to 2,445 m/sec (5,559 to 8,020 ft/sec). Young's modulus, shear modulus, and Poisson's ratio can be calculated from the values of the compressional wave velocities and shear wave velocities. The range in calculated dynamic Young's modulus is from 1.9 to 4.1 x 10^5 kg/sq cm (2.7 to 5.9 x 10^6 psi) with an average value of 3 x 10^5 (4.2 x 10^6) and a standard deviation of 4.9 x 10^4 (0.7 x 10^6). Values of dynamic shear modulus range from 7 to 16 x 10^4 kg/sq cm (1.0 to 2.3 x 10^6 psi) with an average value of 1.1 x 10^5 (1.6 x 10^6) and a standard deviation of 2.1 x 10^4 (0.3 x 10^6).

An uphole velocity survey was conducted in 1972 in boring 406. A shear wave velocity survey also was conducted using boring 414 and other surrounding borings at distances up to 270 m (900 ft). The details of these surveys are reported in the foundation investigation addendum to the PSAR[82]. The uphole-measured value of the compressional wave velocity reflects the average value of the various rock types and ranges from 3,780 to 3,930 m/sec (12,400 t0 12,900 ft/sec). The shear wave velocity measured crosshole was approximately 2,345 m/sec (7,700 ft/sec).

In 1978, a 3-D geophysical survey was conducted (Section 2.5.4.4.2). Values of compressional wave velocities, as calculated from this survey, range from 3,680 to 5,575 m/sec (12,080 to 18,290 ft/sec). Shear wave velocities range from 1,100 to 2,250 m/sec (3,620 to 7,380 ft/sec). The calculated shear modulus values range from 3.3 to 14 x 10^4 kg/sq cm (0.469 to 1.995 x 10^6 psi).

A series of shock scope tests was performed on rock samples from the 400 series borings. These values are reported as Table A-3 in the foundation investigation addendum of the PSAR[82]. Reported values of compressional wave velocities range from 1,980 to 8,565 m/sec (6,500 to 28,100 ft/sec).

An additional series of shock scope tests was performed in 1978 on 27 samples from borehole GP-1. Rock types tested included sandstone, siltstone, and graywacke. The shock scope tests were performed in accordance with ASTM Designation D2845-69, Laboratory Determination of Pulse Velocities and Ultrasonic Elastic Constants of Rock. Measured and calculated values of elastic constants are presented in Table 2.5-12.

In summary, the compressional wave velocity, as determined from the shock scope testing, ranges from 3,196 to 5,708 m/sec (10,485 to 18,728 ft/sec). The shear wave velocities ranged from 1,787 to 3,514 m/sec (5,864 to 11,529 ft/sec). Elastic constants calculated from these velocities are Young's modulus, shear modulus, and Poisson's ratio.

Comparisons of the modulus values obtained by the various methods described, classified by rock type and stratigraphic unit, are presented as Figures 2.5-76 and 2.5-90. A similar comparison of Poisson's ratio values is presented as Figure 2.5-91.

Poisson's Ratio

The 3-D survey performed in borehole 206 at the James A. FitzPatrick plant site in 1968 reports an average value of Poisson's ratio of 0.31 and a standard deviation of 0.01. An additional 3-D geophysical survey was performed in 1978 in boreholes GP-1, GP-2, and RT-1 at the Unit 2 site. The surveys provided values of Poisson's ratio for rocks at depths up to 108 m (353 ft). Values obtained average 0.29 with a standard deviation of 0.04. Values from this survey and the previous 3-D survey (1968) were averaged together for inclusion on Figure 2.5-91 for comparison with values of Poisson's ratio from other tests.

A series of unconfined compression tests done of samples from the 400 series of borings allowed for the calculation of Poisson's ratio. Values obtained from these calculations average 0.15 with a standard deviation of 0.05 and are presented as Table A-4 of the foundation investigation addendum to the PSAR[82]. Additional values of Poisson’s ratio were reported as Table III-L-1 of the 1978 Geologic Investigation[94]. The average value of Poisson’s ratio from Table III-L-1 is 0.14 with a standard deviation of 0.01.

In 1978, a series of unconfined compression tests was conducted. In these tests, Poisson's ratio was calculated for samples taken at 90 and 0 deg to the plane of bedding. Table 2.5-16 presents a summary of the results of the testing.

In 1978, a series of shock scope tests on rock cores from boring GP-1 was performed. Table 2.5-12 contains the Poisson's ratio values calculated as a result of these tests. The reported Poisson's ratios have an average value of 0.21 and a standard deviation of 0.06.

In summary, the determinations of Poisson's ratio point out the significance of test variability. Values of Poisson's ratio varied less between rock types than they did between test procedures. It does appear, however, that sandstone tends to have the lowest Poisson's ratio and siltstone has the highest Poisson's ratio.

Swell Tests

The investigation of rock properties included testing of swell properties of the site rocks. The results of this testing were used to assess the relationship between the rock units at Nine Mile Point and the characteristic swell properties of rocks from the region. The testing program consisted of a series of swelling strain observations over an extended period of time, and several tests were specifically designed to evaluate the effects of environmental humidity upon the swelling behavior. Results of the swell tests for the various rock units are presented as Table 2.5-18.

The objective of the swell tests was to index and evaluate the nature of the time-related swelling characteristics of the near-surface rocks at the site. The investigation of rock swell included free swell, ring swell, and triaxial swell tests. The free swell tests included three categories of tests using different sequences of high and low humidity environments. The test results are briefly summarized herein, but a more complete description of the tests is available in the 1978 Geologic Investigation[94]. As a result of the three series of tests, it is apparent that swelling is controlled by water content and that little or no strain due to stress relief alone or other recognizable causes occurs. However, the swell strain rates decrease with an increase of the first stress invariant, and there exists a limiting value of the first stress invariant beyond which no swell deformation occurs.

Unconfined Swell Tests Three categories of unconfined swell tests were performed on rocks from the site in 1977. The main program consisted of a series of strain observations under conditions of greater than 95 percent humidity. These observations were designed to be similar to previous tests conducted on rocks in this region. Such tests have been described by a number of investigators, such as Lee and Klym[204], and Lee and Lo[205], and have been termed free swell tests. A brief description of the three categories of unconfined swell tests and the results of the testing follows. Details of the test procedures and a more complete discussion of the test

results are provided in Section 6.0 of the 1978 Geologic Investigation[94].

Free Swell Tests

Based on the unconfined swell tests of 83 or more days duration of five sandstones, two siltstones, and four interbedded shales, sandstones, and siltstones, the following results were obtained:

1. All samples appeared to swell. The rates and magnitudes of swell exhibited are grossly different and appear to be grouped according to lithology.

2. In almost all tests, the rate of swell normal to bedding exceeds the rate of swell parallel to bedding. For siltstone and interbedded rock samples, the rate of swell normal to bedding commonly exceeds the rate parallel to bedding by a factor of 2 to 3.

3. The sandstones demonstrated the lowest rate of swell of all rock types tested, both parallel and normal to bedding. Quite commonly, swelling essentially ceased after approximately 1 day.

4. Samples that included shale beds demonstrated the greatest rates of strain.

5. Graywacke samples were characterized by inconsistent strain rates. The general ratio of the strain rate normal to bedding compared to that parallel to bedding was inconsistent and varied from 1.0 to approximately 2.7. This is in contrast to the other argillaceous rocks which always demonstrated greater swell normal to bedding than parallel to bedding.

Drying/Slaking Tests Without Prior Unconfined Swell

Five samples collected from the Pulaski Formation in boreholes TD-1 and OC-4 were stored in conditions of ambient site humidity during July and August 1977. Samples tested in this manner consisted of two sandstones, one sandy shale, one subgraywacke, and one graywacke with sandstone and siltstone interbeds. After approximately 12 days the samples were transferred to an atmosphere of greater than 95 percent humidity and the associated strain versus time relations were monitored.

Upon being placed in an environment of high humidity, all samples showed expansion on all axes at high strain rate. After a period of generally less than 10 days of expansion, a marked reduction in strain rate occurred along all axes. Generally, the magnitudes of strain both during drying and subsequent slaking were greater normal to bedding than in the plane of bedding in the relatively argillaceous rock cores.

Drying/Slaking Tests With Prior Unconfined Swell

Four samples that had previously undergone a period of unconfined swelling were monitored in an approximately 50-percent humidity environment and then transferred to a 100-percent humidity environment. Based on the testing, the following results were reported[94].

1. All samples demonstrated that strain, strain rate, and sense of strain are a function of environmental humidity.

2. Change of environmental humidity was accompanied by a reversal of sense of strain. In all tests, a reduction of the environmental humidity was accompanied by the development of shortening strains and an increase in environmental humidity was accompanied by the development of extensional strains.

3. Strain rates following changes of humidity were greater than the rates observed at any time during unconfined swell. Furthermore, in the relatively argillaceous rocks, strain rates following an increase of humidity were greater than those following a decrease of humidity.

4. When fractures did not develop in the relatively argillaceous rocks in the drying period, samples responded by shortening a greater magnitude normal to bedding than in the plane of bedding.

5. The drying/slaking cycle appears to have promoted fracturing and disintegration in two out of the three relatively argillaceous test samples. The sandstone, however, did not appear to fracture or disintegrate as a result of this cycling procedure.

Confined Swell Tests Axial and radial monitoring of rock cores under three-dimensional confining pressure was accomplished with ring tests and triaxial tests.

Ring Tests

Ring tests were conducted on rock cores to measure time-dependent dimensional changes of rock specimens under three-dimensional confining pressure. The ring tests involved the monitoring of rock cores that had been grouted into thin-walled aluminum rings. Theoretically, expansion of the rock in the horizontal direction would result in an equal expansion of the ring that was monitored with strain gauges. Different thickness rings provided different confining pressures and a load frame allowed for application of an axial confining pressure. Three types of ring tests were performed: swell-strain tests (axial stress maintained at overburden pressure), swell-stress tests (axial load adjusted to

maintain constant specimen height) and creep-strain tests (axial stress maintained at overburden pressure and specimen sealed to retain initial water content). A total of 25 ring tests were performed, and the results of the testing are presented as Table 2.5-18. Test procedures are presented in Appendix 2J, Section 2J.2.

In general, the results of the various ring tests show that sealed specimens continue to shrink circumferentially without appreciable influence from changes in axial load. When water is introduced to the specimen, a strong swelling trend ensues. This indicates that water is the predominant requirement for rock expansion and that expansion as a result of stress relief alone does not occur.

<u>Triaxial Swell and Creep Tests</u>

The purpose of the triaxial tests was to measure axial and radial swell strains as a function of time under controlled axial and lateral confining stress conditions. The triaxial tests were intended as an extension of the ring swell and creep tests. They permitted the measurement of dimensional changes of rock specimens subjected to confining pressures higher than could be achieved by the aluminum rings.

Test procedures for the triaxial swell and creep tests are presented as Appendix 2J, Section 2J.3. Test results are summarized in Table 2.5-19.

In general, the triaxial test specimens, after an initial shrinkage, showed expansion in the axial and circumferential directions. Typically, the rate of axial expansion per log cycle was significantly greater than the rate of circumferential expansion. The creep test, in which the specimen was sealed to prevent water content change, showed a continued shrinkage tendency with shrinkage of similar magnitudes in directions parallel and perpendicular to the bedding.

In many respects these results are analogous with those from the ring tests, particularly with regard to the lack of stress relief expansion and to the predominant influence of water. Additionally, the triaxial specimens produced a greater rate of recorded swelling (in spite of the greater degree of confinement) than the ring tests. This would appear to be a result of the larger ratio of wetted surface area to specimen volume for the triaxial specimens than for the ring test samples.

2.5.4.2.3 Strength Properties

This section describes the results of the testing programs that were undertaken to characterize the loading conditions that caused failure in rock samples from the Nine Mile Point site. The testing programs include unconfined compression tests, triaxial compression tests, and direct shear tests.

Unconfined Compression Tests

A series of 10 unconfined compression tests was completed on samples taken from varying depths in the 400 series borings. These tests were done in 1972 as part of the foundation investigation for the PSAR. The compressive strength values varied from 1,220 to 2,020 kg/sq cm (17,300 to 28,700 psi) with an average value of 5,000 (21,000) and a standard deviation of 280 (4,000). Individual test values are listed in Table A-4 of the PSAR[82].

Additional unconfined compressive strength values were submitted as Table III-L-1 of the 1978 Geologic Investigation[94]. These tests were completed in accordance with ASTM Designation D2938-71A, and values reported ranged from 310 kg/sq cm (4,415 psi) for siltstone to 1,900 kg/sq cm (27,000 psi) for sandstone. The average value for all tests was 910 kg/sq cm (13,000 psi) with a standard deviation of 500 (7,000).

Triaxial Compression Tests

A series of triaxial compression tests to determine the strength of rock under lateral restraint was performed in 1977 and 1978. The tests were run on cylindrical specimens 1.3 cm (0.5 in) in diameter and 2.5 cm (1 in) long. Testing procedures and equipment are described in Appendix 2J, Section 2J.4. Results of the testing are summarized herein and are tabulated as Tables 2.5-13 through 2.5-15.

The first series of tests was made in May 1977 on samples from borehole T-3-3. Rock types tested were interbedded siltstone, shale, and sandstone from Unit A of the Pulaski Formation and silty shale from Unit B of the Pulaski Formation. All samples in this series were drilled perpendicular to the bedding plane. The results, as summarized in Table 2.5-13, indicate that for an effective pressure of 15 kg/sq cm (200 psi) the sandstone is the strongest rock type and has an average strength of 2,700 kg/sq cm (39,000 psi) with a standard deviation of 140 (2,000). The interbedded siltstone/shale averaged 980 kg/sq cm (14,000 psi) with a standard deviation of 210 (3,000), and the silty shale averaged 700 kg/sq cm (10,000 psi) with a standard deviation of 140 (2,000). As would be expected, the two confining conditions did not affect the results because the effective confinement was identical.

The second series of triaxial tests was done in July 1977 on samples from boring T-3-7. The samples were a finely layered interbedded siltstone and shale from Unit B of the Pulaski formation. Samples for testing were drilled such that the axis of the 1.3-cm (0.5-in) diameter cores was 30 deg from the plane of bedding. The results from the 12 tests are presented in Table 2.5-14. The average strength value of 190 kg/sq cm (2,700 psi) with a standard deviation of 40 (600) for the 13 tests run at an

effective pressure of 15 kg/sq cm (200 psi) was significantly lower than the average strength value of 980 kg/sq cm (14,000 psi) from the previous tests. It is clear that the shear strength of the bedding planes is much lower than that of the intact material and controls the apparent strength when loaded transversely. As in the previous case, the presence of higher pore pressure does not have a discernible effect on strength.

The final set of triaxial tests was completed in May 1978 on samples from borings RT-1 and RT-2. Included in the tests were samples of sandstone, siltstone, and graywacke. The purpose of the test was to determine the compressive strength of different rock types for different bedding inclinations. Tests on the cores were run at three different confining pressures and were oriented 90 deg to the bedding in the sandstone and 90, 45 and 0 deg to the bedding for the other rock types. The results of the 30 tests performed are found in Table 2.5-15. As expected, the tests indicate that the compressive strength of the various rock types increases with an increase in confining pressure and that the sandstone is the strongest rock type. Samples cut at 45 deg to the bedding, however, show much less response to the effects of confining pressure. Also, the values of compressive strength for the 45 deg samples are significantly lower than the strength for the 90 or 0 deg oriented samples. This indicates that, in loading conditions causing shear along the bedding planes, the material will fail along the bedding at a much lower stress than the compressive strength of the intact rock.

Direct Shear Tests

Direct shear tests were used to measure the shear strength of the horizontal bedding planes in the rock mass; the variation of this shear strength as a function of the overburden normal stress; and the magnitude of shear displacement (peak and residual shear strength values). Samples of shale, mixtures of sandstone and shale, and mixtures of sandstone and graywacke from boreholes RT-1 and RT-2 were tested.

The testing procedure used followed the Suggested Methods for Determining Shear Strength published by the ISRM, Committee on Field Tests[206]. The peak shear strength (maximum shear stress reached in the test) and corresponding normal stress were determined for each specimen as tabulated in Table 2.5-20.

Graphs of shear stress versus displacement had been constructed for the shear displacement prior to reaching residual strength during the second and third normal stress stages (Figure 2.5-92). Straight lines A, B, and C were fit to the steeply ascending portions of these data and are intended to represent upper, median, and lower bounds for the slope of the initial quasi-elastic portion of the stress-displacement curve.

The peak strengths recorded were highly variable as might be expected since, in some cases, intact rock was sheared, and in

other cases there was probably little bedding plane adherence. Residual values were measured by shearing intact rock only until a condition of residual strength has been reached. The values of coefficient of friction tan ϕ ranged from 0.16 to 0.57 with a mean value of 0.32 (ϕ average = 17.5 deg). As anticipated, they appear to vary little from one stratigraphic unit to the next, and the values may be taken as representative of the shale bedding planes, in general, at this site.

2.5.4.3 Exploration

Extensive geologic exploration and investigations for several purposes have been conducted at the Unit 2 site.

These explorations can be generally categorized according to their purpose:

1. General exploration which was conducted prior to excavation for site engineering structures.

2. Bedrock inspection and mapping documenting the condition of the excavations for site foundations.

3. Supplementary geologic exploration consisting of detailed investigations of specific geologic structures discovered during the excavation for site foundations and exploratory trenches.

4. Rock mechanics investigations of the bedrock properties and state of stress to augment the supplementary geologic exploration and assess the stability of the bedrock at the site.

A chronological summary of exploration at the site is presented in Table 2.5-21, and a list of the participants is presented in Table 2.5-22. A summary list of all borings completed at the Nine Mile Point site is presented in Table 2.5-23.

All exploration at the site is described in this section. Reference to other reports and docketed materials is made for specific information not presented in the FSAR. Detailed maps, cross sections, and photographs showing specific geologic relationships pertinent to discussion in other subsections of the FSAR are presented with this subsection. Additionally, maps, cross sections, and boring logs portraying site geologic conditions that have not been presented in other docketed materials are appended to this section.

2.5.4.3.1 General Exploration

The general exploration of geologic conditions at the Unit 2 site and vicinity was described in the PSAR and the FSAR for two existing nuclear power plants: Unit 1 and James A. FitzPatrick

(Docket No. 50-220 and Docket No. 30-333, respectively). Further information concerning geologic conditions at the site was provided in the PSAR for Unit 2 (Docket No. 50-410). Additionally, general exploration of subsurface conditions in the vicinity of the intake/discharge tunnels was conducted prior to their excavation. Eight NX-size borings (L-1 through L-8) were drilled for this purpose. Their locations are shown on Figure 2.5-93 and the logs of these borings are presented in Appendix 2K.

2.5.4.3.2 Bedrock Inspection and Mapping

The inspection and required mapping of bedrock surfaces exposed during excavation for the facility were performed by Stone & Webster Engineering Corporation (SWEC) geologists and verified on behalf of the Applicant by Dames & Moore geologists. This exploration was conducted in compliance with portions of the regulatory guides and responses to NRC Questions addressed to the PSAR. Excavations for all Category I structures, as well as excavations exposing anomalous geologic structures, were mapped in detail. All other excavations were inspected for the presence of unusual geologic features.

The geologic maps from this program, together with the limits of the excavations and the locations of safety-related facilities, are included in Appendix 2H.

2.5.4.3.3 Supplementary Geologic Exploration

Prior to specific geologic investigations at Unit 2, two supplementary investigations of geologic structures intersecting the James A. FitzPatrick plant excavations had been completed. In the spring of 1969, a west-northwest striking, steeply dipping, geologic structure was revealed in the excavation for the turbine building for the FitzPatrick facility. The subsequent report[153] concluded that this structure should not be considered significant in terms of project safety, seismic, or structural design. Then, in the fall of 1970, a small, west-northwest, moderately southward dipping normal fault was encountered while excavating the intake and discharge tunnels for this facility. The report describing this fault, entitled Study of Small Fault in Vicinity of Discharge Tunnel[93], concluded that it is a minor local feature of limited extent, and not of deep-seated tectonic origin.

Several phases of supplementary geologic exploration (Table 2.5-21) have been completed at the Unit 2 site. This work consisted of two investigations focusing on the geologic character and safety-related aspects of two geologic structures encountered in bedrock excavations for the facilities, i.e., the Cooling Tower fault and the Radwaste thrust fault. These investigations were conducted from 1976 through 1980, and have been completely documented in other docketed reports[94,154].

The various elements of the scope of work for each of the investigations are outlined in Table 2.5-21. Additionally, subsections depicting figures that show the location of the explorations, as well as selected maps, profiles, and cross sections pertaining to each investigation, are specified in Table 2.5-21. Numerous consultants were retained to supplement the investigations; they are listed with their corresponding contributions in Table 2.5-22.

Cooling Tower Fault Investigation

The Cooling Tower fault investigation was conducted from late 1976 to early 1978. It was a multidisciplinary investigation of the high-angle fault discovered in the original Cooling Tower excavation (Figure 2.5-28). This structure was investigated with a series of trenches and two lines of closely spaced borings (Table 2.5-21 and Figures 2.5-28 and 2.5-93). At the same time, available exposures of two other faults (Drainage Ditch and Barge Slip) in the northeastern portion of the site were also examined in detail.

The studies were directed toward defining the nature and extent of the Cooling Tower fault, the sequence of deformation, the relationship of the bedrock deformation to that of the overlying sediments, and the safety-related aspects of this structure. In conjunction with the geologic investigation, a study was conducted of the bedrock stresses at the site as described in Section 2.5.4.3.4. Results of the Cooling Tower fault investigation are summarized in Section 2.5.4.1.2 and have been presented in detail in a previous report[94]. Figures 2.5-31, 2.5-32, 2.5-34, 2.5-35, and 2.5-43 through 2.5-51 present maps and sections of the various excavations where the Cooling Tower fault was exposed.

Radwaste Thrust Fault Investigation

The investigation of shallow dipping thrust structures and low-amplitude folds, exposed in bedrock excavations at the site, began in late 1976 and continued until late 1980. These features were initially discovered in the autumn of 1976 in the excavations for the power complex (i.e., the north and south radwaste trenches, heater bay, normal switchgear building, circulating water intake building, and north notch of the reactor containment [Appendix 2H]). In early spring of 1977, thrust faults were discovered near the base of the cooling water intake shaft. An investigation of the age and cause of these structures was conducted during this period and reported as part of the Cooling Tower fault report[94].

In April 1979, similar structures consisting of zones of bedding plane slip and associated folding were found in the circulating water piping trenches for the relocated cooling tower. Additional low-angle thrust faults were discovered in each lake water tunnel during their excavation from late 1979 to 1980.

Also at this time, additional investigations of the thrust structure exposed in the north radwaste trench were initiated. In the spring of 1980, additional bedding slip zones were observed in the excavation for the relocated cooling tower.

The exploration for the investigation of these structures principally utilized exposure in the walls of existing excavations. However, additional excavation was performed to explore the larger of the thrust faults in the north radwaste trench (Figures 2.5-63 through 2.5-66 and Appendix 2H). Also, a series of borings was drilled to explore the eastward extent of this feature (Table 2.5-21 and Figure 2.5-93). Bedrock stresses within the thrust block, bounded by the high-angle faults (Section 2.5.1.2.3), were investigated in conjunction with the geologic studies as discussed in Section 2.5.4.3.4.

The purpose of the multidiscipline investigation of these structures (Table 2.5-21) was to attempt to define their age, origin, and safety-related significance. The results of the thrust structure investigation are summarized in Section 2.5.1.2 and presented in detail in other docketed reports[94,154].

2.5.4.3.4 Rock Mechanics Investigations

Rock mechanics investigations have been conducted at the site in a number of phases since 1977. The scope of work performed is extensive and can be subdivided into three basic programs. These are:

1. In situ stress measurements.
2. Evaluation of rock properties.
3. Bedrock monitoring program.

The elements of the scope of work for each of these programs are outlined in Table 2.5-21. Additionally, the table identifies subsections of this report wherein figures showing the location of this exploration, with maps, cross sections, or graphs pertaining to these investigations may be found. Consultants contributing to rock mechanics investigations are listed in Table 2.5-22.

<u>In Situ Stress Measurement Program</u>

A large number of in situ stress measurements[206], using the overcoring technique, have been conducted in 14 boreholes drilled at the site (Table 2.5-21). This program was undertaken in two phases (OC-series and RS-series) to augment the investigations of the Cooling Tower fault and Radwaste fault, respectively. The OC-series measurements were made at locations both within and outside the fault bounded block (Section 2.5.1.2.3). The RS-series measurements were made in four boreholes located in the interior of this block. The boreholes were situated along a line

essentially parallel to the dip direction of the Radwaste fault and situated progressively further from the front of the structure (Figure 2.5-87).

The purpose of both series of measurements was to evaluate the general characteristics of the contemporary near-surface stresses in the vicinity of the faults, and at the site in general. Additionally, the program was outlined to provide additional rock mechanics data to assist in the evaluation of the potential for movement on these structures during the anticipated life of the nuclear facilities.

Results of the in situ stress measurement program are summarized in Section 2.5.4.1.4. The individual data analyzed, in assessment of in situ stresses, have been presented in reports concerning the fault investigations[94,154].

Program to Evaluate Rock Properties

This program was conducted to evaluate four types of rock properties in view of their possible impact on geological and rock mechanical assessments at the site:

1. The unconfined swelling strains of 16 samples from boring TD-1 were measured under a variety of relative humidity conditions (Section 2.5.4.2). The purpose of this testing was to assess the possibility that past geologic changes may have promoted the development of swelling strains in the strata adjacent to the Cooling Tower fault, and to assess how future environmental changes might affect the bedrock surrounding the site foundations.

2. A program of undercoring of stress-relieved rock cores was conducted using four cores drilled from the rock surface of the cooling tower piping trench within 60 m (200 ft) of the fault (Figure 2.5-43). An additional core was selected from a horizontal borehole at 32 m (104 ft) below the top of rock at the intake shaft location (Table 2.5-21). Based both upon these records and appropriate stress measurement results, an assessment was made of residual strain effects (Section 2.5.4.2). This assessment was then used to judge the nature of the stored strain energy recorded in the rock mass at Nine Mile Point by overcoring, and the likely error incurred in the estimation of applied boundary stresses caused by any residual strain effects.

3. Bedrock geophysical testing was performed using three borings (GP-1, GP-2, and RT-1) as described in Section 2.5.4.4.2. The purpose of this testing was to establish the in situ dynamic properties of the bedrock beneath the site.

4. A program of laboratory testing of elastic and time-dependent rock properties was performed on samples collected from various locations as outlined in Section 2.5.4.2 and Table 2.5-21.

Rock Monitoring Program

The rock monitoring program was initiated at the site in early 1977, and monitoring of instrumentation will continue until January 1985. This program consists of four parts:

1. Between early 1977 and late 1981, 10 extensometers, 23 inclinometers, and 3 sets of stressmeters were installed in boreholes at and near the site (Section 2.5.4.13, Figure 2.5-93, and Table 2.5-21). The inclinometers and extensometers were installed for two purposes:

   a. To monitor possible bedrock movements of the rock mass around the excavations for input to the design of critical site foundations.

   b. To monitor possible movements along the Radwaste Thrust fault.

   The stressmeters were also installed for two purposes:

   a. To monitor changes in stress across the Cooling Tower fault.

   b. To monitor changes in stress associated with excavation and dewatering of the intake/discharge shaft and tunnels onsite.

   Results of this program are summarized in Section 2.5.4.13. Logs of these boreholes that were not presented in previously docketed reports are attached as Appendix 2K.

2. Measurement of closure and displacement of the walls of several excavations (intake shaft, trench 5, and 12-line wall slot) were made using a tape extensometer and precise surveying methods to assess the effects of relief of in situ stress. The results of monitoring these excavations are described in Section 2.5.4.13.

3. A system of mechanical gauges and electrical displacement sensors was installed in 1981 and 1982 at 11 locations around the reactor containment (Tables 2.5-21 and 2.5-33). The purpose of installing these instruments is to monitor possible changes in the widths of gaps between Category I structures to check for possible influence of bedrock movements. Results

of this monitoring program are presented in Section 2.5.4.13.

4. The monitoring of groundwater levels at the site was included as part of this program. Groundwater levels were measured in 19 temporary and permanent standpipe and piezometer installations from 1975 to 1981. The locations of these installations are discussed in Section 2.5.4.13 and the locations of the pneumatic piezometers (PI-1 through PI-5, and PI-20 and -21) are shown on Figure 2.5-93. These instruments were installed at different times and during different programs as specified in Table 2.5-21. The water level distribution and fluctuation is discussed in Section 2.4.13.

2.5.4.3.5 Geologic Subsurface Profiles

Geologic and subsurface profiles showing the geologic formations encountered during excavation and mapping of the underlying areas of all Category I structures for Unit 2 are shown on Figures 2.5-94 and 2.5-95. The locations of these cross sections are shown on Figure 2.5-96.

The stratigraphic division of the subsurface materials is based upon the criteria defined in Section 2.5.1.2.2.

These profiles are presented to illustrate the relationship of the foundations of all Category I structures to the geologic formations encountered. For profiles illustrating the subsurface geologic structures, refer to Section 2.5.4.3.1.

2.5.4.4 Geophysical Surveys

Since 1964 a number of field geophysical surveys have been conducted on and in close proximity to the Unit 2 site. In 1964 a seismic refraction survey was completed, and in 1968 a downhole 3-D velocity survey was run at the James A. FitzPatrick site. A 1972 uphole and shear wave velocity survey is reported in the Unit 2 site. Because all the other geophysical surveys are reported on elsewhere, only the 1978 survey will be detailed in this document.

The 1978 survey was conducted to establish the in situ dynamic properties of the rock beneath the site. These dynamic properties were used to confirm the design of the reactor foundation. The general location of these geophysical surveys is shown on Figure 2.5-93, and a detailed plot plan showing the locations of borings RT-1, GP-1, and GP-2 is presented as Figure 2.5-97. The three holes were arranged in a triangular pattern to permit the evaluation of velocity anisotropy in the horizontal plane. Borehole logging consisted of caliper, density, gamma ray, and 3-D velocity logging in boring GP-1. Density measurements were obtained to develop elastic property values and

representative density values of the rock at the depths at which crosshole measurements were made. The values obtained (Tables 2.5-6 through 2.5-8) are consistent with density values obtained by other techniques at Nile Mile Point (Section 2.5.4.2).

Gamma ray logs were obtained for hole-to-hole correlations of lithology. Borehole subsurface orientation surveys were performed in all three borings to permit calculation of subsurface hole-to-hole separation distances. Two subsurface orientation surveys in each boring were performed. Figure 2.5-97 illustrates the locations of the boreholes with depth as determined from the orientation survey.

2.5.4.4.1 3-D Velocity Logging

A 3-D velocity log was obtained in boring GP-1 to measure the compressional and shear wave velocities of the rock and for use in computing other elastic property values. The 3-D velocity tool contains an acoustic transmitter isolated from a receiver located a fixed vertical distance away in the same hole. A continuous record is made of all the events comprising the signal recorded as a function of wave transit time. From these records apparent velocities were computed (Section 2.5.4.2.2). In general, the velocities calculated from the 3-D velocity survey are 10 to 15 percent lower than the value obtained from the crosshole compressional survey.

2.5.4.4.2 Uphole Compressional Wave Survey

An uphole survey was conducted in borings GP-1 and GP-2 to evaluate vertically traveling compressional waves. In this survey, vertical hammer impacts were made on the ground surface adjacent to each boring, and recordings were made at depth intervals of 6 m (20 ft).

The data obtained from the uphole compressional wave surveys in GP-1 and GP-2 were evaluated by timing the onset of motion in the vertical element of the borehole geophone. The travel times were corrected to vertical travel paths and were plotted against depth to produce the time depth plots shown on Figures 2.5-98 and 2.5-99. Best-fit straight line segments drawn through the arrival time data were used to compute compressional wave velocities. The values derived from these surveys are, in general, agreement with the values obtained by other methods (Section 2.5.4.4.4).

2.5.4.4.3 Crosshole Shear Wave Survey

A downhole shear wave hammer was used as the energy source for the crosshole shear wave survey. The downhole hammer was locked in place in the hole and a slide weight of 15 kg (32 lb) was used to impart shear wave energy into the rock by striking the top of the hammer body for downward impact and the bottom of the hammer body for upward impact. A three-component borehole geophone was

secured in the recording borehole at the same elevation as the locked-in-place hammer, and a sequence of downward and then upward impacts were made and recorded. The resultant record illustrated waveforms for each geophone element for both upward and downward impacts.

Upon completion of the recording in RT-1 (using GP-2 as the energy source borehole), the entire procedure was repeated with boring GP-1 serving as the energy hole and borings GP-2 and RT-1 serving as the recording holes. The crosshole wave transit times determined from this survey were corrected for a transient signal time delay inherent in the design of the seismograph amplifier filters.

The results of the borehole orientation survey were used to determine minimum and maximum separation distances between borings at each depth for which a crosshole wave transit time had been computed. The corrected shear wave arrival times and the observed compressional wave arrival times were accurate to within ±0.10 and ±0.06 msec, respectively.

The resultant minimum time was used with the maximum distance to determine maximum velocity. Conversely, the maximum time was used with the minimum distance to determine the minimum velocity. The shear wave velocities determined in this manner are presented in Tables 2.5-6 through 2.5-8.

Figure 2.5-100 presents a graph of the average shear wave velocity computed for each depth interval tested and each leg of the triangular configuration. The shear wave velocity derived from a regression analysis using time and distance data from all three legs at each survey depth is also presented on Figure 2.5-100.

2.5.4.4.4 Crosshole Compressional Wave Velocity

Compressional wave velocities determined by the hammer method were supplemented with additional records having recorded compressional wave arrivals with explosives as the energy source. Crosshole compressional records were obtained using boring GP-1 as the energy hole with borings GP-2 and RT-1 as the recording holes, and using boring RT-1 as the energy hole with borings GP-1 and GP-2 as the recording holes.

The computed crosshole compressional wave velocities are presented on Figure 2.5-101 and in Tables 2.5-6 through 2.5-8. As can be noted from the velocity solutions for each leg of the triangular configuration, there is excellent agreement between the values.

A comparison between the compressional wave velocities obtained from the 3-D velocity logging of boring GP-1 and the compressional wave velocities obtained from the compressional

crosshole study indicates that the 3-D survey velocities are consistently 10 to 15 percent lower than the crosshole velocities. This relationship would seem to be consistent with the lithologic character of the rock. Horizontally traveling waves, specifically those observed during the crosshole work, will exhibit a velocity approaching the velocity of the most competent layers encountered along the travel path. Vertically traveling waves propagating through the same rock will be influenced to a greater degree by the less competent, lower velocity layers. Such is the case because in the vertical direction the competent layers are separated by less competent layers, while horizontally, the more competent layers can show continuity between two points of similar elevation.

2.5.4.4.5 Summary

The in situ dynamic properties of the rock present at the Nine Mile Point site are best represented by the compressional and shear wave velocities derived from the seismic crosshole surveys. The velocity values derived from the GP-1 to GP-2 leg of the triangular configuration of boreholes are considered to be the most reliable data because GP-1 to GP-2 is the longest leg and thereby minimizes the effect of systematic error.

The borehole logging results and the uphole compressional wave velocity survey results indicate general agreement with the crosshole results. Both of these methods measured velocities of vertically traveling waves. Because of the lithologic character of the rock at the site, the velocities from these surveys are somewhat lower than those derived by the crosshole method.

Velocity anisotropy in the horizontal plane was not observed. The magnitude of the observed velocities along each leg of the triangular configuration, when considered with the range in computed values, showed no trends indicative of anisotropy. However, such could exist within the range of the observed velocities. If this were the case, the degree of anisotropy would be very limited.

2.5.4.5 Excavation and Backfill

2.5.4.5.1 Excavation

The shallow depth of encountered soil materials overlying the Oswego sandstone was removed by standard mechanical excavation methods. The underlying bedrock was removed mostly by blasting techniques with occasional small volume areas (less than 1.5 cu m[2 cu yd]) being excavated by mechanical means. The vertical rock faces of excavations for Category I structures were supported by a combination of rock dowels and welded wire fabric. Blast monitoring was performed to ensure that peak radial particle velocities were within specified limits. As excavation progressed, dewatering operations became necessary. The plant dewatering system is discussed in Section 2.5.4.6. The locations

and profiles showing the units of rock excavation for foundations of the main plant structures are discussed in Section 2.5.4.3.5 and appear in Figures 2.5-94 through 2.5-96. Overburden materials, which are not shown on these profiles, were excavated on an approximate 1.0(vertical):1.5(horizontal) slope.

All blasting operations for excavation for structures at the Unit 2 construction site are controlled by a systematic program. Methods for controlling overbreak include line drilling and controlled perimeter blasting, i.e., cushion blasting, presplitting, and smooth wall blasting. The control and monitoring of the blasting program are discussed below.

Control Criteria

Blasting control was achieved by limiting the charge per delay to that which produces specified limiting levels of ground vibration. Peak radial particle velocity is the measure of the level of vibration. Measurements were made by 3-component seismographs, such as Sprengnether VS-1100 or VS-1200. The charge weight per delay used in blasting was limited such that the peak radial particle velocity measured satisfied the most limiting of the following criteria:

1. The peak radial particle velocity measured within the existing Unit 1 facility was:

    i) At no time exceed curve 1 in Figure 2.5-96D, Peak Particle Velocity Versus Frequency, Unit 1.

    ii) Not exceed curve 2 in Figure 2.5-96D when the unit is in operation.

2. The peak radial particle velocity did not exceed 2 in/sec at a distance of 120 ft measured on the foundation material, rock, or overburden.

3. Where a structure was closer than 120 ft to the blast, the peak radial particle velocity at the structure was limited to 4.5 in/sec measured on the foundation material, rock, or overburden immediately adjacent to the limit of the structure closest to the blast.

4. When blasting was being conducted at any time 0 to 24 hr after the placing of concrete on the site, the peak particle velocity on the foundation material, rock, or overburden at the location of the fresh concrete was limited to the following:

Time	Peak Particle Velocity
0 to 11 hr	0.10 in/sec
11 to 24 hr	2.00 in/sec

Monitoring

The peak radial particle velocity was measured at the location of strategic structures, and at the location of fresh concrete for any particular blast. Each blast was monitored within the Unit 1 facility and Unit 2 area.

The peak radial particle velocity is defined as the maximum velocity scaled from the seismograph strip chart in any one of the three orthogonal axes recorded. The frequency is defined as the frequency at the maximum velocity which will be calculated from the seismograph strip chart for the half cycle which has as its center point the maximum velocity.

For the commencement of the work, the charge per delay was not allowed to exceed that determined on the basis of the envelopes shown in the log-log plots of scaled distance versus particle velocity in Figures 2.5-96A, 2.5-96B, and 2.5-96C. (Scaled distance D is defined as R, the distance in feet from the blast to the reference location, divided by $W^{1}/^{3}$, where W is the maximum charge per delay in pounds.) The data used to produce these three envelopes were obtained from the blasting done at the site of the James A. FitzPatrick Nuclear Power Plant, Power Authority of the State of New York, which is approximately 2,500 ft to the east of Unit 2. The envelope in Figure 2.5-96A represents confined presplit blasting. The envelope in Figure 2.5-96B represents confined production blasting. The envelope in Figure 2.5-96C represents the blasting done to an open face.

Blasting data were recorded and plotted. When 20 data points were accumulated on each plot, a new envelope was drawn, if necessary, such that 95 percent of all data points lay beneath the envelope. However, if during the collection of these 20 data points more than one point fell above the current envelope, the envelope was immediately adjusted to satisfy the above criterion before the next shot was made.

Release for Blasting

Prior to the release for blasting, the following items were reviewed and approved:

1. Blast holes location

2. Loading of each hole

3. Blast pattern

4. Intended time of detonation

5. Location of all structures within 120 ft of the blast location.

The use of controlled blasting methods produced very good overall results. The excavation walls were, in general, very smooth with little overbreak. Most overbreak occurred on outside corners and at the top of rock. Bedrock was left in place after blasting was thoroughly inspected. Loosened or detached blocks were removed, and safety wire mesh was attached to the rock face and secured with resin anchor bolts. In addition, rock dowels along the edge of major deep excavation areas, as shown on Figure 2.5-113, were installed to provide an additional measure of protection for personnel and heavy equipment during construction. The rock dowel installation consisted of two rows of dowels 1.5 to 5 ft back from the edge of rock cuts drilled and grouted 5 ft to 41 ft 9 in deep overall and 20 to 30 ft deep at reactor excavation, on a batter of 5° from vertical, as shown on Figure 2.5-96E and Figure 2.5-96F.

Areas of shallow excavation, where rock dowels were not needed, were also thoroughly inspected. Any loosened or detached blocks after blasting were removed. Bedrock left in place after excavation was found being reasonably intact.

2.5.4.5.2 Backfill

All major Category I structures are founded on bedrock except for a Category I electrical ductline and manhole which are founded on Category I structural fill. Category I structural fill was also placed beneath certain Category I floor slabs (Section 3.7.2.4A). In these cases, the fill material does not function as a support medium but only as a construction form. Category I structures are backfilled with either granular fill, compressible fill, or concrete. The fill is typically 4.3 m (14 ft) to 5.2 m (17 ft) thick; however, it extends down to a depth of 8.2 m (27 ft) in the area south of the reactor building and east of the control building. A thorough and systematic program of testing, inspection, and documentation was implemented during backfill operations.

<u>Structural and Granular Fill</u>

<u>Sources and Quantities</u>

The backfill materials were obtained from offsite sources following a borrow pit study in which bag samples from selected sources were obtained for the purpose of classification and grain size analysis. The source pits used to supply Category I fill materials for the Unit 2 site are shown in Table 2.5-24.

<u>Properties</u>

Prior to the placement of structural and granular fill, laboratory investigations had been conducted to determine index and mechanical properties; for Category I structural fill, the dynamic properties were also determined. These fills are classified as a well-graded sandy gravel or gravelly sand (GW-SW)

with few or no fines (ASTM D-2487). The gradation requirements of the fill are shown on Figure 2.5-102 and 2.5-102A.

Static strength parameters of Category I structural fill were obtained from consolidated-drained triaxial shear tests. Results of these tests appear in Tables 2.5-25 through 2.5-27 and Figures 2.5-129 through 2.5-132. Dynamic test results appear in Section 2.5.4.8. The average index and mechanical properties are summarized in Table 2.5-28.

Compaction and Placement Criteria

Moisture control was specified between 3 percent above and below the optimum moisture content as determined by ASTM D1557, Method D. Compaction obtained a density of not less than 95 percent of the maximum density determined by ASTM D1557, Method D, for structural fill, and not less than 90 percent of the maximum density determined by ASTM 1557, Method D, for granular fill.

For those limited areas where Category I structural fill is used as a support media (beneath an electrical ductline and manhole), limiting density tests were conducted prior to placement. Compaction requirements for structural fill and all other granular soil backfills do not include a relative density criterion. This is due to the fact that all Modified Proctor tests show a well-defined relationship between dry density and moisture content. A summary of pertinent details is as follows:

Borrow Pit: Meany Maiden Lane Pit

Total Volume: 4473 m^3 (5850 yd^3)

In-place dry density (avg): 2.16 g/cc (134.7 pcf)

Limiting Densities (ASTM D2049) - 1 test

Minimum 1.81 g/cc (113.4 pcf)

Maximum 2.19 g/cc (136.7 pcf)

Relative Density: 92.8%

Structural and granular fill were placed in loose uniform horizontal layers not exceeding 0.31 m (12 in) in thickness and compacted with at least four passes of a suitable compactor. The in-place density of the compacted fill was measured by the sand cone method and/or a nuclear testing device. The compaction for all placements of structural fill exceeds 95 percent of the average maximum dry density. The compaction for all placements of granular fill exceeds 90 percent of the maximum dry density.

Quality Control Tests

A systematic program of testing, inspection, and documentation was implemented during the backfill operations. This quality control (QC) program is summarized in Table 2.5-28A. The results of QC tests for the Category I structural fill placed below the Category I ductline and manhole are shown on Figure 2.5-102B.

2.5.4.5.2.1 Compressible Fill

Composition and Location

There are four compressible backfill materials which have been used at the Unit 2 site: vermiculite, vermiculite concrete, vermiculite-sand, Nufoam and/or Rodafoam. A description of each material is given in Section 2.5.4.10.2. Plan and typical sections showing limits and location of these materials are shown on Figure 2.5-105A to Figure 2.5-105F.

Properties

Vermiculite backfill material is vermiculite Grade No. 2 with the standard gradation requirement shown in Table 2.5-36. Vermiculite concrete backfill materials consist of one part of portland cement to eight parts of vermiculite Grade No. 4 by volume. The standard gradation requirement is shown in Table 2.5-36. The wet density of the vermiculite concrete does not exceed 62 pcf. Vermiculite-sand backfill materials consist of one part of vermiculite Grade No. 2 and three parts of concrete sand. The gradation requirement of concrete sand is shown in Table 2.5-37. The maximum dry density of vermiculite-sand does not exceed 70 pcf. Nufoam and Rodafoam are premolded cross-linked polyethylene foam which possesses a density of 2.4 pcf.

Compaction and Placement Criteria

Vermiculite backfill was poured dry into designated spaces between vertical rock faces and Category I walls. No compaction of the fill was permitted. Water was applied by lightly spraying onto the backfill to accelerate settlements. The quantity of water was carefully applied to preclude flooding of the backfill space.

Vermiculite concrete backfill was placed on water-free surfaces between foundation mats and vertical rock faces. There was no restriction for vertical drop or layer thickness. The material was cured for at least 72 hr after the mixture was placed and maintained at or above 40°F surface temperature. Each placement was protected from the weather and foot traffic as necessary.

Vermiculite sand backfill was placed as required to mitigate lateral pressures developed during design rock movements. It was placed on a pea stone base in horizontal loose lifts which did not exceed 16 in in thickness. Water was added during mixing to attain a moisture content between 16 and 20 percent. The

compactive effort consisted of 4 passes per lift with small walk-behind vibrating compactors to achieve an in-place dry density between 50 and 70 pcf. Chutes were used during placement to ensure that free-fall did not occur in excess of 6 ft.

Nufoam and Rodafoam were installed to completely fill the designed space as shown on Figure 2.5-105G. Board sections of Nufoam/Rodafoam were set together closely so that edges are in contact and debris or concrete did not penetrate the joints. Concrete placement against the foams was limited to a rate of 2 ft/hr, thereby minimizing the fluid pressure on the foams.

Quality Control Tests

All compressible backfill placement was subjected to a systematic QC program as summarized in Tables 2.5-38 through 2.5-41. Results of QC tests meet specified requirements.

2.5.4.5.2.2 Concrete

Lean and porous concrete was used beneath and around seismic Category I structures. These fills are QA Category I mixes designated as mix "D" and "E", respectively. The minimum allowable compressive strength at 28 days is 1000 psi. Actual strength of all concrete placements exceeded 1000 psi.

Lean concrete was used beneath certain structures to level excavated rock surfaces. Porous concrete was used under the reactor building mat as part of the dewatering system. Mixing, delivering, placing, and testing of these mixes were performed in accordance with ACI-301, Specifications for Structural Concrete for Buildings, and is described in Section 3.8.4.6.

2.5.4.6 Groundwater Conditions

2.5.4.6.1 Groundwater Effects on Foundation Integrity

All major Category I structures are founded on sound, flat-lying, consolidated sedimentary rocks of low to very low porosity and permeability (Section 2.4.13). Consequently, groundwater conditions have little or no influence on the foundation integrity of the safety-related facilities. Critical engineering properties of the foundation material, including effects of moisture variations, are discussed in Section 2.5.4.2.

2.5.4.6.2 Groundwater Distribution and Regime

The groundwater distribution and regime were investigated in the plant complex area by monitoring water levels in piezometers, and by appropriate observations and tests in excavations. The groundwater distribution in bedrock is primarily governed by the arrangement of lithologies within a stratified hydrologic system composed of alternate water-bearing and water-confining units (Section 2.4.13). This system is divided into two parts of

unequal thickness, which are characterized by specific hydrologic properties:

1. A thin top portion that maintains a full hydraulic connection with the overlying porous soils and the ground surface, and thus belongs to the groundwater-table (unconfined) aquifer.

2. A thick lower portion containing numerous impermeable zones intercalated with hydraulically isolated permeable zones that form a series of confined to semiconfined aquifers.

Hence, the groundwater table aquifer comprises the uppermost bedrock interval together with the immediately overlying soil material of the overburden veneer (Section 2.4.13.1.2). This bedrock interval is generally 1.5 to 6 m (5 to 20 ft) thick[82], and consists mostly of thick-bedded and erosion-resistant sandstone that is colloquially called the cap-rock in reference to its appearance at the top of bedrock. The subcropping sandstone coincides with the weathered zone of bedrock marked by bedding plane dilation, wide opening of joints, and sporadic soil filling in voids. It has very pronounced secondary porosity and relatively high conductivity, both of which are entirely attributed to discontinuities. The overlying soil material, however, possesses primary porosity and relatively low permeability. The calculated values of permeability for soil and rock materials are presented in Section 2.4.13.2.2.

A series of confined to semiconfined aquifers encompasses innumerable individual or closely associated sandstone beds. These beds are situated between and hydraulically separated from each other by siltstone and/or shale which are finer grained and generally nonjointed to infrequently or incompletely jointed lithologies acting as aquitards or aquicludes. The sandstone aquifers are bounded by bedding plane joints and are systematically intersected by two major sets of subvertical joints striking east-northeast and northwest. All of these joints function as conduits by permitting groundwater flow, and as storage by providing space for groundwater accumulation. Joint frequencies are inversely proportional to the bed thickness[91]. As a result, thin-bedded aquifer units are more porous and permeable than thick-bedded units. Joint openings decrease with depth and, therefore, the deeper aquifers are less porous and less permeable than their near-surface counterparts.

The spatial arrangement of and relations among confined aquifers are intricate owing to such factors as the lenticular shape of beds, the grading from one lithology to another in all directions, and the presence of faults (Section 2.4.13). Various bedrock lithologies actually form an assemblage of lenses rather than continuous layers, resulting in lateral limitations of individual aquifers, in most cases at very short distances. High-angle strike-slip faults and low-angle thrust faults, on the

other hand, have created local hydraulic connection among stacked water-bearing sandstone lenses.

Groundwater conditions in the plant area are studied and defined on the basis of the following determinations.

<u>Seepage in Excavations</u> It has been noted throughout excavations, particularly for the screenwell shaft, the lake water tunnels, and the reactor foundation, that groundwater seepage from the rock walls occurred either along horizontal bedding planes at various levels or along the inclined fracture surfaces. The most conspicuous seepage from a bedding plane was reported for the screenwell shaft walls around el 62 m (203 ft). All intersections between rock walls and fractures were accompanied by water seepage. The most conspicuous seepage of this type was observed in the west reactor arc around el 52 m (170 ft) and in take-water Tunnel No. 2 approximately 308 m (1,010 ft) from the portal.

<u>Artesian Flow from Boreholes</u> During excavations for the reactor foundation and the screenwell shaft, a number of vertical boreholes, mostly drilled for the purpose of blasting, exhibited short-lasting artesian flow at various levels ranging approximately from el 50 m (163 ft) at the reactor rock floor, where water was flowing from subdrilled blastholes, to el 71 m (234 ft) at the top of the second lift in the reactor excavation, where water was flowing out of a temporarily installed heave-monument borehole (Section 2.5.4.13). These observations indicated the presence of confining beds separating a series of stacked individual aquifers.

<u>Geologic Mapping Data on Jointing</u> In the course of geologic mapping of excavations, it was noted throughout the plant complex that only sandstone is systematically jointed, with frequencies becoming greater in thinner beds, creating favorable conditions for groundwater flow and accumulation. In contrast, siltstone and shale have not been jointed to the extent of becoming permeable, except in the fault zones, and therefore behave as water-confining horizons.

<u>Water Levels in Piezometers</u> Two sets of piezometers have been installed in the plant complex area at different times for different purposes, with the intention of keeping them operative until the end of construction in 1985. One set includes standpipes, and the other consists of pneumatic types. The piezometer locations and other related information are presented on Figure 2.5-103.

The first set includes a group of four standpipes installed to the north and south of the reactor excavation during spring and summer of 1978. An additional two standpipes were installed in the 20- to 30-cm (8- to 12-in) rock slot about 11 m (35 ft) west of the 12-line wall; these were operative only for one month due to construction interference. The prime objective of these

piezometers was to establish the phreatic surface in the surrounding bedrock. Readings of groundwater levels are compiled in Table 2.5-29.

The second set includes a group of five pneumatic piezometers around the screenwell shaft (four of which were installed at the end of 1979 and one at the end of 1981), and a group of two additional pneumatic piezometers installed east of the reactor excavation at the end of 1981. Each piezometer of the former group contains two separate sensors at different depths, exposed to rock sections 0.6 to 2.1 m (2 to 7 ft) long, whereas each of the latter group has only one sensor within an interval 2.4 to 4 m (8 to 13 ft) long. These piezometers are a part of the instrumentation program designed to monitor rock displacements (Section 2.5.4.13.9). The results of the groundwater monitoring (Table 2.5-30) provide convincing evidence that aquifers at various depths have different hydraulic heads, indicating the existence of confining beds between these aquifers.

The groundwater regime in both the unconfined and confined aquifers, defined by flow direction, gradients, velocities, recharge and discharge, etc., are discussed in Section 2.4.13.

2.5.4.6.3 Dewatering

Water seepage into excavations has been moderate and within the expected range, never causing obstacles to construction. During the plant excavation and construction, dewatering was achieved by pumping water out of the screenwell shaft and reactor foundation sumps at rates dictated by the amount of seepage. Other structures have shallower foundations that have been naturally drained into these two deep excavations, with or without temporary pumping. It is estimated that about 13 l/sec (200 gpm) is an average flow rate into the reactor excavation, and approximately one-half of that flows into the screenwell shaft. Considering the relatively large depth and surface area of rock faces exposed in excavations, these flow rates indicate a rather low average permeability of bedrock.

During the plant operational life, dewatering of the reactor foundation will be continuously maintained, although it does not have any safety-related function. This nonsafety-related permanent dewatering system will be inspected as part of the normal daily tours during the life of the plant. The Operator will verify that the pump is operational.

In the event the dewatering system is not in operation, the groundwater table would rise to its normal elevation. The magnitude of the swell of the bedrock caused by this rise in the water table at a point is determined by multiplying the rate of strain by the difference in elevation between the top of rock and the drawdown curve (groundwater contours) which is shown on Figure 2.5-112 at that point. Additionally, even if the dewatering system were nonfunctional, Category I plant structures

are designed for a vertical strain rate of 0.005 percent, as explained in Section 2.5.4.1.4.

This nonsafety-related dewatering system at the reactor building (as shown on Figures 2.5-133 and 2.5-134) consists of an above-mat drainage zone and a below-mat drainage layer. Each was constructed of porous concrete with embedded porous concrete pipes installed to direct the flow of water to the sumps. Two sumps are provided, each containing submersible pumps to pump the collected water out of the sump and into the storm sewer. In the event of a failure of one of the pumps, the pump in the second sump will be able to pump the collected water. Each sump and each vertical access shaft connecting the sumps to the surface consists of concrete pipes.

2.5.4.6.4 Permeability Determinations

Permeabilities of the foundation materials are estimated on the basis of recent studies performed at the Unit 2 site, and previous studies conducted at the adjacent Unit 1 and James A. FitzPatrick sites[82]. The permeability studies for Unit 2 consisted of the following:

1. Groundwater level measurements in 13 piezometers and in all exploratory borings penetrating bedrock.
2. Three pressure tests for 3 m (10 ft) long successive bedrock intervals in exploratory borings 408, 410, and 419.
3. Seepage measurements in the screenwell shaft and reactor excavations.

The permeability studies for the Unit 1 and James A. FitzPatrick sites included the following field and laboratory tests:

1. Groundwater level survey in all exploratory borings penetrated into bedrock.
2. One pumping test in exploratory boring 108 converted into a pumping well (with three surrounding borings utilized as observation wells) at the Unit 1 site.
3. Eleven field percolation tests in overburden soils.
4. Five laboratory tests of undisturbed soil samples.

The permeability values of bedrock and overburden soil materials derived from these field and laboratory tests are presented in Section 2.4.13.2.2.

2.5.4.6.5 Fluctuation and Monitoring of Groundwater Levels

The original piezometric heads in the confined aquifers at the site were slightly above the groundwater table (Section 2.4.13.2.2). Postexcavation hydraulic heads have been lowered by dewatering as recorded by sensors of the pneumatic piezometers, which also show that these artesian heads are still considerably above the upper boundaries of their aquifers. Considering the relatively small size and low storage volume of individual aquifers along with their continuous drainage into excavations, resulting in reservoir depletion, it is expected that the piezometric heads of these aquifers will further decrease, provided that the dewatering of excavations is maintained. No further monitoring is planned upon completion of construction.

The groundwater table at the site, including the plant complex area, has been defined by water levels measured in the exploratory borings drilled prior to the commencement of excavation, and in the piezometers installed after completion of the reactor excavation and most of the screenwell shaft excavation. The original groundwater table in the plant complex area was approximately at el 76 m (248 ft) (Section 2.4.13.2.1). However, dewatering of excavations lowered the groundwater table substantially and changed the pattern of flow by diverting it toward the excavation.

The maximum seasonal variation of the groundwater table observed during the 1978-1981 monitoring period was 2.3 m (7.6 ft) (Table 2.5-29) reported for standpipe piezometer P-4, located within the cone of depression created by dewatering of the reactor excavation. If the natural groundwater conditions were restored, it is estimated that the maximum long-term fluctuation would reach 3.4 m (11 ft), with the groundwater table ranging from a low level at el 74 m (244 ft) to a high level at el 78 m (255 ft). The estimated flood design level is at el 80 m (261 ft) which is also the Station grade level (Section 2.4).

2.5.4.7 Response of Soil and Rock to Dynamic Loading

2.5.4.7.1 Response of Rock to Dynamic Loading

The site is underlain by sedimentary rock formations consisting of interbedded sandstones, siltstones, and shales (Section 2.5.1). All major Category I structures are founded on bedrock.

The dynamic properties of the rock mass are based on geophysical surveys that included both uphole/downhole and crosshole seismic surveys that measured shear as well as compression wave velocities to a depth of 107 m (350 ft) into the rock mass (Section 2.5.4.4). The seismic tests involved strain levels on the order of 10^{-3} to 10^{-4} percent which are higher than those imposed by any dynamic loading, including the SSE; thus, the behavior of the rock mass during the seismic surveys should provide a conservative estimate of its response to dynamic loadings. As indicated in Section 3.7A.2.4.1, dynamic analyses for structures were performed using fixed base models because

shear wave velocities in the bedrock exceeded 1,070 m/sec (3,500 ft/sec). Thus, there is no need to establish a relationship between shear modulus (or damping ratio) and strain level.

The following elastic moduli were used in evaluating the dynamic response of rock (Section 3.7A.2):

G = Shear modulus = 1.5×10^7 kN/m^2 (2.1×10^6 psi)

E = Young's modulus = 3.5×10^7 kN/m^2 (5.0×10^6 psi)

The moduli are conservative values based on data from the geophysical surveys cited above. The values reflect the increase in E and G with depth as well as the anisotropic character of the rock mass.

Section 3.7A.2.4 discusses soil/structure interaction effects.

2.5.4.7.2 Response of Soil to Dynamic Loading

Electrical ductline 907, part of ductline 922, and manhole no. 1, Cl-IE, are the only Category I structures founded on Category I structural fill (Section 2.5.4.5.2). The amplification of ground motion by the backfill has been determined by using a computer program entitled SHAKE[280] that computes the responses in a system of homogeneous, viscoelastic soil/rock layers of infinite horizontal extent subjected to vertically traveling shear waves. The program is based on the continuous solution to the wave equation adapted for use with transient motions through the fast Fourier transform algorithm. An iterative procedure is used to obtain values of shear modulus and damping that are compatible with the effective strain in each layer according to the input soil properties (Figure 2.5-150). Acceleration time-histories of three previous earthquakes recorded at rock sites[281] are input as base motions: 1) 1935 Helena, Montana, earthquake, 2) 1971 San Fernando, California, earthquake at Santa Anita Dam, and 3) 1971 San Fernando, California, earthquake at Lake Hughes Array Station 4. The maximum acceleration of each selected earthquake record is close to the Unit 2 design SSE of 0.15 g and is considered suitable for this seismic response analysis.

To be conservative, the shear moduli were determined by the Seed and Idriss relationship[282] for sand with 75 percent relative density (Figure 2.5-151). The responses of the input ground motions at the foundation grade of each structure indicate thatthe maximum horizontal acceleration and displacement are 0.25 g and 0.013 in, respectively. The envelope of maximum responses for each input earthquake is shown on Figure 2.5-152 and Table 2.5-45. The plan and typical section of these buried structures are shown on Figure 2.5-153.

The ductlines and manhole have been evaluated for these amplified seismic motions, as described in Section 3.7A.2.4.

2.5.4.8 Liquefaction Potential

All major Category I structures are founded on bedrock or concrete fill. The Category I structural fill is limited to small areas that are bounded by foundation walls resting on bedrock and the fill functions only as a form during construction.

Electrical ductline 907, a Category I structure that extends eastward from the control building, is supported on Category I structural fill. The ductline terminates at manhole no. 1 CL-IE, also supported on Category I structural fill. Electrical ductline 922, which extends northward from manhole no. 1 and terminates at the reactor building, is also a Category I structure and is partially supported on Category I structural fill.

The depth of the fill cited in the preceding paragraphs varies from 2.7 to 5.5 m (9 to 18 ft). Most of the Category I structural fill lies below the normal groundwater level (el 78 m [255 ft]).

The material and placement requirements for Category I structural fill are discussed in Section 2.5.4.5. The compaction criterion (Section 2.5.4.5) ensures that Category I structural fill will have a relative density of at least 70 percent.

The liquefaction potential of Category I structural fill during the SSE was analyzed according to the technique proposed by Seed and Idriss[207].

The purpose of the analysis is to determine the ratio of the cyclic shear stress required to cause liquefaction to the shear stress induced by the SSE. This ratio yields the factor of safety against liquefaction at a particular depth and is defined as follows:

$$\text{Safety factor} = \frac{\tau}{\tau_{avg}}$$

where:

τ = Cyclic shear stress required to cause liquefaction

τ_{avg} = Average cyclic shear stress induced by the SSE The results of the analysis, including input parameters, are shown in Table 2.5-44.

The cyclic shear stress required to cause liquefaction is based on cyclic triaxial tests which were conducted on representative material from two borrow sources[208,209]. Results of these tests appear in Tables 2.5-31 and 2.5-32, and Figures 2.5-104 and

2.5-105. Cyclic tests were not repeated for each new source; however, the engineering properties and geologic characteristics of the new sources were examined to ensure that the new borrow sources were comparable to those tested. The cyclic shear stress required to cause liquefaction was determined in accordance with the following relationship:

$$\tau/\bar{\sigma}_{\upsilon} = \left(\sigma_d / 2\bar{\sigma}_c\right) C_R$$

where:

τ = Cyclic shear stress required to cause liquefaction

$\bar{\sigma}_{\upsilon}$ = Effective vertical stress at depth of interest

σ_d = Laboratory cyclic deviator stress

$\bar{\sigma}_c$ = Initial ambient pressure under which the sample was consolidated

C_R = Correction factor applied to laboratory triaxial data to model field conditions

Various conservative parameters are used to determine $\tau/\bar{\sigma}_{\upsilon}$. These are as follows:

1. Initial liquefaction is defined as the point at which samples experienced a 2.5 percent double amplitude strain.
2. The value of relative density, D_r, is considered to be 70 percent. Actual field relative density may well exceed 90 percent.

The cyclic shear stress induced by the SSE is based upon field density results (see Section 2.5.4.5 and Figure 2.5-102B) and the following equation:

$$^{\tau}avg = 0.65\ x \frac{^{\gamma}T^{h}}{g}\ x\ A_{\max}\ x\ \gamma_d$$

where:

τ_{avg} = Average shear stress induced by the SSE

γ_T = Total unit weight of the soil

h = Depth of interest

A_{max} = Maximum ground acceleration = 0.15 g

γ_d = Reduction factor based on depth to account for soil deformability

The groundwater level is taken to be at el 255 ft 0 in (see Section 2.4.13.5). The structural fill below the ductlines and manhole is considered to have a total unit weight of 145 pcf.

The design criteria for liquefaction is discussed in Section 2.5.4.11. Analyses indicate that Category I structural fill has an adequate factor of safety against liquefaction.

2.5.4.9 Earthquake Design Basis

The selection of the SSE and the OBE is discussed in detail in Sections 2.5.2.6 and 2.5.2.7. The SSE is based on a MMI VI earthquake. The maximum horizontal bedrock acceleration for such an earthquake would be less than 10 percent of gravity. All Category I structures are conservatively designed for a SSE with a horizontal acceleration of 15 percent of gravity. Vertical motions are scaled to the horizontal motions in accordance with RG 1.60.

As indicated in Table 3.7A-2, all major Category I structures are founded on rock. Some Category I ductlines and floor slabs are supported on compacted structural fill (Section 2.5.4.8). The liquefaction potential and seismic response of the fill are based on the SSE described above.

2.5.4.10 Static Stability

The fine-grained, dense, and siliceous Oswego sandstone has a relatively high unconfined compressive strength. The principal bedding planes are practically horizontal, and consequently foundation loads are applied essentially perpendicular to bedding. Thus, foundation bearing conditions are most satisfactory[258]. Bedrock strength properties are discussed in Section 2.5.4.2.

2.5.4.10.1 Bearing Capacity and Settlement

Major Category I structures are founded on bedrock. The allowable bedrock bearing capacities used for design are 958 kn/sq m (10 tons/sq ft) for foundations within the top 3 m (10 ft) of bedrock and 1,915 kn/sq m (20 tons/sq ft) for foundations deeper than 10 ft below the bedrock surface. The possibility of settlement of the safety-related structures due to the potential closure of bedrock dilation (Section 2.5.4.1) under structure loads is of no stability concern. However, since the radwaste building is formed near the ramp of the thrust structure, a differential settlement of 0.254 cm (0.1 in) was used for design to be conservative.

Certain Category I structures such as electrical ductlines and manholes are supported by Category I structural fill (Section 2.5.4.5). The allowable soil bearing pressure is 191.6 kn/sq m (4,000 psf). Static settlements are essentially elastic and occur almost immediately upon application of the load. The maximum anticipated settlements are negligible due to the small dead loads associated with the aforementioned structures and the high density of the fill.

2.5.4.10.2 Lateral Soil Pressure

Backfills placed adjacent to underground structural walls impose lateral pressures on those walls. Backfills used include sand and gravel backfill, loose vermiculite, vermiculite concrete, Nufoam/Rodofoam and vermiculite sand. All of these materials are discussed in this section. The magnitude of the lateral pressure on any particular wall depends primarily upon the deformability/strength of its adjacent backfill(s), the groundwater conditions and the rock squeeze/fault movement design requirements for that wall.

Rock squeeze/fault movement design criteria are presented in Section 2.5.4.11. As discussed in that section, horizontal rock squeeze was modeled as a continuous irreversible process in which the rock mass displaces toward the reactor excavation. The design fault movement is a 1.00 in irreversible displacement of the radwaste fault along its strike, i.e., N70°W. Where applicable, the two displacements were added vectorially; however, fault movement was ignored along those walls on which the movement would reduce lateral pressures. In summary, the backfills may be subjected to monotonically increasing compressive strains. All of the backfills were placed according to a thorough and systematic program of testing, inspection, and documentation that ensured the backfills can accommodate the compressive strains without imposing excessive lateral pressures on Category I structural walls.

Some of the backfills listed above were used below rock grade whereas others were used above rock grade.

Figures 2.5-105A through 2.5-105F show the lateral extent of the backfills.

Figures 2.5-105G and 2.5-105H, typical cross sections along the southeast arc of the secondary containment wall and the south wall of the south auxiliary bay, show the vertical limits of the various backfills. Figure 2.5-105J shows the plan view and a typical cross section along the underground exhaust tunnel to the main stack. Each of the backfills and its attendant lateral stresses is discussed below.

Pressure Below Rock Grade

Gaps are provided between all Category I exterior walls/mats and vertical rock faces. These gaps range in width between 15.2 cm (6 in) and 30.5 cm (12 in) and are filled with a variety of compressible materials that minimize the load transfer to building walls due to horizontal rock displacements (Section 2.5.4.11). As a result, there are small lateral pressures acting on walls below rock grade. These compressible materials are as follows:

1. Loose vermiculite.

2. Vermiculite concrete.

3. Nufoam and/or Rodofoam.

These are individually discussed in the following paragraphs:

Vermiculite is a highly compressible micaceous mineral. It was placed above foundation mats in the gap between the vertical rock faces and the exterior walls (Figure 2.5-105H). The loose vermiculite was dumped into the gaps which were subsequently sealed at rock grade with flashing to ensure the loose vermiculite was not exposed to any surcharges. Thus, the vermiculite remained in a virgin state and settled solely under its own weight. Lateral pressures due to rock squeeze and fault movement were computed based on results of one-dimensional, incremental consolidation tests. A typical compression curve is shown on Figure 2.5-106.

Vermiculite concrete is a mix having a proportion of 1 to 8 by volume of cement to vermiculite. It was used as the compressible material adjacent to foundation mats (Figures 2.5-105G and 2.5-105H). All vermiculite used in the mix had a dry unit weight of between 0.096 and 0.16 g/cc (6 and 10 pcf). One-dimensional compression tests were conducted for use in predicting lateral loads transferred to structures due to horizontal rock movements. A summary of 13 one-dimensional confined compression tests performed on samples taken from different placement sections at the site is presented on Figure 2.5-107.

Nufoam is a cross-linked polyethylene foam that was placed as an alternate to vermiculite principally for construction convenience (Figures 2.5-105A through 2.5-105F). As demonstrated by the compressive stress-strain curve (Figure 2.5-108), the Nufoam is also highly compressible. Rodofoam is a cross-linked polyethylene foam similar to Nufoam that is also used for construction convenience. Its compressive stress-strain curve appears on Figure 2.5-109.

<u>Pressure Above Rock Grade</u>

The components of lateral pressures on Category I exterior walls due to compacted backfill above rock grade were computed as shown

on Figure 2.5-110. Each of the components is discussed as follows:

Static Soil Pressure The K_o value for structural fill shown on Figure 2.5-110 is based on Jaky's[276] formula $K_o = 1-\sin\phi$, where ϕ is the angle of internal friction. Table 2.5-28 indicates that the design ϕ for structural fill is 40°; however, a conservative value of 35° was used in Jaky's formula to yield a K_o of 0.43. Compaction induced stresses were evaluated using the method presented by D'Appolonia et al[277]. These stresses fell essentially within the static soil pressure diagram defined by K_o, as shown on Figure 2.5-110.

Static Water Pressure Three different groundwater conditions were considered in calculating lateral pressures. Flood groundwater level (el 261 ft) and normal groundwater level (el 255 ft) are discussed in Section 2.4.13.5. A third condition, low groundwater level, was examined because of the fact that a dewatering system (nonsafety related) will be in operation over the life of the plant. The location of the parabolic drawdown curve is based on available piezometric data (Section 2.5.4.6) and a conservatively assumed water entry level of el 164 ft at the reactor building. Approximate groundwater contours for the dewatering condition are shown on Figure 2.5-112. Total lateral stresses for all three groundwater conditions were calculated in accordance with Figure 2.5-110. The most conservative of the three distributions was used in design.

Static Surcharge Pressure The static surcharge pressure is based on a uniform vertical loading of 19.8 kn/m^2 (400 psf) at ground surface. The lateral surcharge pressure is assumed to be uniform with depth.

Soil Pressure Due to Horizontal Rock Displacement As previously indicated, all Category I foundation walls are designed to withstand horizontal bedrock displacements due to rock squeeze and fault movement. The magnitude of the design displacement for any particular wall depends upon its location and orientation. Each Category I wall was examined to determine its design displacement with respect to the criteria presented in Section 2.5.4.11. In general, the sand/gravel backfill is modeled as a semirigid mass that experiences the same horizontal displacements as the bedrock surface. For lateral stress computations, the bedrock movement is assumed to increase the effective lateral stress coefficient from a K_o value of 0.43 to a K value of 3.00; hence, the K_R of 3.00 - 0.43 = 2.57 is shown on Figure 2.5-110. The coefficient of 3.00 represents an intermediate value between at rest and passive earth pressure conditions. The design displacement for some Category I walls is 1.3 cm (0.5 in) or less. The yield ratio (i.e., displacement/wall height) vs lateral pressure relations for such walls indicates that the final lateral stress coefficient after displacement will be less than 3.0. Some Category I walls, in particular the southeast arc

of the secondary containment, have design displacements of almost 5 cm (2 in). Yield ratios for these walls would produce lateral pressures with coefficients far greater than 3.0. These walls are protected by an adjacent zone of compressible material that can absorb the rock displacements with a minimal stress transfer to the walls. As shown on Figure 2.5-105G, the compressible zone consists of a 152-cm (60-in) (minimum) wide layer of compacted vermiculite sand that extends from the top of rock to essentially yard grade. (The sand/gravel backfill between el 261 ft and el 259 ft serves as a protective layer for the vermiculite sand.) Vermiculite sand is a mix of three parts vermiculite to one part sand by volume. The in-place dry unit weight is between 0.80 and 1.12 g/cc (50 and 70 pcf). One-dimensional compression test results are shown on Figure 2.5-111. The walls are designed for a K_R in the sand/gravel backfill of 2.57. As the lateral stress increases, the vermiculite sand will deform in a manner similar to Figure 2.5-111. The 152-cm (60-in) minimum width was chosen so that the strain that occurs during K_R will be at least 5 cm (2 in). For example, at el 245 ft (assumed top of rock) K_R of 2.57 corresponds to a horizontal stress increase from 66.0 kn/m^2 (1.33 ksf) to 352.2 kn/m^2 (7.09 ksf). Figure 2.5-111 indicates that the vermiculite sand should strain approximately 3.8 percent as the stress changes from 66.0 kn/m^2 to 352.2 kn/m^2. A 3.8 percent strain corresponds to 0.90 cm (2.28 in) of deformation in a 152-cm (60-in) layer.

<u>Dynamic Soil Pressure</u> Dynamic soil pressures are based on the Mononobe-Okabe approach as presented by Seed and Whitman[278]. Mononobe-Okabe based their method on active conditions; however, the model tests cited by Seed and Whitman to verify the Mononobe-Okabe approach were conducted on nonyielding structures. Thus, empirical evidence suggests that the Mononobe-Okabe method may be applicable to rigid structures. A conservative assessment of the input parameters was used in developing the dynamic soil pressure distribution shown on Figure 2.5-110. The distribution is based on a ϕ of 35°, whereas Figure 2.5-132 indicates that the actual ϕ is at least 40°. Figure 2.5-105I appears in Seed and Whitman's paper and it indicates that for ϕ = 35° and k_h, the horizontal seismic coefficient, = 0.15, the dynamic increment coefficient should be 0.08. As shown on Figure 2.5-100, a value of (3/4)(0.15) = 0.11 was used for design.

Dynamic soil pressure did not warrant an intensive analysis because it typically contributes only a small fraction of the total lateral force on foundation walls. The fraction is particularly small for those walls that must be designed for substantial rock displacements caused by rock squeeze and fault movement (Section 2.5.4.11).

For example, consider the east wall of the secondary containment. Dynamic soil pressures constitute less than 5 percent of the total lateral force. The lateral force due to rock movement

alone is 13 times greater than the force due to dynamic soil pressure.

Dynamic Water Pressure Dynamic water pressures are based on the procedure presented by Westergaard[279]. The dynamic water pressure distribution shown on Figure 2.5-110 is conservative in that the Westergaard solution assumes a freestanding body of water and hence neglects the effect of the soil skeleton.

Dynamic Surcharge Pressure As shown on Figure 2.5-110, dynamic surcharge pressure is treated as a uniform lateral load with magnitude $k_h(K_o)q$, where q is the uniform vertical surcharge at the ground surface.

Lateral seismic pressures include dynamic soil pressure, dynamic surcharge pressure, and rock displacement pressure. The latter involves a component due to fault movement (Section 2.5.1.2.3). The evaluation of the seismic response of the fill did not include any displacement calculations (other than fault movement) because no major Category I structures are founded on fill. Section 2.5.4.8 describes two Category I structures, electrical ductline 907, and manhole no. 1 CL-IE, which are founded on Category I structural fill. The lateral displacement that develops between these two structures during a SSE event was conservatively calculated as 0.04 cm (0.10 in).

2.5.4.11 Design Criteria

The criteria and methods used in the design of Category I structures are discussed below. These criteria are shown in Table 2.5-42 together with additional design parameters used in the analysis of Category I structures.

Lateral Pressure

Exterior subsurface walls subject to lateral soil loads were designed according to Figure 2.5-110.

Liquefaction

The minimum factor of safety against liquefaction is 1.5. Factor of safety is defined as the ratio of the cyclic shear stress required to cause liquefaction to the shear stress developed during dynamic loading (Section 2.5.4.8).

Bearing Capacity

The allowable bearing capacity for Category I structural fill is 191.6 kn/sq m (4,000 psf). The allowable bearing capacities for rock are 958 and 1,915 kn/sq m (10 and 20 ton/sq ft) (Section 2.5.4.10).

Settlement

With the exception of the radwaste building (Section 2.5.4.10), there are no criteria for settlement since most loads on structural fill are insufficient to cause appreciable settlement and all major Category I structures are founded on bedrock.

Rock Movement

All Category I structures were designed to accommodate the effects of the following rock movements.

1. Radwaste Fault Movement

   All Category I structures were designed to withstand the bedrock displacements and attendant forces that could develop due to the postulated movement of the Radwaste Fault. The geologic interpretation of the Radwaste Fault is discussed in Section 2.5.1.2.3. The geologic cross sections are shown on Figures 2.5-62 through 2.5-65. This section summarizes the general nature, orientation, and projected movement of the fault and describes the engineering implications of the movement and the design criteria for each Category I structure listed in Table 2.5-43.

   A cross section of the Radwaste Fault Zone is shown on Figure 2.5-64. The average dip of the shear zone is approximately 15 deg. This zone appeared in several building excavations and in a number of the 800 series borings (Section 2.5.4.3.3). Figure 2.5-136 summarizes the results of the field observations. The fault is essentially horizontal at the reactor building. It ramps upward just west of that building and outcrops along an irregular trace that extends from the screenwell area to the normal switchgear area.

   Section 2.5.1.2.3 examines the kinematics of the Radwaste Fault and discusses a numerical modeling of the fault (Figure 2.5-68) which concluded that 2.54 cm (1.0 in) is a conservative estimate of the fault's future movement. The movement would be monotonic in a N70°W direction.

   Criteria for Structural Design

   Figure 2.5-136 shows the boundary contours of the ramp of the fault at el 190 ft on the east and el 235 ft on the west. The boundaries also define the three zones of fault movement for structural design.

   Zone A encompasses all the area west of where the fault outcrops. This zone is unaffected by the fault movement and, hence, has design motions of zero in both the horizontal and vertical directions.

   Zone B is east of the el 190 ft contour, i.e., the bottom of the ramp. The fault is essentially horizontal in this zone

and the design motions are 2.54 cm (1.0 in) in the horizontal direction and zero in the vertical direction.

Zone C includes the entire ramp. The horizontal design motion is 2.54 cm (1.0 in) while the vertical design motion varies from zero at the eastern boundary to 0.76 cm (0.3 in) at the western boundary.

Figure 2.5-137 illustrates four typical building configurations relative to fault location.

Type 1 structures have foundations which are entirely above the fault zone.

Type 2 structures are situated near the area where the fault outcrops - the structures straddle the boundary between Zones A and B. The fault intersects either the east wall of the structure (Type 2A) or the foundation of the structure (Type 2B).

Type 3 structures have foundations below the fault and the fault intersects both the east and west walls of the structure.

In general, Type 1 structures would move as a rigid body with the fault and would not be affected by the fault movement. There is no relative motion between the foundation and underlying bedrock in Types 1, 2A, and 3 structures. Type 2B structures have foundations which would be subjected to shear displacements during fault movement.

The Radwaste Fault design criteria for the foundation of each Category I structure listed in Table 2.5-43 is discussed below. The locations of the cross sections are shown on Figure 2.5-138. These criteria are solely for foundation design.

- Control Building (Figure 2.5-139) - Lowest foundation elevation: 201.5 ft; Type 1 structure. No special provisions for Radwaste Fault Movement (RFM) required.

- Diesel Generator Building (Figure 2.5-140) - Lowest foundation elevation: 242.0 ft; Type 1 structure. No special provisions for RFM required.

- North Electrical Tunnel (Figure 2.5-141) - Lowest foundation elevation: 211.5 ft; Type 1 structure. No special provisions for RFM required.

- South Electrical Tunnel (Figure 2.5-142) - Lowest foundation elevation: 211.5 ft; Type 1 structure. No special provisions for RFM required.

- Main Stack and Underground Exhaust Tunnel (Figure 2.5-143) - Lowest foundation elevation: 230.0 ft; Type 1 structures. No special provisions for RFM required.

- Reactor Building and Auxiliary Bays (Figure 2.5-144) - Lowest foundation elevation: 145.5 ft; Type 3 structures. No special provisions for RFM required.

- Screenwell Building and Service Water Tunnel (Figure 2.5-145) - Lowest foundation elevation: 215.0 ft; Type 2A structures. Structures have been evaluated for 1.0 in horizontal and 0.3 in vertical RFM, and it has been determined that there will be no detrimental effect on the structures.

- Standby Gas Treatment Building and Railroad Access Lock (Figure 2.5-146) - Lowest foundation elevation: 234.0 ft; Type 1 structures. No special provisions for RFM required.

- Electrical Ductline 907 (Figure 2.5-147) - Structure is founded on Category I structural fill in Zone A. No special provision for RFM required.

- Intake Structures and Tunnels (Figures 1.2-29 and 1.2-30) - Structures are not affected by RFM.

- Auxiliary Services Building (Figure 2.5-148) - Lowest foundation elevation: 206.5 ft; Type 1 structure. No special provisions for RFM required.

- Electrical Bay Including Service Water Tunnel (Figure 2.5-149) - Lowest foundation elevation: 245.0 ft. Structures are partially supported by the circulating water pipe encasement which is a Type 2B structure. The structures have been evaluated for 1.0 in horizontal and 0.25 in vertical RFM, and it has been determined that there will be no detrimental effect on the structures.

- Radwaste fault movement also may affect the lateral pressures on some buildings. In general, the foundation walls for Category I buildings are protected by layers of compressible material as discussed in Section 2.5.4.10.2.

2. <u>Other Fault Movement</u>

Postulated future movements along the thrust structures in the screenwell shaft and lake water tunnels are discussed in the response to NRC Question F231.15 in Appendix 2N and are estimated not to exceed a fraction of an inch (Section 2.5.1.2.3). However, to be conservative, maximum movements of 0.64 cm (1/4 in) along the thrust structures in the

screenwell shaft and lake water tunnels were implemented in the design.

3. Rock Squeeze A set of conservative rock squeeze values was adopted in the design basis. The following values include any rock swell effects that may occur upon onsite rewatering (Section 2.5.4.1.4):

   a. A maximum allowance of 2.54 cm (1 in) inward movement of excavated rock walls.

   b. Maximum horizontal extentional strains of 0.02 percent at the face of the reactor excavation, decaying to lower values away from the excavation wall.

   c. Maximum shear strains of 0.04 percent in the corners of the reactor excavation.

   d. Maximum vertical strains of 0.005 percent, applicable to rock above the groundwater drawdown levels.

4. 12-Line Wall Rock Displacement The 12-line wall was analyzed for the time-dependent rock displacements described in Section 2.5.4.1.4. The wall will safely sustain the loading caused by the rock displacements in combination with the service loadings.

Slope Stability

See Section 2.5.5.

2.5.4.12 Techniques to Improve Subsurface Conditions

Rock dowels were installed as temporary support of vertical rock faces during construction. Figure 2.5-113 indicates the areas where rock dowels were installed. No grouting work was performed.

For those Category I structures supported by Category I structural fill, such as electrical ductlines and manholes, the compaction criteria (Section 2.5.4.5) are sufficient to provide a stable subsurface condition.

2.5.4.13 Subsurface Instrumentation

Several types of instrumentation for the surveillance of foundations have been installed in proximity to the excavations for Unit 2. This instrumentation includes:

1. Monuments and pins for the monitoring of the rock slot west of the 12-line wall.

2. Heave monuments in the bottom of the reactor excavation.

3. Survey control pins in the bedrock along the axes of the reactor excavation.

4. Survey monuments near the perimeter of the control and reactor building excavations.

5. Strain gauges on the reinforcing steel within the 12-line wall.

6. Stressmeters in boreholes.

7. Inclinometers in boreholes.

8. Multipoint extensometers in boreholes.

9. Borehole piezometers.

10. Linear displacement sensors in structural gaps around the reactor complex.

Items 1 through 8 were detailed in response Q361.5 to a NRC Request for Additional Information[154]. This section updates the February 29, 1980, data recorded through December 1981 and provides descriptions of additional instrumentation that has been installed. Further updating of subsurface instrumentation findings and conclusions pertaining to performance of the Seismic Category I facilities for the December 1981 to December 1982 period is provided in the response to NRC Question F241.16 in Appendix 2N. A final report on the results of the monitoring of the instrumentation will be presented as an amendment (Section 1.5.2).

In this section, the status and data from all the instrumentation are reviewed. Results are presented only to illustrate rock behavior reflected in instrument response. Table 2.5-33 lists the instruments installed at the Unit 2 site and notes the status of monitoring by December 1981.

2.5.4.13.1 Rock Slot Monitoring

Because the 12-line wall was poured directly against rock, a 20.3- to 30.5-cm (8- to 12-in) wide slot was drilled in the bedrock west of the wall. The slot was drilled down to el 61.6 m (202 ft) and was backfilled with loose vermiculite. Three pairs of concrete monuments were embedded in the rock surface and six pairs of pins were installed on either side of the rock slot to monitor any rock movement. From January 1977 to September 1979, measurements of the closing and opening of the slot were taken and results from these readings are presented in response to Q361.5[154].

The closing and opening of the slot exhibits a distinct trend associated with air temperature. Seasonal and daily fluctuations

were recorded and a net closure of 0.08 to 0.25 cm (0.03 to 0.1 in) was measured between the first and second summers of monitoring.

2.5.4.13.2 Heave Monuments

Three heave monuments were installed at the bottom of boreholes near the center of the reactor containment prior to the start of the excavation. The purpose of these monuments was to monitor and document any heave of the bottom of the reactor containment caused by the removal of bedrock for the excavation. No useful data were obtained from the heave monuments because of malfunctioning related to installation and/or equipment failures.

2.5.4.13.3 Survey Control Pins

Four survey control pins were installed on the bedrock surface along the axes of the reactor building at the beginning of excavation. These control pins were established for plant construction survey control and were surveyed by methods of construction level accuracy. The pins were originally surveyed in August 1975 and then resurveyed in March 1976, July 1977, and August 1977. Within the limits of accuracy of the survey equipment, no movements of the pins were detected.

2.5.4.13.4 Survey Monuments

Sixteen monuments were installed at the rock surface near the edge of the control and reactor building excavations in October 1976 to monitor possible rock movements. The positions of these monuments were surveyed with construction level accuracy until September 1978, after which they became inaccessible because of construction obstructions. No movements were detected within limits of accuracy of the survey methods.

2.5.4.13.5 12-Line Wall Strain Gauges

In 1979, a total of 12 strain gauges were affixed to the reinforcement rods within the 12-line wall to assist in measuring possible deflections of the wall that could be caused by time-dependent rock movements. Initial readings were taken in May 1979 and readings have continued. As of June 1982, changes measured in strain gauge values can be correlated with seasonal changes in temperature.

2.5.4.13.6 Stressmeters

Changes in bedrock stresses have been monitored at the Unit 2 site since July 1977 when 10 vibrating wire stressmeters were installed in four boreholes previously drilled for stress determinations. Response Q361.5[154] details the orientation and installation of the stressmeters and reports the results through March 1979. Two of the stressmeter locations (OC-5 and OC-7) were chosen to monitor stress in the vicinity of the Unit 2

excavations. The remainder were installed to measure possible changes in the regional stress field (OC-1) and stresses adjacent to the Cooling Tower fault (OC-2). Figure 2.5-87 illustrates the locations of the stressmeters, and Figures 2.5-114 through 2.5-117 transmit the results of the stressmeter monitoring.

Vibrating wire stressmeters in borehole OC-1 document the long-term stability of the orientation and magnitude of the regional stress field. No significant changes have occurred in stresses in the OC-1 vicinity in over 4 yr (Figure 2.5-114).

Data from stressmeter OC-2 (Figure 2.5-115, sheets 1 and 2) appear to indicate gradual decreases in stress levels adjacent to the Cooling Tower fault[94]; however, some uncertainty exists about the contribution of system drift to the observed stress decreases. Therefore, the OC-2 data are interpreted as confirming the expected decrease in stress levels adjacent to the Cooling Tower fault[94], but are not used to quantify the effect.

Stressmeters in the vicinity of the intake shaft indicate a stable stress field temporarily disturbed by construction-related dewatering and rewatering of the rock mass. Water level changes in the shaft have produced quick response in the vertical and inclined gauges in OC-5 close to the shaft, and gradual long-term response in the vertical and horizontal gauges in OC-7 farther from the shaft (Figures 2.5-116 and 2.5-117, sheets 1 and 2). A 1.8-kg/sq cm (25-psi) increase in the horizontal (100°W) gauge of OC-5 may indicate that a small increase in the maximum principal stress occurred between November 1978 and March 1979. This increase leveled off after March 1979 and remained unchanged until the instrument was destroyed by tunnel excavation early in 1980. This apparent increase in stress is not confirmed by any duplicate instrumentation.

2.5.4.13.7 Inclinometers

Several groups of inclinometers have been installed at the Unit 2 site. In March and April 1977, four inclinometers (11 through 14) were installed at the construction site to monitor excavation displacements. Between November and December 1979, seven inclinometers (SI-1 through SI-7) were constructed in the vicinity of the cooling water shaft to monitor horizontal control of the rock mass during dewatering of the shaft and subsequent construction activities. Inclinometers were constructed in boreholes 803, 805, 806, 810, and RS-2 during May through July 1980 as part of the investigation of the Radwaste thrust fault. Nine inclinometers (SI-8 through 10, SI-20 through 23, 820, and 821) were installed during August through October 1981 to monitor the rock mass in the vicinity of the gap provided between the structures and the rock.

The inclinometers installed at Nine Mile Point consist of 3.34-in ID PVC casings that are grouted into 11.4 to 15.2 cm (4.5 to 6 in) diameter boreholes. The inside of the casings is grooved

with four equally-spaced longitudinal guides that allow a movable borehole sensor to travel the length of the casing. The sensor provides an electrical signal proportional to the angle of inclination from its vertical axis. This electrical signal is conveyed to the surface via a cable and recorded on the portable digital indicator. Initially, readings were recorded on data sheets, but after June 1980, all readings were recorded on magnetic tape. The mathematic computations to analyze deflections are accomplished with the computer program, INCLIN.

I-Series Inclinometers

The four I-series inclinometers (I-1, I-2, I-3, and I-4) were installed at the Unit 2 construction site in March 1977 to monitor rock mass displacements near excavations. Locations of the inclinometers are shown on Figures 2.5-93 and 2.5-118, and the geometry of each installation and its relation to adjacent excavations are described in the response to NRC Question Q361.5[154]. The excavations adjacent to these inclinometers were completed 6 to 16 months prior to installation. Readings taken prior to March 1979 are presented in Reference 154. No significant displacements have been recorded in these inclinometers since that time.

Inclinometer I-1, located on the east side of the cooling tower piping trench, produced a record of periodic uniform tilting without significant zones of offsets. This cyclic pattern of uniform tilting followed by reversals to near-zero displacement is attributed to either seasonal thermal expansion and contraction or to uncorrected systematic instrument errors that are complicated by the excessive tilt of the inclinometer casing (72 deg) and the relatively shallow depth of installation. These uncertainties cannot be resolved by further evaluation of the data. If, however, the displacements are attributable to seasonal fluctuations, they are of small magnitude (6 mm, 0.24 in or less) and show no apparent long-term trends for increased displacement.

Inclinometer I-2, located 4.6 m (15 ft) south of the eastwest section of the circulating water piping trench, recorded a zone of bedding plane slip between el 67 to 66 m (220 to 218 ft). This progressive displacement toward the trench was recorded for approximately 3 months until the trench was filled with concrete in September and October 1977. The concrete effectively halted the displacements, and readings were discontinued after June 1981.

Interpretation of the data from inclinometer I-3 is hampered by the inability to position the instrument probe at the exact initial reading positions because of the addition of extensions to the inclinometer casing. Because these placement errors resulted in a sinusoidal pattern on the displacement record, it is not possible to identify any real displacements that may have occurred.

Inclinometer I-4 is located approximately 15 m (50 ft) west of the cooling tower piping trench. Nearly 5 yr of monitoring have not identified any significant displacement trends.

SI-Series Inclinometers

In November and December 1979, seven inclinometers (SI-1 through SI-7) were installed in the vicinity of the cooling water intake shaft (Figure 2.5-93) to assess the effects of the drying and rewetting of the intake shaft, the excavation of the tunnels, and the deepening of the shaft. Readings from the inclinometers are taken on a nominal monthly basis, and this frequency will be continued until after the rewatering of the shaft.

The records of the inclinometers on the west side of the intake shaft all illustrate a definite response of the rock in the walls of the shaft to the lowering of the floor of the shaft in January and September 1980. The inclinometers closest to the shaft (SI-4 and SI-3) show a quicker and larger reaction to the excavation than the inclinometer (SI-2) located 21 m (70 ft) west of the shaft. The time-displacement records for inclinometer SI-4 through December 1981 are included as Figure 2.5-119 (sheets 1-4) to illustrate the response of the bedrock to the blasting.

Inclinometer installations east of the shaft also react to the excavation activities in the shaft and lake water tunnels. Prior to its destruction by tunneling operations, SI-1 had displayed a westward relative displacement of approximately 4 mm (0.16 in) at the elevation of the east cooling water tunnel. Inclinometer SI-7, installed to a depth of 40 m (150 ft), became blocked from an unidentified cause at a depth of 19 m (63 ft) in January 1980. Comparison of the displacement records for the month of January indicates that the east wall of the shaft had moved inward toward the shaft more than the west wall.

Inclinometer records for installations along the northwest corner of the shaft (SI-5 and SI-6) are dominated by eastward displacements in response to the excavation of the north circulating water tunnel. Maximum displacements of approximately 11 mm (0.43 in) were recorded October 1980 with no additional displacements since then.

In August 1981, seven additional inclinometers of the SI- series were installed as part of a rock monitoring program. Three of the inclinometers (SI-8, SI-9, and SI-10) are located in the vicinity of the shaft, and four of the inclinometers (SI-20, SI-21, SI-22, and SI-23) are located near the reactor excavation. Nominal monthly readings began in November 1981, and results will be evaluated and presented in an amendment.

800-Series Inclinometers

The 800-series inclinometers (803, 805, 806, 810, and RS-2) were installed between May and July 1980. These instruments were placed in vertical boreholes east of the exposure of the Radwaste thrust structure in the north radwaste trench (Figure 2.5-118). The main purpose of these instruments is to monitor minor rock adjustments that might be associated with the Radwaste thrust structure (Section 2.5.1.2). In the 800-series, only inclinometers 803 and 805 have recorded any displacements beyond the error band of the inclinometer system.

As a result of the monitoring to date, two principal categories of horizontal displacement have been detected: 1) unrecovered displacements, and 2) cyclic seasonal displacements. Small, unrecovered horizontal displacements have been measured in inclinometers 803 and 805 and the cyclic variations in displacement, while evident in many of the inclinometers at the site, are best displayed with the data from inclinometer 803. The time-dependent displacement records of inclinometers 803 and 805 are presented as Figures 2.5-120 (sheets 1-4) and 2.5-121 (sheets 1-3).

Inclinometer 803 has recorded relative horizontal rock mass displacements in excess of instrument errors. These displacements, recorded on both the north-south and east-west axis, appear in the zone between el 68.9 to 67.7 m (226 to 222 ft) where a gradual southwestward displacement occurs within the Transition zone of the Oswego Sandstone. The rate of displacement varies with time. Most rapid displacements occur in June, July, and August, after which the displacement rate slows and finally reverses in January and February. However, the reversal of displacements to the northeast only partially recovers the southwestward displacement of the summer months (Figure 2.5-122). The unrecovered displacement between el 68.9 and 67.7 m occurred at an annual rate of 0.6 ±0.2 mm (0.024 ± 0.008) for the period of monitoring. No significant displacements were recorded below 9 m (30 ft) (el 67 m, 219 ft) in inclinometer 803.

The seasonal climatic movements in the upper part of the rock mass are illustrated on Figure 2.5-123, which is a plot of the relative movements of the readings at 1.5 vs 6.4 m (5 vs 21 ft). These displacements are approximately in-phase with seasonal air temperature fluctuations and are believed to be caused by the thermal expansion and contraction of the rock mass. These displacements can fluctuate through a total range of approximately 2.5 mm (0.098 in) on an annual basis and are fully recovered during each yearly cycle. During August through October 1981, two additional inclinometers, 820 and 821, were added to the 800-series installations to supplement inclinometer 803.

Inclinometer 805, located approximately 110 m (360 ft) east of inclinometer 803, displays a relative horizontal displacement

across the 18- to 19-m (59- to 61-ft) depth interval (Figure 2.5-121) with an approximately westward displacement of the overlying strata with respect to the strata below 19 m (61 ft). The total relative westerly displacement, determined by comparison of data from December 16, 1981, with those of May 29, 1980, amounts to 0.2 ±0.05 mm (0.008 ±0.002 in). Above and below the 18 to 19 m (59 to 61 ft) interval, the 805 data show only very small and variable displacements that do not exceed instrument error.

The exact cause of the unrecovered displacements recorded in inclinometers 803 and 805 is not presently known. Additionally, it appears anomalous that the rock mass in the vicinity of 803 is affected by seasonal variations in temperature to a depth of more than 6.1 m (20 ft). Readings from 803 and 805 will continue, and analysis of the data relative to inclinometers and other instruments (gap gauges, extensometers, piezometers) in the area will be made. Results of the monitoring will be reported in an amendment.

2.5.4.13.8 Extensometers

Several series of extensometers have been installed at the Nine Mile Point site. As part of a program to monitor vertical strains, two multipoint wire extensometers (MPX-1 and MPX-2) were installed in 1977, and in 1979 four rod extensometers (EX-1, EX-2, EX-3, and EX-4) were installed in the vicinity of the cooling water intake shaft. Subsequently, two additional vertical rod extensometers (EX-5 and EX-6) were installed on the east bench of the intake shaft to replace MPX-1; one additional vertical extensometer (EX-20) was installed on the east side of the reactor excavation; and one horizontal extensometer (HEX-1) was installed into the east wall of the reactor excavation (Figure 2.5-93).

MPX-Series

Specifications of the MPX installations and results of readings through March 1979 have been presented as part of the response to NRC question Q361.5[154]. Approximately monthly readings of the MPX installations continued until June 1981 when readings at MPX-2 were discontinued, and August 1981 when readings at MPX-1 were discontinued. Subsequent inspection of MPX-1 indicated that the head assembly was partially rusted and that at least the recent data were suspect. However, several general results can be drawn from the MPX monitoring. Primarily, MPX-1 documented the long-term contraction of the rock mass below el 61 m (200 ft) in the east wall of the intake shaft as a result of the dewatering of the shaft. The 27 m (90 ft) of dewatered rock section resulted in a total contraction of 2.24 mm (0.088 in) during the interval of monitoring (1977 through 1981).

MPX-2, located approximately 4.6 m (15 ft) from the northeast corner of the reactor excavation, recorded very little vertical variation of the rock mass in the reactor area. The reactor excavation was monitored in a dewatered state. MPX-2 has not been replaced, but a vertical rod-type extensometer has been installed on the east side of the reactor excavation approximately 53 m (175 ft) south-southeast of MPX-2.

EX-Series

Extensometers EX-1 through EX-4 are all located on the west side of the cooling water intake shaft and illustrate the effect of the dewatering of the shaft in December 1979. All the extensometers demonstrate a general, small contraction of the rock mass as a result of dewatering. This effect is greatest in the extensometer closest to the shaft (EX-3) and least at the most remote installation (EX-1), where a smaller section of the rock column has been dewatered. After 18 months of monitoring, a contraction rate of approximately 0.05 cm (0.02 in)/yr has been recorded at EX-2. As an illustration of the contraction resulting from dewatering, the readings from EX-4 are presented as Figure 2.5-124.

Above el 52 m (170 ft), the rock mass in the vicinity of the shaft displays a cyclic pattern of contraction and dilation that is interpreted to be a seasonal fluctuation. These fluctuations reach a maximum of 0.13 to 0.20 cm (0.05 to 0.08 in) between maxima and minima.

Extensometers EX-5, EX-6, EX-20, and HEX-1 with thermistors on each anchor and head were installed in the fall of 1981. Reading the extensometers in conjunction with the thermistors will enable an evaluation of the temperature effects on vertical movements. To date, there are insufficient data to reach any conclusions regarding the monitoring of these instruments. Results of the monitoring will be reported in an amendment.

2.5.4.13.9 Piezometers

During the course of geologic investigations at the Nine Mile Point site, a number of piezometers and standpipes were installed to measure groundwater levels. Section 2.4.13.2.1 provides details on the locations and results of the standpipe installations. Table 2.5-33 lists the piezometer installations in the vicinity of the main excavations. Four of the piezometers (PI-1 through PI-4) were installed in the vicinity of the cooling water intake shaft in November 1979 to monitor the dewatering of the shaft (Figure 2.5-93). In the fall of 1981 three additional pore pressure transducer-type piezometers (PI-5, PI-20, and PI-21) were installed (Figure 2.5-93) as part of the rock monitoring program. Results from the monitoring of these piezometers will be reported in an amendment.

2.5.4.13.10 Linear Displacement Sensors

Starting in the fall of 1981, the installation of a system of mechanical and electrical gauges and displacement sensors was begun in the reactor complex to monitor the structural gaps between Category I structures. These gaps were constructed to provide space for possible horizontal movements related to rock squeeze (Section 2.5.4.1.4) and the Radwaste thrust structure (Section 2.5.1.2.3). A total of 21 gauges will be installed in 11 locations (Table 2.5-33 and Figure 2.5-125).

The locations chosen for monitoring are those with the narrowest gaps between the concrete structures and in areas where possible rock squeeze displacement are predicted to be the largest. Additionally, the instruments will be oriented to monitor displacement normal to the reactor containment as well as shear movement. In some locations, both mechanical and electrical gauges are to be installed to provide a duplicate system. Temperature measurements will be recorded at location G-3 (Figure 2.5-125), and readings at all the locations will be taken monthly. Any significant results will be reported in an amendment.

2.5.4.14 Construction Notes

Since the excavated bedrock at the adjacent Unit 1 and FitzPatrick plants was sound and stable, rock dowels were not originally considered for the excavation of the Unit 2 site. Due to extended construction delays of Unit 2, the excavated rock surfaces were exposed to much longer atmospheric effects and weathering than those of adjacent sites. To preclude possible rock detachment and spalling around the perimeter of excavations, rock dowels were installed (Figure 2.5-113). A maximum of three to four air track drills were utilized to drill the holes and install the dowels. Approximately 1,000 dowels varying from 3.1 m (10 ft) to 9.1 m (30 ft) in length were installed in two parallel rows spaced 0.5 m (1.5 ft) and 2.0 m (6.5 ft) from the face of the excavation and at 1.5 m (5.0 ft) center to center. Though some dowels were installed vertically, most dowels were placed at an inclination of up to 26.5 deg away from the excavation face. Dowel anchorage and encapsulation were provided by Portland cement grout with a Bentonite admixture.

The original excavation plan for Unit 2 included a bedrock pedestal 7.6 m (25 ft) wide by 15.2 m (50 ft) long by 9.1 m (30 ft) high (el 64 to 74 m [211 to 242 ft]) located on the electrical cable tunnel bench at the northwest corner of the reactor excavation. The pedestal was to provide support for the radwaste pipe tunnel. To preserve the integrity of this rock mass during excavation, approximately 55 resin grouted prestressed rock bolts 12.2 m (40 ft) in length were installed prior to blasting. Though several precautions were taken during blasting around the perimeter, it became evident after exposure on the north end that approximately one-third of the volume of the rock had shifted at the base of the pedestal. Additional

investigations indicated that the integrity of the remainder of the pedestal was suspect; thus the entire rock mass was removed and replaced with concrete fill.

A number of rock instrumentation systems were installed during construction. These systems were designed to monitor time-dependent rock movements that might occur in response to excavations for the Unit 2 power complex. The instrumentation program included:

1. Monitoring opening and closure movement of a rock slot behind the 12-line wall.

2. Monitoring heave in the bottom of the reactor containment.

3. Monitoring survey control pins on bedrock surface along axes of reactor excavation.

4. Monitoring survey monuments on bedrock surface at the edge of the control and reactor building excavations.

5. Monitoring strain change in reinforcement within the 12-line wall.

The results of the monitoring program are discussed in Section 2.5.4.13.

At the completion of overburden removal for the cooling tower circulating water pipe trench, a fault was identified in the bedrock surface at the northeast quadrant of the originally proposed cooling tower location. To meet the construction schedule the cooling tower was relocated before the completion of the fault study (Section 2.5.1.2.3). Thus the centerline of the cooling tower was moved 154.8 m (508 ft) northeast so that the perimeter of the structure is now 15.2 m (50 ft) from the surface exposure of the fault.

The radwaste/heater bay fault (Section 2.5.1.2.3) was originally identified in the summer of 1976 during routine excavation inspections[154,210]. The immediate impact on construction was to delay fill concrete placement and erection of certain structural walls in these areas to allow for completion of geologic studies. By November 1976 these areas were released to construction following NRC and NMPC inspections. The heater bay fault exposure was covered shortly thereafter with fill concrete and a structural wall.

As investigations progressed in 1977, the site geology/structure interactions became more extensive. Additional borings and excavation trenches were needed and, once again, construction in the radwaste area was halted.

In late 1979 portions of the existing electrical ductline trench through the radwaste area were deepened and widened for detailed fault study. Holds on the design and construction of the radwaste building were extended pending the completion of structural impact evaluation. This situation continued through 1980 when it was decided to complete the radwaste excavation in order to assess the full impact to the radwaste structure. The excavation was completed in the summer of 1981. In September 1981 the foundation design was completed and the radwaste area was released to construction.

2.5.5 Stability of Slopes

The site is located in an area that is gently sloping toward Lake Ontario. The topographic expression is generally subdued and irregular. Major Category I structures are founded on bedrock with the final plant grade established at el 78 m (260.6 ft), approximately the same as the preconstruction surface elevation. Therefore, there are no natural slopes subject to failure during the SSE.

2.5.5.1 Slope Characteristics

There are no cut or fill slopes whose postulated failure could affect the safe shutdown of the unit immediately following the SSE. There are, however, two structures whose postulated failure during storm conditions (Sections 2.4.2 and 2.4.5) could adversely affect the safe shutdown of the unit: the revetment ditch system along the shorefront and the PMP flood control berm aligned around the plant perimeter. A plan and section view of the revetment ditch system is shown on Figures 2.5-126 and 2.5-127. A plan view of the flood control berm appears on Figure 2.5-128.

2.5.5.2 Design Criteria and Analyses

The slopes of the revetment ditch system were designed to be stable under conditions imposed by the PMWS. Model tests were performed to determine the static stability of the system. Design criteria and results of these tests are discussed in Section 2.4.5.1 to 2.4.5.5. The revetment ditch system was also analyzed with respect to the factor of safety versus slope failure during a combination of storm and earthquake events. The following cases were considered:

Case I - Lake level at el 248.8’ (25-yr flood) + SSE (0.15 g)
Case II - Lake level at el 249.5’ (100-yr flood) + OBE (0.075 g)

Pseudo-static stability analyses were conducted for both of the above cases using an ICETRAN computer program entitled Limiting Equilibrium Analysis in Soil Engineering (Lease II). In this program, a given stability problem is specified to the computer

in free format as a series of points and lines which comprise the soil layers and their associated piezometric surfaces. Any two-dimensional slope stability problem can be analyzed. Once the geometry, physical data, and strength parameters have been specified, a command is given to use the desired analytical procedure, whether it be the Normal, Bishop, or Morgenstern - Price method.

In the pseudo-static seismic analysis of the revetment ditch system, slopes were analyzed using the modified Bishop method with a search for the minimum factors of safety. The seismic coefficients (0.15 for SSE and 0.075 for OBE) are applied in the horizontal and vertical direction at the centroid of a given slice. The effect of these accelerations in the Lease II program is an inertial force (that is, the weight of each slice multiplied by the seismic coefficient) which acts in the direction of sliding.

Conservative estimates were specified for unit weight and strength. The dolos armor units were assumed to act as a surcharge load. No benefit was considered for its interlocking tendencies during movement. Figure 2.5-135 illustrates the input geometry, parameters, and resulting critical failure surfaces. The minimum factors of safety (modified Bishop's method) are 1.07 for Case I (SSE) and 1.51 for Case II (OBE). Therefore, the revetment ditch system is stable under these conservative environmental loadings.

2.5.5.3 Soil Investigation

Borings were drilled along the alignment of the revetment ditch system[202].

2.5.5.4 Compacted Fill

This section provides information related to material properties, construction procedures, and placement control of the rock fill required for the revetment ditch system as shown on Figures 2.5-126, 2.5-127, 2.5-127a, and 2.5-127b. Hydrologic and meteorologic aspects of the shore protection facility at the Unit 2 site are described in Section 2.4.3.

2.5.5.4.1 Rock Fill

The rock fill consists of backslope armor units, underlayers, and granular filters. They are quarried materials which consist of angular to subangular, hard, clean, sound, and durable rock with a minimum specific gravity of 2.50. All rock materials are verified through a quality test program that includes petrographic examination, bulk specific gravity and absorption test, and accelerated weathering test (freezing-thawing and wetting-drying). The rock is free of any deleterious materials, such as flat, elongated, friable, decomposed, micaceous, or argillaceous pieces, and is highly resistant to weathering and

disintegration under freezing-thawing and wetting-drying conditions. The smallest dimension of any piece of backslope armor and first underlayer is at least one-third of its largest dimension.

Gradation limits and weight requirements for the backslope armor and underlayers are listed in Table 2.5-34. Gradation limits for granular filters are listed in Table 2.5-35.

2.5.5.4.2 Construction Procedure

Existing ground beneath the armor units and underlayers is excavated to bedrock, as shown on Figure 2.5-127. Any soft, loose, or deleterious soils which are not suitable as foundation materials are excavated and backfilled to specified grades with controlled granular material. The area is graded and sloped to route runoff away from the work area without erosion or ponding.

Placement of granular filter and stone follows the grading operations. Damaged subgrades are repaired prior to placement of granular filter and underlayers.

The lateral drainage ditch is constructed with a minimum slope of 0.5 percent, from a high point located about midway between outlets to the lake. The grading of the ditch is shown on Figure 2.5-127.

The granular filter is placed in a manner to minimize segregation. Improperly placed granular filter is repaired or removed and replaced.

The size and specific gravity requirements of the backslope rock armor units and underlayers are contained in Section 2.5.5.4.1. The rocks used as backslope armors and underlayers are angular to subangular. These requirements assure that the rocks used in the prototype will reproduce that used in the model[211]. The underlayer rock is randomly placed, then graded to provide minimum segregation. The placement starts at the toe and proceeds up the slope to the required lines and grades that are determined by surveying. The first underlayer and the second underlayer each consist of a minimum of two layers of rock.

The backslope rock armor units are placed individually and in a manner to avoid displacing underlying materials and to minimize chipping of rocks. The backslope armor consists of one single layer of rocks. The armor units are placed with maximum interlocking without overhanging or floaters. Placement of armor units commences at the bottom of the slope and continues up the slope.

The front armor is dolos concrete armor. Each dolos has a minimum weight of 2,205 kg (4,900 lb). Dolos placement commences at the toe of the revetment slope and proceeds up the slope. All dolosse are placed individually and randomly, and they are

uniformly distributed in two layers on the slopes. Dolosse at the toe of the slopes are placed in such a manner that the horizontal flukes are located inward and adjacent flukes are touching.

A systematic program of testing, inspection, and documentation was implemented during construction of the revetment ditch system. After each blasting operation, stones were selected by comparing with the predetermined reference sample stones for final inspection.

A systematic inspection program was used in accepting lots of stones. The detailed description and the results of QC inspection on granular filter, rock underlayers, and concrete dolos are presented in References 2 and 7. Results of QC tests were provided to the NRC in Reference 289.

2.5.5.4.3 Post-Construction Considerations

An inspection and surveillance program for the completed revetment will be in effect to detect any changes in revetment crest elevation or cross section. The revetment ditch system will be visually inspected a minimum of once a year during the life of the plant. If changes in excess of established criteria are detected, remedial measures will be taken.

2.5.6 Flood Control Berm

2.5.6.1 General

The purpose of the flood control berm (FCB) is to protect the plant complex during the PMP flood flows. The severe storm conditions under which the FCB is designed to function are discussed in detail in Section 2.4.2.3.3. The FCB consists of four separate but adjoining embankments as shown on Figure 2.5-154. The segments of the berm will be referred to in this section as the east berm, west berm, southeast berm, and lake road berm. This alignment serves to route flood flows to Lake Ontario around the plant perimeter. The berm is constructed of Category I structural fill (Section 2.5.4.5.2), and heights range from approximately 2 ft to 20 ft. The foundation materials vary from hard glacial till to recently placed construction fill. Category I structural fill design side slopes are either 1.5H:1V or 2H:1V, where H = horizontal and V = vertical. Grass cover is provided for erosion protection.

2.5.6.2 Exploration

General exploration of geologic conditions at Unit 2 is discussed in detail in Section 2.5.4.3. Additional exploration of subsurface conditions along the alignment of the FCB was conducted in 1983. The purpose of this program was to determine the dimensions, characteristics, and engineering parameters of the various earth and rock strata which comprise the berm

foundation. The exploration proceeded in stages consisting of field explorations and laboratory tests as outlined below:

1. Field Explorations

    a. Test borings and undisturbed block samples.

    b. Groundwater observations.

    c. Percolation tests.

2. Laboratory Tests

    a. Direct shear tests.

    b. Static triaxial compression tests.

    c. Permeability tests.

    d. Consolidation tests.

    e. Particle size analysis.

    f. Compaction tests.

2.5.6.2.1 Field Exploration Program

Test Borings and Undisturbed Block Samples

Twenty-eight Nx borings were drilled along the alignment of the berm, the locations of which are shown on Figure 2.5-155. These borings range in depth from 4.5 ft to 27 ft and were drilled under continuous observation utilizing truck-mounted, rotary-core drilling equipment. Soil samples were recovered throughout the overburden strata by means of a standard split spoon sampler. These samples were taken by standard penetration test procedures. All borings were drilled to refusal on rock with approximately 2 ft of rock coring. Detailed boring log descriptions of encountered soil and rock are presented on Figures 2.5-158 through 2.5-185. The soils were classified in accordance with the Unified Soil Classification system.

Two 9 in x 9 in x 9 in undisturbed block samples were taken for consolidation, direct shear, and triaxial compression tests (Section 2.5.6.2.2). The locations of these block samples are shown on Figure 2.5-155. A detailed illustration of the soil samples is shown on Figure 2.5-156.

Groundwater Observations

The groundwater table in the main plant area has been defined by water levels measured in previous exploratory borings and piezometers (Section 2.5.4.6.5). To further assess hydraulic heads and, in particular, in the plant outer areas where the FCB

is located, six standpipe-type piezometers were installed in various boreholes. The location of these piezometers is shown on Figure 2.5-155. Water levels were measured by means of an electronic recorder and are shown graphically with respect to time on Figure 2.5-157. The average seasonal variation of the groundwater table observed during the 1983-1984 monitoring period was about 2 ft. This compares favorably with the variation reported for earlier piezometers.

Percolation Tests

A total of 17 field percolation tests were performed in various boreholes throughout the site in order to obtain estimates of the permeability of the foundation soils. The test locations are shown on Figure 2.5-155. These tests were of the falling head type and were performed on all soil zones which comprise the foundation of the FCB (Section 2.5.6.3.1). The permeability of the materials in the test sections was determined in accordance with the following formula[284]:

$$K = \frac{r_1^2}{2L\Delta_t}\left[\tfrac{1}{2}\ \sinh^{-1} \frac{(L)}{r_e}\ loge\ \frac{(2H_1 - L)}{2H_2 - L}\ -\ loge\ \frac{(2H_1\ H_2\ -\ LH_2)}{2H_1\ H_2\ -\ LH_1} \right]$$

Where:

K	=	Average permeability of the test section, ft/s
L	=	Length of test section, ft
r_1	=	Inside radius of drop pipe, ft
r_e	=	Effective radius of test section, ft
Δ_t	=	Time intervals (t_1-t_0, t_2-t_1), sec
$\sinh^{-1}$	=	Inverse hyperbolic sine
loge	=	Natural logarithm
H	=	Height of water column from bottom of test interval to water surface in standpipe, ft (H_0, H_1, H_2 heights at time of measurement t_0, t_1, t_2, etc.)

The resulting permeability ranges for each zone are presented in Table 2.5-45A.

Soil Stratigraphy

Longitudinal profiles depicting the subsurface stratification are shown on Figure 2.5-186 through 2.5-189. These indicate that the

natural subsurface stratification at the site is generally as follows:

1. Granular soils (construction fill).
2. Organic soils.
3. Lacustrine deposits.
4. Glacial till.
5. Sandstone bedrock.

The granular soils were encountered throughout the investigation and display a wide range of gradation and composition. Based on boring log descriptions and gradation tests (Figure 2.5-200), this material is described as compact to very dense, silty sand to sandy silt with intermittent gravel-size zones (SM-GM).

The lacustrine soils are glacially related sand/silt to silty clay deposits that cover certain portions of the site. They are predominantly grayish in color and vary from 1 ft to 4 ft in thickness. Based on boring log descriptions and gradation tests (Figure 2.5-200), this material varies from dense, fine sand with silt to stiff, silty clay (ML-CL).

Organic soils along the FCB alignment are described as soft to stiff buried topsoils containing varying amounts of peaty materials. These soils are pocketed and lenticular and generally very dark gray to black. The plasticity range is slight to moderate.

Below the organic and lacustrine deposits and generally throughout the site, there is a very dense glacial till layer comprised of a heterogeneous mixture of gravel, sand, silt, and clay. The thickness of this layer varies from 0 ft to 23 ft. Bases on boring log descriptions and five gradation tests (Figure 2.5-200), glacial till is classified as silty sand to slightly plastic silt with varying amounts of clay with occasional cobbles and boulders.

The Oswego sandstone bedrock is found between el 246 ft and 276 ft. The top of rock is deepest at the southern end of the east berm and highest at the southernmost end of the southeast berm.

2.5.6.2.2 Laboratory Testing Program

The purpose of the laboratory testing program was to determine index, mechanical, and strength properties of the foundation and embankment materials of the FCB. The types of tests performed were as follows:

1. Embankment Materials

a. Gradation.

b. Compaction.

c. Permeability.

d. Triaxial compression.

2. Foundation Materials

a. Gradation.

b. Triaxial compression.

c. Direct shear.

d. Consolidation.

The results of these tests are described in the following sections.

Embankment Material Laboratory Tests

Embankment materials are comprised of Category I structural fill. Bag samples from three separate borrow pits, i.e., Whelsky, Keller, and Meany, were obtained in order to determine their viability as potential sources.

Gradation Tests A number of samples from each pit were tested for particle range distribution. These tests were performed in accordance with Appendix V[283]. These materials are classified as a well-graded sandy gravel or gravelly sand (GW-SW) with few or no fines (ASTM D-2487). Gradation curves illustrating the results of the particle size analyses are shown on Figure 2.5-190.

Compaction Tests A total of five compaction tests were performed of representative samples of structural fill obtained from the three potential borrow sources. These tests were performed in accordance with ASTM D-1557, Method D. The results of the compaction tests are graphically illustrated on Figures 2.5-191 through 2.5-193 and are summarized below:

Potential Borrow Sources	Maximum Dry Density (pcf)	Optimum Water Content (%)
Whelsky - northern pit (sample no. 1)	137.7	7.3
Keller - northern pit (sample no. 2)	134.1	7.7
Keller - northern pit (sample no. 3)	135.9	7.9
Meany (sample no. 4)	136.5	7.8
Meany (sample no. 5)	135.4	7.6

The minimum and maximum dry densities for the potential borrow materials were determined by vibratory table method in accordance with ASTM D-2049. A summary of the test results is as follows:

Potential Borrow Sources	Maximum Dry Density (pcf)	Minimum Dry Density (pcf)
Whelsky - northern pit (sample no. 1)	135.9	117.0
Keller - northern pit (sample no. 2)	134.5	115.6
Keller - northern pit (sample no. 3)	132.9	114.3
Meany (sample no. 4)	132.9	110.3

Triaxial Compression Tests A number of static, consolidated, undrained triaxial compression tests were performed on compacted samples from each borrow pit. These tests were performed in accordance with Appendix X[283]. The material properties of each specimen are presented in Tables 2.5-46A, B, and C. The results of strength tests for each pit are shown on Figures 2.5-194 through 2.5-196. The results of these tests are summarized below:

Potential Borrow Sources	Average c′ (psf)	Average ϕ' (degree)
Whelsky	0	40
Keller	0	42
Meany	0	40

Permeability Tests A number of constant head permeability tests were performed on samples of compacted structural fill. These tests were performed in accordance with Appendix VII[1]. Samples were compacted to at least 90 percent of the maximum dry density obtained in modified proctor tests. Results showing total flow versus time are presented for each of the borrow sources on Figures 2.5-197 through 2.5-199. The range of calculated permeabilities for each borrow pit based on the test results is presented below:

Potential Borrow Sources	Range of Permeability, k (cm/sec)
Whelsky - northern	6.7×10^{-4} to 4.7×10^{-5}
Keller - northern	6.7×10^{-4} to 4.5×10^{-5}
Meany	1.4×10^{-4} to 2.9×10^{-6}

Design Criteria Based on the above test results, the following parameters of structural fill were used for design purposes:

1. The effective internal friction angle, $\phi' = 40°$.

2. The total unit weight, γm = 130 pcf.

3. The permeability, k = 7 x 10^{-4} cm/sec.

Foundation Material Laboratory Tests

Foundation materials consist of natural and construction soils as described in Soil Stratigraphy under Section 2.5.6.2.1. Split spoon samples recovered throughout the overburden strata were subjected to gradation tests. Representative, undisturbed block samples were also obtained and tested under triaxial shear, direct shear, and consolidation conditions.

Gradation Tests A number of selected soil samples from the boring program were tested to determine their corresponding grain size distribution. These tests were performed in accordance with Appendix V[283]. Gradation curves illustrating the results of the particle size analyses are shown on Figure 2.5-200.

Atterberg limits tests were also performed for organic silt and lacustrine deposits in accordance with Appendix III[283]. The test results are summarized in Table 2.5-47.

Consolidation Tests Undisturbed samples of organic silt and lacustrine deposits were subjected to consolidation tests. These tests were performed in accordance with Appendix VIII[283]. Typical results of consolidation tests are shown on Figure 2.5-201.

The test results indicate that organic silt and lacustrine deposits are highly overconsolidated. The overconsolidation ratio ranges from 5 to 6.

Direct Shear Tests

Drained direct shear tests were performed on a selected lacustrine deposit sample in accordance with Appendix X[283]. A summary of the test results is shown on Figure 2.5-202. The test results indicate that the effective internal friction angle, ϕ', is 22.6 deg and the effective cohesion, c', is 350 psf.

Triaxial Compression Tests

A number of static, consolidated, undrained triaxial compression tests were performed on representative undisturbed samples of organic silt and lacustrine deposits. These tests were performed in accordance with Appendix X[283]. The material properties of these soils are shown in Table 2.5-48. The results of strength tests are presented on Figures 2.5-203 through 2.5-205. Test results are summarized as follows:

	Effective	Effective Angle of Internal

Soil Type	Cohesion, c′, (psf)	Friction, ϕ', (deg)
Organic silt	586	31.5
Silty clay	543	23.0
Silty clay	182	28.6

Design Criteria

Based on test results discussed in previous sections, the following values of soil parameters were adopted for design purposes:

Moist unit weight, γ_m	
Organic silt, pcf	98
Silty clay, pcf	124
Saturated unit weight, γ_s	
Organic silt, pcf	103
Silty clay, pcf	125
Cohesion, c′	
Organic silt, psf	580
Silty clay, psf	180
Angle of internal friction, ϕ'	
Organic silt, deg	31
Silty clay, deg	28

2.5.6.3 Foundation and Abutment Treatment

The foundation for the FCB consists of those soil zones which meet permeability and stability requirements. No grouting or dental work is required.

To provide an effective barrier against seepage, any soil zones possessing high permeability are excavated to a depth at least 1 ft below the bottom of the previous layer. Depending on geometry considerations, two types of foundations are used: cutoff trenches or level spread foundations.

A typical section for a level spread foundation is shown on Figure 2.5-206. Subgrade treatment for this type of foundation is accomplished by bringing the existing topography along the berm alignment to design lines and grades. Any stump holes, cavities, or depressions are broken down, flattened, and then scarified. Proofrolling is then performed on the foundation materials to eliminate the possibility of loose zones which could cause differential settlement cracks in the embankment. This proofrolling consists of compacting the soil materials by twice the number of passes normally required. After this phase of foundation preparation, thorough and systematic inspections are made of the resulting surfaces to determine whether there exists any zones of excessive rutting, settlement, or other

irregularities that require further corrective measures. All unsuitable zones are removed and replaced with compacted structural fill (see Direct Shear Tests under Section 2.5.6.2.2).

A typical section for a cutoff trench foundation is shown on Figure 2.5-206. Subgrade treatment for this type of foundation consisted of stripping the existing topography and excavating to the design lines and grades of the cutoff trench. The slopes and bottom of the cutoff trench are scarified and then compacted by twice the number of passes normally required. The remainder of the foundation preparation is performed in the same manner as for level spread foundations.

Abutments exist for the east, southeast, and lake road berms. For the east berm, the abutment is the concrete mat of a construction warehouse; therefore, no abutment treatment is required. For the southeast and lake road berms, the compacted embankment is placed on the existing lake road slope along the longitudinal axis. A typical section of this placement is shown on Figure 2.5-207. Abutment treatment for this section consisted of providing an additional 4-ft layer of structural fill against the lake road slope.

2.5.6.3.1 Liquefaction Potential

As discussed in Sections 2.4.13.2 and 2.4.13.5, the normal groundwater table at the plant area is conservatively assumed at el 255 ft for design purposes. Since rock grade at boring locations BB-1, BB-2, BB-6 through BB-9, BB-19, BB-20, BB-22, and BB-23 (see Test Borings and Undisturbed Block Samples under Section 2.5.6.2.1) is below the normal groundwater table, the soils at these locations were evaluated against liquefaction potential during the SSE (Section 2.5.2.6) in accordance with the method presented by Seed and Idriss[285].

The steps used to determine liquefaction potential at each boring location are as follows:

1. Calculate both total and effective vertical overburden pressures at a depth below the normal groundwater table.

2. Find the stress reduction coefficient, γd[207].

3. Compute $\left(\tau\ average/\sigma^{1}v\right) \cong 0.65\,\dfrac{(\sigma v)}{\sigma^{1}v}\,\dfrac{(a)}{g}\ r_d$ [207]

Where:

$$\frac{\tau\ average}{T^{l}v} = 0.0975\ rd\ \frac{(\sigma v)}{\sigma^{l}v}$$

4. Find (τ average/σ^1v) based on blow counts, N-values[285].

5. Apply correction factor of 1.32[285] to the value in Step 4. If the corrected value of (τ average/σ^1v) in Step 4 is greater than the calculated value in Step 3, liquefaction will not occur.

There were no adjustments made to the measured blow counts (N-values) for overburden pressures since the maximum overburden pressure in these borings does not exceed 1 ton/sq ft.

The results of the analysis indicate that there is no potential for liquefaction during the SSE along the flood control berm. The analysis summary for each boring location is presented in Table 2.5-49. The N-value profiles at these boring locations are shown on Figure 2.5-208.

2.5.6.4 Embankment

2.5.6.4.1 General Features

Typical sections for each berm segment are shown on Figure 2.5-209. These sections illustrate berm features such as height, slope, zoning, and location and usage of materials in the embankment.

2.5.6.4.2 Compaction and Placement Control of Structural Fill

Compaction of the berm foundation is discussed in Section 2.5.6.3. Compaction of embankment backfill consists of a minimum of four passes of suitable compaction equipment. All granular materials are placed in horizontal loose lifts having loose lift thicknesses not exceeding 12 in for heavy vibrating compactors and 6 in for walk-behind vibrating compactors. For portions of the berm not accessible to rollers, loose lift thicknesses do not exceed 4 in. Satisfactory compaction resulted in a soil dry density of not less than 95 percent of the maximum dry density as determined by ASTM D-1557, Method D (see Compaction Tests under Section 2.5.6.2.2). Moisture control was such that the resulting moisture content was plus or minus 3 percent of the optimum moisture content. The in-place density of the structural fill is measured by the sand cone method and/or a nuclear testing device. A systematic program of testing, inspection, and documentation is implemented during the backfill operations. This QC program is summarized in Table 2.5-50.

Partial fill surfaces are protected during periods of wet weather by crowning and rolling to smooth a partial fill surface as protection against excessive absorption of moisture and to facilitate runoff. Upon resumption of operations, all materials which are excessively soft are removed and stockpiled for use after the material has dried to acceptable limits.

2.5.6.4.3 Slope Protection

Slope protection for the FCB consists of properly cultivated grass to prevent erosion from surface runoff during rainstorms. This should eliminate the development of erosion gullies on the upstream and downstream sides and at changes in embankment slope. Due to the fact that the berm is not exposed to a large reservoir, except during the flood event, wave protection is not considered.

Seed and mulch was distributed on all embankment slopes. Lime and fertilizer also was applied.

2.5.6.5 Slope Stability

The slopes of the FCB were designed to be stable under a combination of environmental conditions. The following cases were considered for each segment of the berm:

- Case 1 - Water level at el 255 ft (normal case).
- Case 2 - Maximum water level during 25-yr rainstorm and SSE (0.15 g).
- Case 3 - Maximum water level during PMF.
- Case 4 - Maximum water level during one-half PMF and OBE (0.075 g).

During the 25-yr rainstorm, rainfall will be contained in ditches and culverts of the site drainage system. Therefore, Case 2 is simply the combination of normal groundwater at el 255 ft and SSE. In this analysis, the maximum water level during PMF was based on Hydromet Report Nos. 51 and 52. The maximum water level during PMF used in Case 4 was based on one-half the PMF water level in Hydromet Report Nos. 51 and 52.

Static and pseudo-static stability analyses were conducted for each of the above cases using an ICETRAN computer program (Section 2.5.5.2). Shear strength testing was performed to determine design parameters. Results of this testing are discussed in Section 2.5.6.2.2.

The static stability of the FCB was analyzed by Bishop's Method. The critical section of each particular berm segment was considered. The preceding four cases were applied to these sections, and the resulting factors of safety were determined.

The results are summarized on Figures 2.5-210 through 2.5-212. As shown on these figures, the minimum factors of safety are 1.49, 1.04, 1.13, and 1.02 for Cases 1 through 4, respectively. Therefore, the FCB is stable under the design environmental loadings.

2.5.6.6 Seepage Control

The seepage parameters used in the design of the FCB are discussed in exploration and testing sections (Section 2.5.6.2). The allowable seepage rate through the FCB during the PMP flood is 10 cfs, which is discussed in detail in Section 2.4.2.3.3. Seepage control consists of strict maintenance of density criteria for the structural fill in the embankment (Section 2.5.6.4.3). No other special construction requirements or seepage control features are included in the design of the FCB. Total seepage along the individual berm segments is not expected to exceed 3.3 cfs; therefore, the FCB is suitably designed against excess seepage during the PMP flood flow.

2.5.7 References

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TABLE 2.5-1
Sheet 1 of 1
HOMOGENIZATION TEMPERATURES FROM
FLUID INCLUSIONS IN MINERALIZATION

Mineral	Location	Primary	Pseudosecondary	Secondary	Reference
Fluorite	Penfield, NY	-	132°-142°C (270°-288°F)	-	109
Fluorite	Madoc, Ontario	122°-132°C (252°-270°F)	-	85°-105°C (185°-221°F)	110
Calcite	Ontario, NY	225°-300°C (437°-572°F)	-	75°-125°C (167°-257°F)	89

TABLE 2.5-2
Sheet 1 of 7
HISTORIC SEISMIC EVENTS KNOWN TO HAVE OCCURRED
WITHIN 321.8 KM (200 MI) OF NINE MILE POINT

Year	Month	Day	Hour	Minutes	Seconds	Latitude (N)	Longitude (W)	MMI	Magnitude	Distance (km)	Reference
1981	09	16	14	41	33	43.4	76.4	-	2.7 MC	12	WCC
1981	11	03	11	42	19	43.8	76.6	-	1.5 MC	21.2	WCC
1981	11	03	11	06	53	43.7	76.6	-	1.6 MC	21.7	WCC
1974	03	09	06	32	-	43.8	76.8	-	-	42	NYS
1925	05	23	-	-	-	43.4	77.1	III	-	50	NYS
1925	04	07	20	18	-	43.1	76.1	III	-	56	NYS
1954	02	10	00	37	-	43.0	76.7	-	3.3 ML	59	NYS
1945	04	15	13	15	-	43.0	76.4	III	-	60	NYS
1945	04	15	14	20	-	43.0	76.4	III	-	60	NYS
1945	04	15	15	30	-	43.0	76.4	III	-	60	NYS
1927	03	29	20	30	-	43.0	76.1	III	-	65	NYS
1927	03	31	21	00	-	43.0	76.1	III	-	65	NYS
1981	09	07	06	16	52	42.9	76.1	-	1.8 MC	68	WCC
1946	03	20	02	01	-	44.3	75.9	III	-	70	NYS
1952	11	20	-	-	-	42.9	75.6	III	-	70	NYS
1946	03	20	02	29	-	44.3	75.9	II	-	70	NYS
1946	03	20	03	02	-	44.3	75.9	III	-	70	NYS
1954	09	29	03	50	-	44.0	75.9	-	-	71	NYS
1963	01	30	14	50	-	44.0	75.9	-	3.0 ML	71	NYS
1941	10	09	22	07	-	44.0	75.9	II	-	71	NYS
1941	10	20	21	29	-	44.0	75.9	II	-	71	NYS
1853	03	12	07	00	-	43.7	75.5	VI	-	80	NYS
1977	12	15	08	55	25	43.0	77.4	-	2.6 MB	90	NYS
1946	11	10	11	41	23	42.9	77.5	-	3.1 ML	100	NYS
1980	08	11	14	54	46	43.5	75.2	-	3.3 MB	100	NYS
1963	05	19	19	14	18	43.5	75.2	III	3.5 ML	100	NYS
1980	06	06	13	15	52	43.6	75.2	V	3.8 MB	105	NYS
1980	06	06	13	05	57	43.8	75.0	-	3.1 MN	124	DOC
1867	12	18	08	-	-	44.0	75.2	VI	-	125	NYS
1981	06	20	02	10	45	44.3	75.3	-	2.5 MC	130	WCC
1840	01	16	20	-	-	43.0	75.0	V-VI	-	135	NYS
1927	03	14	14	15	-	44.6	75.4	-	3.7 ML	147	DOC
1965	07	16	11	06	57	42.9	78.2	IV	3.2 ML	148	NYS
1890	05	25	12	10	-	43.0	74.8	V	-	150	NYS
1965	08	27	01	55	56	43.0	78.1	-	3.3 MN	150	DOC
1903	12	25	12	30	-	44.7	75.5	IV	-	152	NYS
1975	11	03	20	54	56	43.9	74.6	IV	3.9 MB	152	NYS

TABLE 2.5-2
Sheet 2 of 7

Year	Month	Day	Hour	Minutes	Seconds	Latitude (N)	Longitude (W)	MMI	Magnitude	Distance (km)	Reference
1922	12	08	21	24	-	44.4	75.1	V	-	156	NYS
1966	01	01	13	23	38	42.9	78.2	VI	4.6 MB	158	NYS
1967	06	13	19	08	54	42.9	78.2	VI	4.4 MB	156	NYS
1927	03	12	22	12	-	44.6	75.2	-	3.7 ML	157	DOC
1937	03	10	05	30	-	44.6	75.2	-	3.7 ML	157	DOC
1973	07	15	10	32	38	44.0	74.4	-	3.2 MN	160	DOC
1973	07	16	08	41	58	43.8	74.5	-	3.3 MN	162	DOC
1971	06	21	02	48	30	43.9	74.5	IV	3.4 ML	163	NYS
1971	05	23	09	30	00	43.9	74.5	V	3.8 MB	163	NYS
1929	08	12	11	24	48	42.9	78.4	VII	-	164	NYS
1971	05	23	06	24	28	43.9	74.5	V	4.1 MB	165	NYS
1929	12	02	22	14	-	42.8	78.3	V	-	168	NYS
1968	09	03	-	-	-	42.8	78.3	IV-V	3.3 ML	168	NYS
1955	08	16	07	35	-	42.9	78.3	V	-	168	NYS
1971	07	10	08	15	-	43.9	74.4	V	3.8 MB	168	NYS
1938	11	18	22	19	06	44.8	75.3	IV-V	-	169	NYS
1973	07	15	08	20	31	43.9	74.4	-	3.4 MB	169	NYS
1952	08	25	00	07	-	43.0	74.5	V	4.3	170	DOC
1956	03	06	23	38	10	44.9	75.4	III	3.1 ML	173	DOC
1913	04	29	00	28	57	44.9	75.3	-	4.4	178	DOC
1929	12	03	12	50	-	42.8	78.3	-	3.6 ML	178	DOC
1913	04	29	00	28	57	44.9	75.3	VI	4.4 ML	179	DOC
1938	01	06	13	28	42	44.9	75.2	III	3.2 ML	185	DOC
1917	05	22	09	00	26	45.1	75.6	-	4.0	190	DOC
1913	08	10	05	15	-	44.0	74.0	V	-	194	NYS
1978	08	21	08	47	11	44.5	74.5	-	3.1 MB	196	NYS
1954	12	13	03	53	52	44.6	74.6	IV	3.6 ML	198	NYS
1944	09	05	04	38	45	45.0	74.7	VIII	5.8 MB	200	NYS
1944	09	05	08	30	49	45.0	74.9	-	3.4 ML	200	DOC
1944	09	05	08	51	06	45.0	74.9	-	4.6 ML	200	DOC
1944	09	05	10	56	51	45.0	74.9	-	3.3 ML	200	DOC
1964	03	29	09	16	-	44.9	74.9	V	-	201	NYS
1944	09	09	23	24	18	45.0	74.9	-	4.1	203	DOC
1944	10	31	08	42	25	44.9	74.9	-	4.0	203	DOC
1957	02	20	15	45	-	44.9	74.9	-	3.6 ML	203	DOC
1958	01	11	16	36	-	44.9	74.9	-	3.6 ML	203	DOC
1941	10	21	06	10	41	44.8	74.8	-	3.3 ML	205	NYS
1980	09	21	20	54	45	43.6	74.0	-	3.2 MB	206	NYS
1880	05	31	-	-	-	45.2	75.3	-	3.7 ML	208	DOC
1928	03	18	15	25	-	44.5	74.3	V-VI	4.1	208	NYS

TABLE 2.5-2
Sheet 3 of 7

Year	Month	Day	Hour	Minutes	Seconds	Latitude (N)	Longitude (W)	MMI	Magnitude	Distance (km)	Reference
1961	03	13	10	55	45	45.2	75.3	-	3.2 ML	208	DOC
1857	10	23	20	15	-	43.2	78.6	VI	-	210	NYS
1974	11	27	10	28	-	43.3	79.1	-	3.3 MB	210	NYS
1916	01	05	13	56	-	43.6	73.7	V	-	212	NYS
1932	12	07	03	15	-	44.4	74.1	-	3.6 ML	213	DOC
1980	02	29	05	53	56	42.6	74.2	-	3.1 MB	214	NYS
1962	03	27	06	35	-	43.0	79.3	V	3.0 ML	214	NYS
1907	01	24	11	30	-	42.8	74.0	V	3.7 ML	216	DOC
1908	07	17	07	10	-	45.4	78.4	-	3.7 ML	216	DOC
1926	01	27	-	-	-	44.3	74.1	-	3.7 ML	216	DOC
1873	04	30	-	-	-	45.0	74.7	-	3.7 ML	218	DOC
1931	04	02	19	54	-	43.5	73.8	VII	4.7 MB	218	NYS
1775	07	06	06	23	51	44.5	73.9	V	-	220	NYS
1916	02	03	04	26	-	42.8	73.9	V-VI	-	220	NYS
1907	11	14	05	00	00	45.5	76.7	-	3.7 ML	221	DOC
1946	11	24	10	20	47	45.2	74.7	-	3.1 ML	221	DOC
1909	12	10	06	24	10	45.4	75.6	-	3.7 ML	222	DOC
1924	11	14	01	32	-	45.5	76.3	-	3.7 ML	223	DOC
1927	11	12	-	-	-	43.1	79.1	-	3.7 ML	223	DOC
1933	07	14	04	48	40	45.4	75.7	-	3.9 ML	223	DOC
1958	08	22	14	25	05	43.0	79.0	IV	3.6 ML	223	DOC
1855	12	17	-	-	-	43.3	73.7	-	3.7 ML	226	DOC
1935	01	28	06	00	-	44.8	74.3	-	3.7 ML	226	DOC
1935	01	28	09	01	32	44.8	74.3	-	3.2 ML	226	DOC
1916	11	02	02	30	-	43.4	73.4	V	-	227	NYS
1889	08	10	-	-	-	43.5	73.8	V	-	227	NYS
1877	11	04	-	-	-	44.5	74.0	VII	5.7 ML	228	DOC
1879	08	21	08	00	-	43.2	79.2	V	3.7 ML	228	DOC
1861	07	12	-	-	-	45.4	75.4	-	5.7 ML	228	DOC
1908	06	16	20	41	52	48.1	74.8	V	4.5	228	DOC
1950	08	05	23	59	07	45.1	74.8	IV	3.5 ML	228	NOA
1977	09	28	17	21	45	44.4	73.9	-	3.1 MN	229	NYS
1980	05	23	08	39	44	44.9	74.5	-	3.4 MB	230	NYS
1981	07	05	21	47	23	45.1	74.6	-	3.3 MC	230	WCC
1981	07	04	23	16	32	45.1	74.6	-	3.5 MC	230	WCC
1907	01	25	06	-	-	44.1	79.1	-	3.7 ML	231	DOC
1946	10	28	20	36	06	41.5	76.6	-	3.6 ML	231	DOC
1921	01	19	10	00	-	43.3	73.7	-	3.7 ML	233	DOC
1921	01	27	-	-	-	43.3	73.7	-	3.7 ML	233	DOC
1933	07	14	04	48	40	45.4	75.7	IV	3.6 ML	233	DOC

TABLE 2.5-2
Sheet 4 of 7

Year	Month	Day	Hour	Minutes	Seconds	Latitude (N)	Longitude (W)	MMI	Magnitude	Distance (km)	Reference
1934	04	15	02	58	13	44.7	73.8	V-VI	4.5 OTT	235	NYS
1883	03	12	-	-	-	45.1	74.5	-	3.7 ML	236	DOC
1897	03	07	-	-	-	43.1	79.2	-	3.7 ML	236	DOC
1950	03	20	22	55	11	41.5	75.8	-	3.3 ML	236	DOC
1954	04	27	02	14	08	43.1	79.2	V	4.1 ML	236	DOC
1963	03	02	20	24	32	41.5	75.7	-	3.4 ML	236	DOC
1958	05	06	19	00	-	42.7	73.8	-	3.6 ML	239	DOC
1952	03	27	06	35	-	43.0	79.3	V	3.0 ML	240	NYS
1950	08	04	14	29	29	45.2	74.7	IV	4.0	242	DOC
1979	06	07	13	45	53	44.4	73.9	-	3.1 MB	243	NYS
1882	11	27	23	30	-	43.0	79.2	-	3.7 ML	244	DOC
1960	01	22	20	53	22	41.5	75.5	-	3.4 ML	244	DOC
1894	12	17	08	-	-	42.4	73.8	IV-V	-	245	NYS
1933	01	21	16	04	39	45.3	74.7	IV	3.8 ML	246	DOC
1949	10	16	23	33	42	45.3	74.8	V	4.0	247	DOC
1941	04	04	08	10	-	44.7	73.9	-	3.3 ML	248	NYS
1898	01	07	06	00	00	45.1	74.3	-	3.7 ML	249	DOC
1873	04	25	19	-	-	44.8	74.2	V	4.4	250	DOC
1956	07	27	01	34	-	44.7	73.8	-	3.4 ML	250	NYS
1906	11	17	14	-	-	45.6	75.4	-	3.7 ML	251	DOC
1963	07	01	19	59	12	42.6	73.8	-	3.3 ML	251	NYS
1951	10	25	07	07	53	45.1	74.7	IV	3.8 ML	251	DOC
1853	03	13	10	-	-	43.1	79.4	-	4.4 ML	254	DOC
1932	03	09	05	23	38	45.5	74.7	-	3.8 ML	254	DOC
1956	01	10	12	08	18	45.7	75.5	III	3.3 ML	254	DOC
1956	02	02	19	24	16	45.5	74.8	III	3.1 ML	254	DOC
1976	07	13	03	51	14	45.2	74.1	-	3.1 MN	254	DOC
1873	07	06	14	30	-	43.0	79.5	V	5.0	255	DOC
1942	05	24	11	33	57	44.7	73.8	-	3.9 ML	255	NYS
1980	07	25	06	22	35	45.1	74.2	-	3.4 MN	255	NEN
1845	10	26	-	-	-	42.5	73.7	V-VI	5.0 ML	256	DOC
1927	10	24	11	00	00	44.7	73.8	-	3.7 ML	256	DOC
1897	05	28	03	16	-	44.5	73.5	VI	-	258	NYS
1877	12	18	10	-	-	45.7	78.9	V	4.5	259	DOC
1924	07	15	00	10	-	45.7	76.5	VI	4.5	261	DOC
1888	01	11	09	00	-	45.8	77.1	-	3.7 ML	264	DOC
1895	05	27	-	-	-	44.5	73.5	-	5.0	264	DOC
1958	07	22	01	46	40	43.0	79.5	V	4.3 ML	264	DOC
1855	02	07	04	30	-	42.0	74.0	V	5.0	266	DOC
1971	09	27	08	47	23	45.7	75.2	-	3.2 MN	266	DOC

TABLE 2.5-2
Sheet 5 of 7

Year	Month	Day	Hour	Minutes	Seconds	Latitude (N)	Longitude (W)	MMI	Magnitude	Distance (km)	Reference
1943	05	09	11	03	12	44.8	73.8	-	3.2 ML	267	NYS
1944	01	22	21	55	09	45.7	76.6	V	4.5	269	DOC
1951	11	06	17	54	46	44.9	73.6	-	4.1 MB	269	NYS
1966	06	25	00	05	51	45.2	73.8	-	3.4 ML	269	DOC
1954	02	21	20	00	-	41.2	75.9	VII	5.7 ML	272	DOC
1954	02	24	03	55	-	41.2	75.9	VI	5.0 ML	272	DOC
1952	10	08	21	40	-	41.7	74.0	V	-	273	NYS
1912	05	27	12	52	-	43.2	79.7	-	4.3 ML	274	DOC
1968	10	19	10	37	18	45.4	74.1	-	3.2 MN	274	DOC
1974	08	08	11	55	33	45.9	76.1	-	3.2 ML	274	DOC
1840	09	10	-	-	-	43.2	79.9	V	4.0	275	DOC
1972	12	16	19	01	36	45.8	75.2	-	3.9 MN	275	DOC
1871	01	03	-	-	-	45.6	74.6	-	4.4 ML	277	DOC
1953	03	31	12	58	34	44.1	73.1	-	4.0 ML	277	DOC
1980	09	21	04	07	32	45.6	74.4	-	3.2 MN	278	NEN
1915	02	21	23	41	-	44.7	73.4	-	3.7 ML	279	DOC
1948	06	09	03	04	12	45.2	73.9	IV	3.7 ML	279	DOC
1948	07	07	07	38	01	45.2	73.9	IV	3.5 ML	279	DOC
1953	04	26	01	20	-	44.7	73.5	-	3.7 ML	279	DOC
1955	10	07	18	09	52	45.2	73.9	IV	3.5 ML	279	DOC
1975	06	09	18	39	23	44.9	73.6	VI	3.5 MB	280	NYS
1910	02	25	-	-	-	43.2	79.8	-	3.7 ML	282	DOC
1954	04	21	15	45	-	44.7	73.5	-	3.6 ML	282	DOC
1952	01	30	04	00	-	44.5	73.2	-	5.0 ML	284	DOC
1955	02	03	02	30	-	44.5	73.2	V	4.3 ML	284	DOC
1974	06	07	19	45	37	41.6	73.9	V	3.3 ML	286	NYS
1939	01	14	08	10	16	43.3	79.9	III	3.3 ML	287	DOC
1962	04	10	14	30	48	44.2	73.1	-	5.0 ML	287	DOC
1958	09	30	00	13	58	45.2	73.7	IV	3.7 ML	287	DOC
1958	03	19	06	39	25	46.0	77.1	III	3.1 ML	287	DOC
1867	12	08	03	-	-	44.0	73.0	-	5.7 ML	288	DOC
1878	10	04	07	30	-	41.5	74.0	V	-	289	NYS
1880	09	06	05	30	-	45.2	73.8	-	3.7 ML	289	DOC
1913	06	08	06	30	-	46.1	74.4	-	3.7 ML	289	DOC
1873	04	30	-	-	-	43.3	79.9	-	3.7 ML	292	DOC
1938	09	07	23	18	19	45.9	74.9	IV	3.4 ML	292	DOC
1962	01	27	12	11	17	45.9	74.9	-	4.0	294	DOC
1978	07	30	10	54	44	45.7	74.3	-	3.8 MN	297	NEN
1980	01	24	05	44	49	46.1	75.1	-	3.2 MN	298	NEN
1941	10	24	14	13	59	45.7	74.3	IV	3.6 ML	299	DOC

TABLE 2.5-2
Sheet 6 of 7

Year	Month	Day	Hour	Minutes	Seconds	Latitude (N)	Longitude (W)	MMI	Magnitude	Distance (km)	Reference
1942	05	20	12	19	23	45.9	74.7	V	4.4 ML	299	DOC
1951	09	03	21	26	25	41.4	73.9	C	3.8 MB	299	NYS
1968	11	03	20	50	49	46.2	76.4	-	3.1 MN	299	DOC
1974	10	23	22	52	57	46.1	75.5	-	3.2 ML	299	DOC
1975	04	03	19	03	17	45.7	74.2	-	3.1 MN	299	DOC
1958	08	04	20	25	58	43.1	80.0	IV	3.9 ML	302	DOC
1877	11	04	06	56	-	45.4	73.9	VI	-	304	WES
1971	05	14	06	20	09	45.1	73.4	-	3.2 MN	305	DOC
1980	04	25	05	43	35	45.4	73.9	-	3.2 MN	305	NEN
1961	08	08	09	36	24	45.9	74.7	III	3.3 ML	305	DOC
1942	10	24	17	27	03	41.0	75.3	-	3.4 ML	307	DOC
1914	02	10	18	31	-	46.2	74.5	VII	5.5	309	DOC
1940	02	10	20	57	17	46.3	76.3	-	4.0	309	DOC
1961	09	15	02	16	56	40.7	75.7	V	4.0 ML	309	NOA
1974	11	02	13	47	56	46.1	75.0	-	3.2 ML	310	DOC
1943	07	06	22	10	14	44.9	73.1	-	4.1	311	DOC
1944	02	05	16	22	00	40.8	76.2	-	3.7 ML	312	DOC
1972	09	12	09	15	38	46.2	77.6	-	3.2 MN	315	DOC
1956	11	04	11	53	24	46.2	75.7	IV	4.0	316	DOC
1963	10	15	13	59	53	46.2	77.6	-	4.5	316	DOC
1861	10	-	-	-	-	45.6	73.7	-	4.4 ML	317	DOC
1951	09	25	15	45	00	46.2	75.4	-	3.7 ML	317	DOC
1950	04	06	16	14	11	46.0	74.5	-	4.0	318	DOC
1963	10	15	12	29	02	46.2	77.6	-	4.4 ML	318	DOC
1964	01	08	08	59	28	46.2	77.5	-	3.3 ML	319	DOC
1964	01	08	10	03	26	46.2	77.5	-	3.9 ML	319	DOC
1964	01	08	10	04	31	46.2	77.5	IV	3.8 MB	319	DOC
1844	11	-	-	-	-	45.5	73.6	-	3.7 ML	319	DOC
1864	10	21	09	10	-	45.5	73.6	-	3.7 ML	319	DOC
1974	12	02	10	58	05	46.3	75.5	-	3.5 ML	319	DOC
1977	07	14	07	39	30	46.0	74.4	-	3.4 MN	319	DOC
1879	06	11	-	-	-	45.6	73.6	-	3.7 ML	320	DOC
1897	03	26	05	04	-	45.5	73.6	-	3.7 ML	320	DOC
1909	02	01	08	20	-	45.5	73.6	-	3.7 ML	320	DOC
1937	11	12	14	43	44	45.9	74.3	-	3.6 ML	320	DOC
1937	11	12	16	57	33	45.9	74.4	IV	3.7 ML	320	DOC
1732	09	16	16	-	-	45.5	73.6	-	7.0	321	DOC
1816	09	02	-	-	-	45.5	73.6	-	5.7 ML	321	DOC
1816	09	16	-	-	-	45.5	73.6	-	5.7 ML	321	DOC
1959	04	13	21	20	19	41.9	73.3	-	3.4 ML	321	DOC
1971	12	18	15	36	24	46.2	74.7	IV	3.9 MN	322	DOC

KEY TO MAGNITUDE:

MB = Body wave
ML = Local (Richter magnitude)
MN = Nuttli
MC = Coda duration

KEY TO REFERENCES:

DOC = Earthquake Data File of the Department of Energy, Mines, and Resources, Earth Physics Branch, Ottawa, Canada.
NEN = Northeastern U.S. Seismic Network, Bulletins: Seismicity of the Northeastern United States, NUREG-WES-238 series, compiled by Western Observatory – Boston College, MA.
NOA = Earthquake Data File of the National Oceanographic and Atmospheric Administration, National Geophysical and Solar-Terrestrial Data Center, Boulder, CO.
NYS = Earthquake Data File of New York Museum and Science Service, Albany, NY (in press).
WES = Data File of Weston Geophysical, Inc., for various nuclear siting studies in northeastern United States.
WCC = Woodward Clyde Consultants[194].

TABLE 2.5-3
Sheet 1 of 3
IN SITU STRESS MEASUREMENTS BY OVERCORING

No.	Reference	Location	Horizontal Stress Maximum (psi)	Horizontal Stress Minimum (psi)	Average Orientation Maximum Horizontal Stress	Depth of Measurement (ft)
212	Hooker and Johnson	Proctor, VT				
		Borehole No. 1	1,427	470	N5E	0.5-1.8
		Borehole No. 2	1,132	517	N24W	0.5-1.8
213	Kaprowski	Sterling, NY				
		Borehole No. 1	1,200	850	N20W	33.2
			1,500	950	N50W	42.8
			1,450	750	N70W	46.7
			1,300	650	N50W	59.3
		(Average)	1,360	800	N50W	
214	Franklin and Hungr	Scarborough, Ont.	245	230	N90E	230
215	Dames & Moore	Somerset, NY				
		Borehole No. 1	640	460	N15W	27.7
			450	40	N45W	31.2
			295	60	N60W	32.6
			340	170	N15W	71.3
		(Average)	430	180	N34W	
		Borehole No. 2	450	240	N15W	
			265	-40	N10W	
			500	140	N15E	
		(Average)	400	110	N3W	
216	Palmer and Lo	Thorold, Ont.				
		Borehole No. 1	1,880	1,750	N27W	41.7
			1,310	756	N88W	51.0
			1,180	956	N62E	53.1
			1,300	966	N76E	56.6
			2,130	1,600	N60E	60.0
			2,130	1,620	N58E	61.0
			2,000	961	N56E	65.0
			1,520	990	N60E	81.0
		(Average)	1,680	1,200	N60E	
217	Lo	Mississauga, Ont.	NA	1,100	N-NE[1]	<85 ft[1]
218	Morton et al					

TABLE 2.5-3
Sheet 2 of 3

No.	Reference	Location	Horizontal Stress Maximum (psi)	Horizontal Stress Minimum (psi)	Average Orientation Maximum Horizontal Stress	Depth of Measurement (ft)
217 218	Lo Morton et al	Duffin Creek, Ont.	NA	1,000	N-NE[1]	<85 ft[1]
217 218	Lo Morton et al	Wesleyville, Ont.	1,600	1,400	N13W	<85 ft[1]
120	Sbar and Sykes	Niagara, NY Borehole No. 1[2]	1,000	-10	N34-55E	6-70
219	American Falls International Board	Borehole No. 2[2]	870	-330	N34-55E	6-70
214	Franklin and Hungr	Niagara, Ont. Borehole No. 1	1,495	NA	NA	115
220	Goldberg et al	Rochester, NY Borehole No. 1	800 1,600 2,000 4,000 4,300	500 1,000 600 1,400 1,500	N76E N80W N80W N78W N86E	24.5 35.9 40.8 45.1 46.4
		(Average)	2,540	1,000	N88W	
		Borehole No. 2[3]	900 1,000 1,900 2,400 1,700	-700 -208 -900 -400 300	N10E N62E N18E N39E N11E	32.2 37.3 46.1 50.6 55.4
		(Average)	1,580	-381	N28E	
		Borehole No. 3	3,000 3,000 -700 1,400	2,000 1,400 -1,400 900	N76E N32W N38W N22W	19.2 40.5 48.2 51.2
		(Average)	1,675	725	N49W	

TABLE 2.5-3
Sheet 3 of 3

No.	Reference	Location	Horizontal Stress Maximum (psi)	Horizontal Stress Minimum (psi)	Average Orientation Maximum Horizontal Stress	Depth of Measurement (ft)
81	NYSE&G	New Haven, NY				
		Borehole G-84	250	157	N78E	21.7
			754	440	N55W	24.5
			275	-71(T)	N21E	30.3
			708	510	N84W	38.0
			1,500	516	N40W	42.3
			767	438	N13W	50.6
		(Average)	709	332	N16W	
		Borehole G-85	356	94	N30E	21.2
			156	13	N04W	22.8
			223	38	N14E	25.5
			263	-84(T)	N29W	29.1
			693	259	N49W	35.6
			546	248	N48W	37.5
		(Average)	373	95	N14W	
		Borehole G-86	1,301	666	N38W	21.8
			443	263	N42W	31.2
			481	460	Hydrostatic	35.4
			900	772	N04W	38.7
			731	452	N20W	41.9
		(Average)	771	523	N26W	

(1) Orientation and depth reported by Morton et al[218].
(2) Sites close to the Niagara Gorge; orientation and magnitude of stress may be influenced by this fact. Note Borehole No.2 is closer to gorge.
(3) Data from Borehole No. 2 may be influenced by effects associated with a nearby open vertical fracture.

NOTE: Positive numbers indicate compressive stresses; negative numbers indicate tensile stresses.

TABLE 2.5-4
Sheet 1 of 1
OC-100 OVERCORE RESULTS

Test	Depth[1]	Rock Type	P (psi)	Q (psi)	θp	x 10^6 psi E_{AVG}	E_1	E_2	E_3	Remarks
1	11' 2"	Massive sandstone	-162	-318	N43E	4.7	4.7	4.4	5.1	
5	43' 6"	Shale/silty sandstone/silty sandstone with shaly laminae	+419	+210	N20E	2.9[2]	-	-	-	Modulus assumed from Test No. 6; core fractured at 43' 7" when recovered.
6	45' 0"	Graywacke	+395	+38	N37E	2.9	2.9	2.9	2.8	
9	49' 3"	Silty sandstone, interbedded shale, and shale clasts	+129	-331	N35E	3.1	3.2	3.2	2.8	Biaxial core fractured 1,000 psi. Buttons at silty sandstone/silty sandstone with shale clasts contact.
10[3]	52' 2"	Silty sandstone	+35(?)	-263(?)	N82W	1.8	2.4	1.7	1.2	Used 1/2" as zero; biaxial core fractured 1,600 psi of first load cycle.
10[4]	63' 1"	Silty sandstone with shaly laminae at 63' 2 3/4"	+157(?)	+87(?)	N43E	3.5	3.5	3.5	3.6	Biaxial test depth at 62' 10".

(1) Below casing collar.
(2) Assumed; negative values denote tensile stresses.
(3) Data termed uncertain due to drift of all axes at the end of test.
(4) Data termed uncertain due to drift of all axes at the beginning and end of test.

TABLE 2.5-5
Sheet 1 of 3
DENSITY VALUES FOR ROCK

PART I

Wet Density

Boring No.	Sample No.	Density (g/cc)	Rock Formation
Sandstone			
RT-1	3A	2.63	Oswego Sandstone
RT-1	4A	2.54	Oswego Sandstone
RT-1	6A	2.57	Oswego Sandstone
GP-1	1	2.58	Oswego Sandstone
GP-1	2	2.63	Oswego Sandstone
GP-1	4	2.65	Transition Zone
GP-1	12	2.62	Pulaski Unit A
GP-1	21	2.69	Pulaski Unit C
GP-1	30	2.79	Whetstone Gulf Unit A
GP-1	44	2.72	Whetstone Gulf Unit B
GP-1	49	2.72	Whetstone Gulf Unit B
Average Sandstone		2.65	
Siltstone			
RT-1	12A	2.65	Transition Zone
RT-2	2A	2.66	Transition Zone
RT-2	6A	2.63	Transition Zone
GP-1	5	2.68	Transition Zone
RT-1	56A	2.65	Pulaski Unit B
RT-1	56B	2.63	Pulaski Unit B
GP-1	22	2.67	Pulaski Unit C
GP-1	28	2.68	Whetstone Gulf Unit A
GP-1	36	2.70	Whetstone Gulf Unit B
GP-1	53	2.69	Whetstone Gulf Unit B
Average Siltstone		2.66	
Graywacke			
GP-1	7	2.67	Pulaski Unit A
GP-1	9	2.65	Pulaski Unit A
GP-1	13	2.67	Pulaski Unit A
GP-1	14	2.79	Pulaski Unit A
RT-1	17A	2.71	Pulaski Unit A
RT-1	20A	2.68	Pulaski Unit A
RT-1	26A	2.68	Pulaski Unit A
RT-1	28A	2.69	Pulaski Unit A

TABLE 2.5-5
Sheet 2 of 3

Boring No.	Sample No.	Density (g/cc)	Rock Formation
RT-1	28B	2.58	Pulaski Unit A
RT-1	47A	2.68	Pulaski Unit A
Average Graywacke		2.67	
Shale			
RT-2	11B	2.66	Transition Zone
RT-2	12B	2.68	Transition Zone
RT-1	35A	2.66	Pulaski Unit A
RT-1	35B	2.67	Pulaski Unit A
RT-1	35C	2.66	Pulaski Unit A
GP-1	6	2.67	Pulaski Unit A
RT-1	61A	2.67	Pulaski Unit C
RT-1	61B	2.63	Pulaski Unit C
RT-1	62B	2.68	Pulaski Unit C
GP-1	18	2.62	Pulaski Unit C
GP-1	24	2.68	Pulaski Unit C
GP-1	27	2.68	Whetstone Gulf Unit A
GP-1	29	2.68	Whetstone Gulf Unit A
GP-1	35	2.68	Whetstone Gulf Unit B
GP-1	39	2.70	Whetstone Gulf Unit B
GP-1	42	2.70	Whetstone Gulf Unit B
GP-1	47	2.71	Whetstone Gulf Unit B
GP-1	55	2.70	Whetstone Gulf Unit B
Average Shale		2.68	
Mixed Samples			
RT-1	36A	2.63	Pulaski Unit A sandstone and siltstone
GP-1	15	2.67	Pulaski Unit B sandstone and shale

TABLE 2.5-5
Sheet 3 of 3

PART II

Dry Density

Boring No.	Sample No.	Density (g/cc)	Rock Type	Rock Formation
RT-1	5A	2.66	Sandstone/ shale	Oswego
RT-1	7A 2.64	Sandstone/	Oswego shale	
RT-1	7B 2.64	Sandstone/	Oswego shale	
RT-1	13A	2.63	Shale	Transition Zone
RT-1	13B	2.63	Shale	Transition Zone
RT-1	19B	2.64	Shale	Pulaski Unit A
RT-1	25A	2.66	Sandstone/ graywacke	Pulaski Unit A
RT-1	34B	2.63	Shale	Pulaski Unit A
RT-1	53A	2.59	Shale	Pulaski Unit B
RT-1	54A	2.64	Shale	Pulaski Unit B
RT-1	54B	2.63	Shale	Pulaski Unit B

TABLE 2.5-6
Sheet 1 of 1
IN SITU ELASTIC PROPERTIES FROM CROSSHOLE SURVEYS
BORING GP-1 TO BORING GP-2

(Time Correction Applied)

Depth (ft)	Distance (ft)			Compres-sional Time (msec)	Shear Time (msec)	Compressional Velocity (ft/sec)			Shear Velocity (ft/sec)			Specific Gravity (g/cc)	Shear Modulus (psi x 10^6)			Poisson's Ratio
	Min	Max	Avg			Min	Max	Avg	Min	Max	Avg		Min	Max	Avg	
0	30.95	30.95	30.95		0.00											
21	31.00	31.10	31.05	2.24	6.28	13478	14266	13862	5805	6051	5926	2.625	1.193	1.295	1.242	0.388
25	31.05	31.10	31.07	2.24	5.92	13500	14266	13873	6235	6506	6368	2.671	1.400	1.524	1.460	0.367
35	31.10	31.15	31.12	2.24	5.78	13522	14289	13895	6426	6713	6566	2.655	1.478	1.613	1.543	0.356
40	31.15	31.25	31.20	2.12	5.78	14289	15170	14717	6436	6735	6582	2.698	1.506	1.650	1.576	0.375
55	31.20	31.35	31.27	2.20	5.08	13805	14650	14216	7536	7957	7741	2.684	2.055	2.291	2.168	0.289
66	31.25	31.50	31.37	2.20	5.04	13827	14720	14261	7622	8077	7844	2.668	2.089	2.346	2.213	0.283
70	31.30	31.50	31.40	2.20	4.90	13850	14720	14273	7904	8378	8135	2.686	2.262	2.541	2.396	0.259
85	31.25	31.70	31.48	1.96	4.86	15470	16684	16059	7972	8522	8240	2.711	2.322	2.654	2.481	0.321
100	31.45	31.85	31.65	2.16	5.18	14167	15167	14653	7417	7884	7645	2.668	1.979	2.235	2.102	0.313
115	31.60	32.10	31.85	2.12	4.96	14495	15583	15024	7861	8403	8125	2.652	2.209	2.524	2.360	0.293
125	31.70	32.20	31.95	2.00	4.48	15388	16598	15975	8955	9641	9288	2.692	2.910	3.373	3.130	0.245
130	31.70	32.25	31.98	2.00	4.84	15388	16624	15987	8128	8716	8414	2.681	2.388	2.746	2.559	0.308
137	31.80	32.35	32.07	2.04	4.76	15143	16338	15723	8325	8936	8622	2.667	2.491	2.871	2.673	0.285
145	31.85	32.45	32.15	2.20	4.92	14093	15164	14614	8003	8585	8286	2.721	2.349	2.703	2.518	0.263
160	31.85	32.55	32.20	2.04	4.88	15167	16439	15784	8084	8703	8385	2.708	2.385	2.765	2.567	0.303
175	31.95	32.65	32.30	2.20	4.84	14137	15257	14682	8192	8824	8500	2.651	2.398	2.783	2.582	0.248
190	32.10	32.80	32.45	2.08	4.84	15000	16238	15601	8231	8865	8539	2.661	2.430	2.819	2.616	0.286
205				2.16	4.84	145	157	151	82	89	85	2.670	2.4	2.8	2.6	0.261
220				2.12	4.92							2.683				0.287
232				2.20	4.88							2.736				0.256
235				1.98	4.88							2.742				0.319
250					4.80							2.714				
265				2.14	4.88							2.721				0.275
280				2.00	4.68							2.735				0.284
295				2.08	4.72							2.712				0.265
300				2.12	4.84							2.736				0.274
310				2.16	4.92							2.711				0.275
316				2.08	4.72							2.717				0.265
321				2.18	4.96							2.727				0.276
340				2.16	5.20							2.710				0.315
353				2.08	5.24							2.721				0.337

TABLE 2.5-7
Sheet 1 of 1
IN SITU ELASTIC PROPERTIES FROM CROSSHOLE SURVEYS
BORING GP-1 TO BORING RT-1

(Time Correction Applied)

Depth (ft)	Distance (ft)			Compres-sional Time (msec)	Shear Time (msec)	Compressional Velocity (ft/sec)			Shear Velocity (ft/sec)			Specific Gravity (g/cc)	Shear Modulus (psi x 10^6)			Poisson's Ratio
	Min	Max	Avg			Min	Max	Avg	Min	Max	Avg		Min	Max	Avg	
0	15.50	15.50	15.50													
21	15.30	15.35	15.32	1.14	3.50	12750	14213	13443	5977	6504	6230	2.625	1.264	1.497	1.373	0.363
25	15.25	15.35	15.30	1.16	3.56	12500	13955	13190	5821	6343	6071	2.671	1.220	1.449	1.327	0.366
35	15.05	15.10	15.07	1.14	4.06	12542	13981	13224	4824	5171	4992	2.655	.833	.957	.892	0.417
40	14.95	15.05	15.00	1.02	3.16	13843	15677	14706	6734	7450	7075	2.698	1.649	2.019	1.821	0.349
55	14.70	14.80	14.75	1.12	2.92	12458	13962	13170	7424	8315	7846	2.684	1.994	2.501	2.227	0.225
66	14.50	14.60	14.55	1.14	2.92	12083	13619	12763	7323	8202	7739	2.668	1.929	2.420	2.154	0.209
70	14.45	14.60	14.52	1.10		12457	14038	13205				2.686				
85	14.30	14.45	14.38	0.96	2.78	14020	16056	14974	7772	8811	8261	2.711	2.207	2.837	2.494	0.281
100	14.20	14.35	14.27	1.04	3.02	12909	14643	13726	6827	7633	7210	2.668	1.676	2.095	1.869	0.310
115	14.20	14.35	14.27	1.04	2.68	12909	14643	13726	7320	8247	7758	2.652	1.915	2.431	2.152	0.265
125	14.20	14.40	14.30	1.00	2.76	13396	15319	14300	7802	8889	8314	2.692	2.209	2.867	2.508	0.245
130	14.25	14.45	14.35	0.92	2.76	14541	16802	15598	7830	6920	8343	2.681	2.216	2.875	2.516	0.300
137	14.30	14.55	14.43	0.96	2.68	14020	16167	15026	8218	9448	8796	2.667	2.428	3.209	2.781	0.239
145	14.40	14.60	14.50	1.04	2.80	13091	14898	13942	7742	8795	8239	2.721	2.198	2.837	2.490	0.232
160	14.65	14.80	14.72	0.92	2.94	14949	17209	16005	7325	8222	7750	2.708	1.959	2.466	2.193	0.347
175	15.05	15.10	15.07	1.10	3.04	12974	14519	13705	7167	7947	7537	2.651	1.835	2.257	2.030	0.283
190	15.50	15.65	15.57	1.06	2.96	13839	15650	14693	7673	6599	8112	2.661	2.112	2.652	2.360	0.281
205				1.12	2.92	134	153	143	8030	9129	8551	2.670	2.3	3.0	2.6	0.225
220				1.18	3.12							2.683				0.263
232				1.16	3.28							2.736				0.317
235				1.12	3.16							2.742				0.306
250				1.12	3.12							2.714				0.296
265				1.24	3.38							2.721				0.305
280				1.14	3.38							2.735				0.329
285				1.30	3.42							2.712				0.287
300				1.24	3.44							2.736				0.318
310				1.32	3.48							2.711				0.293
316				1.34	3.54							2.717				0.307
321				1.36	3.76							2.727				0.333

TABLE 2.5-8
Sheet 1 of 1
IN SITU ELASTIC PROPERTIES FROM CROSSHOLE SURVEYS
BORING RT-1 TO BORING GP-2

(Time Correction Applied)

Depth (ft)	Distance (ft)			Compres-sional Time (msec)	Shear Time (msec)	Compressional Velocity (ft/sec)			Shear Velocity (ft/sec)			Specific Gravity (g/cc)	Shear Modulus (psi x 10^6)			Poisson's Ratio
	Min	Max	Avg			Min	Max	Avg	Min	Max	Avg		Min	Max	Avg	
0	27.27	27.27	27.27													
21	26.85	26.90	26.98	2.00		13034	13866	13437				2.625				
25	26.65	26.70	26.57	1.88	0.00	13737	14670	14189				2.671				
35	26.30	26.40	26.35	1.92	4.32	13283	14194	13724	7781	8302	8034	2.655	2.167	2.467	2.310	0.239
40	25.10	26.20	26.15	1.84	4.16	13737	14719	14212	8106	8675	8381	2.698	2.389	2.737	2.555	0.234
55	25.45	25.65	25.55	1.72	4.32	14298	15452	14855	7530	8066	7790	2.684	2.051	2.354	2.195	0.310
66	24.95	25.20	25.08	1.90	4.28	13414	14483	13931	7470	8025	7789	2.668	2.007	2.316	2.154	0.277
70	24.75	25.00	24.87	1.76	4.22	13599	14706	14134	7546	8117	7822	2.686	2.062	2.385	2.215	0.279
85	23.95	24.40	24.17	1.50	4.00	14428	15844	15109	7827	8531	8167	2.711	2.239	2.660	2.438	0.294
100	23.35	23.75	23.55	1.54	3.98	13735	15032	14360	7681	8363	8010	2.668	2.122	2.515	2.808	0.274
115	22.70	23.15	22.92	1.56	3.68	14012	15433	14696	8285	9114	8684	2.652	2.454	2.970	2.696	0.232
125	22.25	22.70	22.48	1.44	3.80	14833	16449	15608	7780	8534	8143	2.692	2.196	2.643	2.406	0.313
130	22.00	22.45	22.23	1.40	3.68	15068	16754	15875	8029	8839	8419	2.681	2.330	2.823	2.561	0.304
137	21.70	22.15	21.92	1.40	3.72	14863	16580	15661	7806	8585	8181	2.667	2.190	2.650	2.406	0.312
145	21.20	21.75	21.48	1.56	3.94	13086	14900	13766	7067	7768	7405	2.721	1.832	2.213	2.011	0.296
160	20.40	21.10	20.75	1.22	3.78	15938	18190	17008	7183	7992	7573	2.708	1.883	2.332	2.094	0.376
175	19.65	20.40	20.02	1.40	3.72	13459	15224	14304	7068	7907	7472	2.651	1.785	2.234	1.995	0.312
190	18.80	19.70	19.25	1.32	3.36	13623	15635	14583	7769	8874	8297	2.661	2.165	2.825	2.470	0.261
205				1.28	3.30	134	154	143	7627	8727	8153	2.670	2.094	2.741	2.392	0.261
220				1.18	3.12				78	919	85	2.683	2.2	3.0	2.6	0.263
232				1.16	3.04							2.736				0.246
235				1.08	3.08							2.742				0.305
250				1.00	2.82							2.714				0.269
265				1.04	2.86							2.721				0.258
280				0.96	2.80							2.735				0.288
295				0.88	2.64							2.712				0.283
300				0.92	2.56							2.736				0.211
310				0.88	2.64							2.711				0.283
316				0.84	2.52							2.717				0.262
321				0.82	2.50							2.727				0.269

Table 2.5-9
Sheet 1 of 2
POROSITY AND WATER CONTENT

Boring No.	Sample No.	Porosity Percent	Water Content Percent	Rock Formation
Sandstone				
GP-1	1	--	1.9	Oswego
GP-1	2	--	1.6	Oswego
GP-1	4	--	1.4	Transition Zone
GP-1	12	--	1.2	Pulaski Unit A
GP-1	21	--	0.4	Pulaski Unit C
GP-1	30	--	0.3	Whetstone Gulf Unit A
GP-1	44	--	0.6	Whetstone Gulf Unit B
GP-1	49	--	1.0	Whetstone Gulf Unit B
Siltstone				
GP-1	5	--	2.7	Transition Zone
GP-1	22	--	2.1	Pulaski Unit C
GP-1	28	--	1.44	Whetstone Gulf Unit A
GP-1	36	--	1.89	Whetstone Gulf Unit B
GP-1	53	--	1.9	Whetstone Gulf Unit B
Graywacke				
GP-1	7	--	1.8	Pulaski Unit A
GP-1	9	--	2.03	Pulaski Unit A
GP-1	13	--	1.45	Pulaski Unit A
Shale				
RT-1	13A	3.42	1.28	Transition Zone
RT-1	13B	5.64	1.93	Transition Zone
RT-1	19B	4.49	1.71	Pulaski Unit A
RT-1	34B	3.66	1.32	Pulaski Unit A
RT-1	53A	4.38	1.71	Pulaski Unit B
RT-1	54A	5.00	1.82	Pulaski Unit B
RT-1	54B	5.00	2.01	Pulaski Unit B
GP-1	24	--	2.65	Pulaski Unit C
GP-1	27	--	1.85	Whetstone Gulf Unit A
GP-1	29	--	2.34	Whetstone Gulf Unit A
GP-1	35	--	2.05	Whetstone Gulf Unit B

Table 2.5-9
Sheet 2 of 2

Boring No.	Sample No.	Porosity Percent	Water Content Percent	Rock Formation
GP-1	39	--	1.72	Whetstone Gulf Unit B
GP-1	42	--	1.68	Whetstone Gulf Unit B
GP-1	47	--	1.56	Whetstone Gulf Unit B
GP-1	55	--	1.24	Whetstone Gulf Unit B
Mixed Samples				
Sandstone/Shale				
RT-1	7A	2.35	0.89	Oswego
RT-1	5A	5.00	1.87	Oswego
Sandstone/Shale				
RT-1	7B	3.50	1.33	Oswego
Sandstone/Graywacke				
RT-1	25A	3.58	1.29	Pulaski Unit A

TABLE 2.5-10
Sheet 1 of 1
MINERALOGICAL COMPOSITION
ESTIMATED MODAL ANALYSIS BY PETROGRAPHIC EXAMINATION OF THIN SECTIONS

Sample*	Depth	Rock Type (by Thin Section)	Geologic Unit	Component (%)				
				Quartz	Chert	Feldspar	Rock Fragments	Matrix and Accessory Minerals
SR-1	21'11" (6.68 m)	Submature lithic arenite	Oswego Sandstone	60	5	7	20	8
SR-2	23'0" (7.01 m)	Submature lithic arenite	Oswego Sandstone	53	7	8	25	7
SR-3	26'10" (8.18 m)	Lithic graywacke	Transition Zone	45	3	10	25	17
SR-4	27'11" (8.51 m)	Lithic graywacke	Transition Zone	46	2	5	30	17
SR-5	91'0" (27.74 m)	Shale	Pulaski Unit B	25	-	1	-	74
SR-6	92'3" (28.12 m)	Shale	Pulaski Unit B	25	-	1	-	74

* All samples are from Boring TD-1.

TABLE 2.5-11
Sheet 1 of 1
MINERALOGICAL COMPOSITION
MODAL ANALYSIS BY X-RAY DIFFRACTION

Sample	Rock Type (by thin section)	Mineral (%)					
		Silica, Quartz, and Chert	Sodic Feldspar	Chlorite Group	Illite Group	Kaolinite	Other Undifferentiated or as Noted
SR-1	Submature lithic arenite	59	10	8	6	15	2
SR-2	Submature lithic arenite	58	10	10	6	15	1
SR-3	Lithic graywacke	52	12	9	7	17	3
SR-4	Lithic graywacke	50	9	11	9	18	3
SR-5	Shale	34	2	8	19	27	10*
SR-6	Shale	32	2	13	22	23	8*

* Includes a major percentage of halloysite, and probably organic material and rock fragments.

TABLE 2.5-12
Sheet 1 of 2
MODULI FROM SONIC TESTING

Sample No.	Depth	Rock Type	Compression Wave Velocity (V_p) (ft/sec)	Shear Wave Velocity (V_s) (ft/sec)	Uniaxial Stress (psi)	Poisson's Ratio (υ)	Shear Modulus G (x 10^6 psi)	Young's Modulus E (x 10^6 psi)
GP-1-1	12'10" - 13'15"	Sandstone	13,560	8,782	10.0	0.139	2.676	6.096
GP-1-2	25' - 25'6"	Sandstone	15,171	9,435	10.0	0.185	3.150	7.464
GP-1-4	25'11" - 29'8"	Sandstone	14,429	9,829	20.0	0.201	2.787	6.692
GP-1-5	30'6" - 30'11"	Siltstone	11,488	5,028	20.0	0.382	0.912	2.520
GP-1-12	60'2" - 61'2"	Sandstone	15,215	9,819	40.0	0.143	3.400	7.773
GP-1-14	82'1" - 82'9"	Graywacke	18,397	11,238	60.0	0.202	4.744	11.409
GP-1-21	101' - 109'6"	Sandstone	17,372	10,993	75.0	0.166	4.385	10.226
GP-1-30	158'2" - 158'11"	Sandstone	18,728	11,529	110.0	0.195	5.002	11.952
GP-1-44	280'4" - 281'	Sandstone	16,891	10,941	800.0	0.139	4.387	9.991
GP-1-49	322'6" - 323'	Silicious	16,354	10,457	225.0	0.154	4.004	9.242
GP-1-7	40'7" - 41'5"	Graywacke	13,956	8,767	30.0	0.174	2.765	6.491
GP-1-9	53'5" - 53'11"	Graywacke	12,574	7,718	40.0	0.198	2.123	5.087
GP-1-13	70'11" - 71'10"	Graywacke	13,671	8,271	25.0	0.211	2.465	5.971
GP-1-22	110'11" - 111'10"	Sandy siltstone	12,444	5,923	70.0	0.354	1.263	3.419
GP-1-53	339'7" - 340'2"	Sandy siltstone	14,389	9,486	240.0	0.116	3.265	7.286
GP-1-42	251'2" - 252'0"	Shale	11,585 11,585	6,936 6,936	151.3 175.0	0.221 0.221	1.746 1.746	4.263 4.263
GP-1-29	155' - 155'4"	Shale	10,485 10,042	6,347 6,182	151.6 110.0	0.211 0.195	1.448 0.374	3.506 3.283
GP-1-6	39'8" - 40'0"	Shale	11,495 11,495	6,745 7,056	151.5 30.0	0.237 0.198	1.636 1.790	4.049 4.289

TABLE 2.5-12
Sheet 2 of 2

Sample No.	Depth	Rock Type	Compression Wave Velocity (V_p) (ft/sec)	Shear Wave Velocity (V_s) (ft/sec)	Uniaxial Stress (psi)	Poisson's Ratio (υ)	Shear Modulus G (x 10^6 psi)	Young's Modulus E (x 10^6 psi)
GP-1-47	303'5" - 304'0"	Silty shale	11,351 11,351	6,946 6,946	152.8 210.0	0.201 0.201	1.762 1.762	4.232 4.232
GP-1-24	121’9” - 122’4”	Shale	10,854	6,702	151.7	0.192	1.624	3.871
GP-1-55	350’1” - 350’6”	Silty shale	11,847	6,977	245.0	0.235	1.768	4.365
GP-1-36		Siltstone	10,881	6,367	150.9	0.240	1.475	3.658
GP-1-27		Shale	11,388 9,622	5,958 5,864	302.1 100.0	0.312 0.205	1.279 1.239	3.355 2.985
GP-1-15	88'11" - 89'5"	Interbedded sandstone and shale	12,003	7,266	151.0	0.211	1.902	4.605
GP-1-28	152'6" - 153'8"	Sandy siltstone	13,394	7,952	151.0	0.228	2.285	5.612
GP-1-35	195'0" - 195'8"	Shale	11,481	7,010	150.8	0.203	1.775	4.270
GP-1-39	239'3" - 239'8"	Silty sandy shale	12,140	7,461	301.8	0.196	2.023	4.841

TABLE 2.5-13
Sheet 1 of 1
TRIAXIAL TEST RESULTS (T-3-3 SERIES)

Results from Triaxial Compression Testing of Rock Samples from Boring T-3-3

Rock Type	Formation	Confining Pressure		Sample Pore Pressure		Young's Modulus		Maximum Stress	
		psi	kg/cm^2	psi	kg/cm^2	psi x 10^6	x $10^5 kg/cm^2$	psi	kg/cm^2
Siltstone/shale	Pulaski Unit A	200	14	0		4.56	3.21	11,659	820
Siltstone/shale	Pulaski Unit A	200	14	0		4.62	3.25	15,697	1,103
Sandstone	Pulaski Unit A	200	14	0		6.70	4.71	37,435	2,632
Sandstone	Pulaski Unit A	200	14	0		6.71	4.72	40,906	2,877
Silty shale	Pulaski Unit B	200	14	0		3.64	2.56	10,410	2,876
Silty shale	Pulaski Unit B	200	14	0		4.10	2.88	10,532	740
Silty shale	Pulaski Unit B	400	28	0		3.60	2.53	13,536	952
Silty shale	Pulaski Unit B	400	28	200	14	3.98	2.80	7,459	524
Silty shale	Pulaski Unit B	400	28	200	14	3.90	2.74	8,128	571

NOTE: Long axis of cylindrical samples is perpendicular to bedding.

TABLE 2.5-14
Sheet 1 of 1
TRIAXIAL TEST RESULTS (30 DEGREES TO BEDDING)

Modulus Values from Triaxial Testing of Rock Samples
from Boring T-4-7

Confining Pressure (psi)	Sample Pore Pressure (psi)	Young's Modulus (psi x 10^6)	Maximum Stress (psi)
200	0	-	1,665
400	200	0.76	2,224
200	0	-	2,921
400	200	0.86	2,648
200	0	1.30	2,623
200	0	1.13	2,775
400	200	0.85	2,275
200	0	1.20	3,180
400	200	0.98	2,805
200	0	1.19	3,655
400	200	1.07	3,956
400	200	0.59	1,894
200	0	0.95	2,366

NOTE: Rock type was interbedded siltstone and shale from Unit B of the Pulaski formation.

TABLE 2.5-15
Sheet 1 of 1
TRIAXIAL TEST RESULTS (RT SERIES)

Test No.	Sample No.	Rock Type	Relation of Test Specification Axis to Bedding (deg)*	Confining Pressure (psi)	Maximum Point Differential Stress (psi)	Maximum Point Strain (%)	Young's Modulus (psi x 10^6)	Maximum Strain Under Confining Pressure (%)
1	RT 1-1	Sandstone	90	50	31,745	0.81	5.63	5.44
2	RT 1-1		90	750	37,012	0.97	5.62	5.75
3	RT 1-1		90	1,500	39,055	1.06	5.06	6.41
4	RT 2-7	Siltstone	90	50	9,588	0.78	1.61	3.03
5	RT 2-7		90	750	11,586	0.92	1.70	2.79
6	RT 2-7		90	1,500	12,408	1.05	1.75	3.12
7	RT 2-7		45	50	7,073	0.52	1.60	2.53
8	RT 2-7		45	750	7,864	0.67	1.58	3.01
9	RT 2-7		45	1,500	7,571	0.69	1.62	2.68
10	RT 2-7		0	50	9,861	0.47	2.42	2.85
11	RT 2-7		0	750	12,016	0.62	2.52	3.06
12	RT 2-7		0	1,500	12,499	0.74	2.42	3.11
13	RT 1-22	Graywacke	90	50	8,583	1.03	1.28	3.27
14	RT 1-22		90	750	9,485	1.15	1.44	3.25
15	RT 1-22		90	1,500	9,848	1.16	1.48	3.22
16	RT 1-22		45	50	4,114	0.53	0.81	2.06
17	RT 1-22		45	750	3,990	0.59	0.87	2.29
18	RT 1-22		45	1,500	5,913	0.94	1.00	3.43
19	RT 1-22		0	50	4,857	0.38	1.60	2.26
20	RT 1-22		0	750	8,286	0.58	1.98	3.87
21	RT 1-22		0	1,500	9,406	0.71	2.09	3.96
22	RT 1-51		90	50	12,054	0.90	1.96	3.43
23	RT 1-51		90	750	10,393	1.02	1.70	3.43
24	RT 1-51		90	1,500	12,250	1.22	1.58	3.49
25	RT 1-51		45	50	5,668	0.50	1.32	3.26
26	RT 1-51		45	750	7,116	0.69	1.46	2.71
27	RT 1-51		45	1,500	11,208	0.86	1.77	3.39
28	RT 1-51		0	50	9,488	0.40	2.96	2.97
29			0	750	10,628	0.57	2.37	2.96
30			0	1,500	11,155	0.67	2.37	3.16

* Bedding is taken to be perpendicular to the bulk sample core axis.

TABLE 2.5-16
Sheet 1 of 2
UNIAXIAL MODULUS DETERMINATIONS

Specimen No.	Depth (ft)	Rock and Formation	i^0*	10^6 psi E_z	u_{zx}	10^6 psi E_y	u_{yx}	u_{yz}	10^6 psi E_i	Stress Range (psi)
RT1-3A	18.9-19.1	Sandstone Oswego Sandstone	90	4.08 4.12 4.42	0.168 0.168 0.167					2nd loading 218-1865 2nd unloading 4662-311 3rd loading 311-3108
RT1-4A	20.5-20.65	Sandstone Oswego Sandstone	0			5.20 3.15 5.22	0.292 0.445 0.320	0.096 0.125 0.114		2nd loading 377-1840 2nd unloading 1950-220 3rd loading 220-3145
RT1-6A	23.9-24.25	Sandstone Oswego Sandstone	45						2.37 1.79 2.60	2nd loading 109-1876 2nd unloading 1076-125 3rd loading 125-3127
RT1-12A	31.3-31.5	Siltstone (sandy) Transition Zone	0			4.97 4.41 5.45	0.270 0.297 0.270	0.403 0.446 0.393		2nd loading 156-1907 2nd unloading 1907-156 3rd loading 156-3127
RT2-2A	25.55- 25.95	Siltstone (shaly) Transition Zone	90	1.12 1.12 1.14	0.330 0.333 0.321					2nd loading 125-1870 2nd unloading 1870-125 3rd loading 125-1870
RT2-6A	26.9-27.3	Siltstone (sandy) Transition Zone	45						3.29 3.56 3.42	2nd loading 140-2341 2nd unloading 2341-94 3rd loading 94-1873
RT1-56A	96.7-97.1	Siltstone (shaly) Pulaski B	0			1.93	0.177	0.428		1st loading 188-1407 (specimen failed)
RT1-56B	96.7-97.1	Siltstone (shaly) Pulaski B	0			4.70 4.65 3.91	0.331 0.416 0.360	0.437 0.489 0.485		2nd loading 63-1408 2nd unloading 1408-125 3rd loading 125-1940
RT1-17A	41.6-42.0	Graywacke Pulaski A	90	6.27 4.82 6.28	0.331 0.350 0.330					2nd loading 94-1907 2nd unloading 1970-94 3rd loading 94-3127
RT1-20A	46.0-46.4	Graywacke (silty) Pulaski A	0			5.93 4.48 7.15	0.379 0.654 0.284	0.632 0.726 0.619		2nd loading 188-1878 2nd unloading 1878-156 3rd loading 156-1565

TABLE 2.5-16
Sheet 2 of 2

Specimen No.	Depth (ft)	Rock and Formation	i^0*	10^6 psi E_z	u_{zx}	10^6 psi E_y	u_{yx}	u_{yz}	10^6 psi E_i	Stress Range (psi)
RT1-26A	51.2-51.35	Graywacke (shaly) Pulaski	45						3.20 3.18 3.57	2nd loading 1876-31 2nd unloading 1876-31 3rd loading 31-3127
RT1-36A	1.55-61.95	Silty sandstone and 247reywacke (silty) Pulaski Unit 1	90	7.26 6.76 7.94	0.175 0.242 0.211					2nd loading 157-1942 2nd unloading 1911-125 3rd loading 125-1927
RT1-61A	105.7- 106.1	Shale (silty) Pulaski C	90	2.68 2.85 2.81	0.682 0.656 0.655					2nd loading 111-2222 2nd unloading 2222-167 3rd loading 167-2222
RT1-61B	105.7- 106.1	Shale (silty) Pulaski C	0			4.61 4.07 4.71	0.437 0.423 0.401	0.517 0.537 0.524		2nd loading 56-1669 2nd unloading 1669-167 3rd loading 167-1669
RT1-62D	106.4- 106.9	Shale (silty) Pulaski C	45						1.39 1.46 1.43	2nd loading 56-1680 2nd unloading 1680-112 3rd loading 112-1680

*i^0 = Inclination of core to bedding.

TABLE 2.5-17
Sheet 1 of 1
YOUNG'S MODULUS DETERMINED BY BIAXIAL TEST

(modulus x 10^6 psi)

Lithologic Unit	Rock Type Tested	No. Tests	Minimum	Mean	Maximum	Standard Deviation (x 10^6 psi)
Oswego Sandstone	Sandstone	37	1.6	4.3	8.5	1.53
Transition Zone of Oswego Sandstone	Sandstone, shale, and graywacke	17	2.7	5.1	7.9	1.25
Pulaski Formation Unit A	Graywacke and sandstone	52	1.8	4.6	8.3	1.28
Pulaski Formation Unit B	Sandstone	5	3.2	4.1	4.9	0.70
Pulaski Formation Unit C	Shale, siltstone, and sandstone	31	1.9	3.4	7.1	0.99
Whetstone Gulf Formation Unit A	Sandstone	2	4.3	4.3	4.4	0.1
Whetstone Gulf Formation Unit B	Sandstone	4	4.1	4.7	5.8	0.8
Total		148	1.6	4.3	8.5	1.34

TABLE 2.5-18
Sheet 1 of 3
SWELL TEST RESULTS

Sample No.	Test No.	Rock Type	Rock Unit	Test Type	Vertical Stress σ_{υ} (psi)	Horizontal Stress σ R(psi)	First Stress Invariant $\sigma_{\upsilon} + 2\sigma_{h}$ (psi)	Vertical Swell Strain Rate (10^{-2}%/ $\log_{10}$ day)	Horizontal Swell Strain Rate (10^{-2}%/ $\log_{10}$ day)
RT2-1	5	Sandstone	Oswego	Swell Strain	25	75-140	175-305	0	0
TD-1	1	Sandstone	Oswego	Free Swell	0	0	0	0	0
TD-2	1	Sandstone	Oswego	Free Swell	0	0	0	0	0
RT1-14	4	Shale	Transition Zone	Swell Strain	37	44-75	125-187	12	1.7
RT1-14	4	Shale	Transition Zone	Swell Strain	19	88-107	195-233	12	0.9
RT1-15	8	Shale with siltstone lenses	Transition Zone	Swell Strain	37	60-112	157-261	11	1.2
RT1-15	8	Shale with siltstone lenses	Transition Zone	Swell Strain	19	110-140	254-299	11	0.55
RT1-9	18	Siltstone	Transition Zone	Swell Strain	72	56-72	180-220	0	0.7
RT1-9	18	Siltstone	Transition Zone	Swell Stress	85	75	238	0	0
RT1-10	25	Siltstone	Transition Zone	Swell Stress	70	75-111	220-292	0	0.75
RT1-10	25	Siltstone	Transition Zone	Swell Stress	120	104	328	0	0
RT2-4	21	Sandy siltstone	Transition Zone	Swell Stress	115	90-150	295-415	0	1.65
RT2-4	21	Sandy siltstone	Transition Zone	Swell Stress	115	155-178	425-471	0	0.4
TD-1	3	Graywacke	Transition Zone	Free Swell	0	0	0	12.4	4.2
TD-1	4	Shale siltstone	Transition Zone	Free Swell	0	0	0	10.4	3.2
TD-1	5	Sandstone	Transition Zone	Free Swell	0	0	0	0	0.35
RT1-23	6	Graywacke	Pulaski A	Swell Strain	47	110-130	267-307	4.0	0.9
RT1-23	6	Graywacke	Pulaski A	Swell Strain	23	130-148	283-319	3.0	0.8

TABLE 2.5-18
Sheet 2 of 3

Sample No.	Test No.	Rock Type	Rock Unit	Test Type	Vertical Stress σ_{υ} (psi)	Horizontal Stress σ R(psi)	First Stress Invariant $\sigma_{\upsilon} + 2\sigma_{h}$ (psi)	Vertical Swell Strain Rate (10^{-2}%/ $\log_{10}$ day)	Horizontal Swell Strain Rate (10^{-2}%/ $\log_{10}$ day)
RT1-24	10	Graywacke	Pulaski A	Swell Strain	23	133-161	289-345	0.25	0.5
RT1-29	2	Graywacke	Pulaski A	Swell Strain	23-47	52	127-151	0	0
RT1-37	14	Graywacke	Pulaski A	Swell Strain	63	93-148	249-359	5.0	2.1
RT1-17	14	Graywacke	Pulaski A	Swell Strain	63	150-170	360-403	5.0	0.9
RT1-44	11	Graywacke	Pulaski A	Swell Strain	66	130-165	324-396	5.5	1.7
RT1-44	11	Graywacke	Pulaski A	Swell Strain	66	165-190	396-446	4.0	0.77
RT1-30	20	Graywacke	Pulaski A	Swell Stress	55-220	0-60	55-340	0	3.4
RT1-30	20	Graywacke	Pulaski A	Swell Stress	220-600	60-70	340-740	0	0.25
RT1-33	72	Graywacke	Pulaski A	Swell Stress	20-45	120-160	260-365	0	1.4
RT1-33	22	Graywacke	Pulaski A	Swell Stress	45-70	160-170	365-410	0	0.25
RT1-48	17	Graywacke	Pulaski A	Swell Stress	60	75-110	210-280	0	1.55
RT1-40	17	Graywacke	Pulaski A	Swell Stress	60-75	110-120	280-315	0	0.5
TD-1	6	Graywacke	Pulaski A	Free Swell	0	0	0	5.6	2.2
TD-1	7	Sandstone	Pulaski A	Free Swell	0	0	0	0.18	0.4
TD-1	8	Graywacke	Pulaski A	Free Swell	0	0	0	4.6	5.0
TD-1	9	Graywacke	Pulaski A	Free Swell	0	0	0	12.0	5.3
RT1-52	19	Shale	Pulaski B	Swell Stress	200-460	50-100	300-660	0	3.4
RT1-52	19	Shale	Pulaski B	Swell Stress	460-550	100	600-750	0	0.3
RT1-55	21	Shale	Pulaski B	Swell Stress	75-225	130-140	335-505	0	0
RT1-60	7	Silty shale	Pulaski C	Swell Stress	01	127-150	335-381	0.35	0.4

TABLE 2.5-18
Sheet 3 of 3

Sample No.	Test No.	Rock Type	Rock Unit	Test Type	Vertical Stress $\sigma_v\ (psi)$	Horizontal Stress σ R(psi)	First Stress Invariant $\sigma_v + 2\sigma_h$ (psi)	Vertical Swell Strain Rate (10^{-2}%/ $\log_{10}$ day)	Horizontal Swell Strain Rate (10^{-2}%/ $\log_{10}$ day)
TD-1	10	Graywacke	Pulaski B	Free Swell	0	0	0	13.1	4.4
TD-1	11	Sandstone	Pulaski B	Free Swell	0	0	0	0.2	0.45
TD-1	12	Shale	Pulaski B	Free Swell	0	0	0	-	7.3
TD-1	13	Shale with sandstone lenses	Pulaski B	Free Swell	0	0	0	26.0	7.0
TD-1	14	Shale with sandstone interbeds	Pulaski C	Free Swell	0	0	0	17.0	6.4
TD-1	16	Interbedded shale, sandstone and siltstone	Pulaski C	Free Swell	0	0	0	16.4	6.7
TD-1	17	Siltstone	Pulaski C	Free Swell	0	0	0	15.2	5.4
RT1-63	9	Silty shale	Whetstone Gulf A	Swell Strain	82	120-105	322-452	12.5	1.5
RT1-66	1	Silty shale	Whetstone Gulf A	Swell Strain	131	111-180	353-491	8.0	1.35
RT1-77	3	Silty shale	Whetstone Gulf A	Swell Strain	130	75-110	280-350	0.1	0.95
RT1-77	3	Silty shale	Whetstone Gulf A	Swell Strain	65	125-145	315-355	1.25	0.75

TABLE 2.5-19
Sheet 1 of 1
TRIAXIAL SWELL AND CREEP TEST RESULTS

Rock Type	Sample No.	Cell Pressure (psi)	Strain Rates (10^{-9}/min)[1]					
			Axial			Circumferential		
			Cycle 1	Cycle 2	Cycle 3	Cycle 1	Cycle 2	Cycle 3
Silty shale	RT1-35C	900 406 29	1,300 -89 1,800	740 28 -160	13 -64 32	190 89 740	100 -37 -64	10 -74 5
Black shale	RT2-12B	900 406 29	7,200 -2,800 -2,800	1,000 -56 28	120 -32 250	1,500 0 -278	-42 0 56	40 7 54
Graywacke with silt lenses	RT1-47A	900 406 29	-280 -1,100 -1,800	-860 110 -320	210 110 -100	-5,000 560 -3,500	-250 140 -630	110 83 -390
Graywacke with silt lenses	RT1-28A	900 406 29	560 280 560	-610 -56 -28	160 -12 8	-1,700 -370 -1,000	-280 9 200	21 7 9
Graywacke with silt lenses	RT1-28B	406 203 29	-1,000 -89 0	-190 47 0	250 11 -12	-5,000 190 -190	-330 28 -9	120 7 -5
Silty shale	RT1-35A	1,393	12,000	-1,100	-200	-3,200	-240	-50
Silty shale[2]	RT1-35B	406 203 44	6,900 -740 -370	-260 -56 -110	-60 -32 -24	-1,100 0 280	-250 0 -19	-26 -5 -8

(1) Swell strains are positive; shrinkage strains are negative.
(2) Denotes tested specimen was sealed to prevent water loss or gain. Other tests were executed with water supplied to specimen.

TABLE 2.5-20
Sheet 1 of 1
DIRECT SHEAR STRENGTH OF INTACT BEDDING PLANES AS A FUNCTION
OF NORMAL STRESS AND MULTIPLE DISPLACEMENT*

Sample No.	Peak Shear Strength (psi)		Residual Shear Strength																	
			First Normal Load (psi)						Second Normal Load (psi)						Third Normal Load (psi)					
			1st 1/2″		2nd 1″		3rd 1/2″		1st 1/2″		2nd 1″		3rd 1/2″		1st 1/2″		2nd 1″		3rd 1/2″	
	τ	σ_n	τ	σ_n	τ	σ_n	τ	σ_n	τ	σ_n	τ	σ_n	τ	σ_n	τ	σ_n	τ	σ_n	τ	σ_n
RT1-5	153.3	10.2	36.9	11.3	10.8	10.6	13.4	10.6	30.7	17.0	12.1	16.1	14.4	14.9	28.7	20.7	14.7	21.1	13.5	20.5
RT1-7	388.2	20.7	43.9	22.5	16.4	21.2	16.9	20.5	37.9	31.1	16.0	31.9	13.4	32.2	38.0	39.7	19.8	42.7	14.5	42.9
RT1-13	36.7	30.3	24.2	34.0	17.2	29.9	16.1	32.0	31.7	48.5	19.1	48.3	18.5	48.3	36.1	64.6	22.7	59.8	24.2	64.4
RT1-19	524.5	21.0	116.8	22.4	19.9	21.0	12.6	20.6	76.2	30.5	69.8	30.6	13.9	30.9	60.3	40.6	26.6	40.9	20.5	42.1
RT1-25	361.1	40.1	44.1	43.7	23.9	42.6	23.3	42.6	52.1	62.5	28.0	63.8	31.9	61.4	62.9	83.2	34.0	84.7	41.4	81.6
RT1-34	302.3	30.3	33.4	32.7	22.3	29.8	17.1	31.9	44.8	46.5	31.2	44.6	24.3	47.7	50.9	62.0	39.2	59.5	28.3	63.5
RT1-39	50.2	62.8	49.9	69.2	27.8	65.4	34.0	64.0	54.1	93.3	39.5	95.1	39.3	92.0	61.8	124.2	45.1	127.1	46.9	128.3
RT1-41	80.5	25.3	28.2	26.0	15.5	25.8	17.5	26.0	27.2	38.3	21.0	37.1	16.2	40.7	31.4	51.9	22.9	52.4	21.7	49.4
RT1-42	252.5	50.6	38.6	56.8	20.2	53.0	19.1	53.9	50.6	77.3	31.4	79.7	28.8	80.7	52.9	103.3	41.8	105.9	38.0	107.9
RT1-43	314.1	81.3	56.3	90.3	28.7	85.5	28.1	84.6	67.8	124.9	52.6	127.5	38.6	128.2	75.6	166.8	66.4	169.5	51.5	165.9
RT1-53	154.8	41.3	41.0	45.0	16.4	42.2	14.8	42.4	45.6	62.6	23.6	63.6	23.3	63.9	61.9	83.5	29.9	84.8	32.2	85.2
RT1-54	334.0	81.2	70.4	86.1	36.1	83.2	24.8	86.1	81.4	125.2	46.8	127.3	32.8	122.4	106.9	165.5	64.9	166.1	42.6	162.5
RT1-59	402.3	121.8	90.3	134.8	46.7	127.1	55.5	127.6	117.6	187.3	76.0	190.6	79.5	191.8	143.3	250.0	102.2	254.1	94.8	246.3
RT2-3	76.0	19.9	22.3	21.1	10.5	21.2	9.4	21.2	24.0	31.1	13.0	31.9	12.4	32.0	22.2	41.3	14.9	42.7	14.8	42.6
RT2-8	110.2	10.0	18.7	10.9	7.0	10.5	8.6	10.5	21.0	15.5	9.5	15.9	8.7	15.9	24.1	21.5	11.6	21.2	10.6	21.3
RT2-9	134.5	29.9	37.1	32.5	11.7	31.8	17.7	31.9	49.2	61.9	22.1	63.7	26.7	61.3	71.6	123.6	45.8	127.5	50.6	127.4
RT2-10	250.6	40.1	51.2	45.0	16.3	42.4	19.1	42.6	72.4	83.0	37.4	84.9	38.3	81.9	112.8	164.9	77.0	169.9	66.3	170.4

* Intact 5.2-in core specimens were sheared in a saturated condition under the specified normal load until the peak value was reached. The specimen was then sheared 0.5″ forward, 1″ backward, and 0.5″ forward at each of three normal loads.

τ = Applied shear stress in psi (maximum value for peak test, average for residual tests).

σ_n = Applied normal stress in psi.

TABLE 2.5-21
Sheet 1 of 5
EXPLORATION AT THE UNIT 2 SITE
AFTER GENERAL SITE EXPLORATION (POST-PSAR)

Investigation	Date	Scope of Work	FSAR Section	Reference
General excavation mapping	5/75 through 8/76	Mapping and/or photographing all Category I excavations and anomalous features.	2.5.4.3 Appendix 2H 2.5.1.2.3	
Thrust structures within power block complex	8/76 through 9/76	Detailed mapping and photographs North and south radwaste trenches North wall heater bay Circulating water intake Southern circulating water piping encasement Normal switchgear building Northwest reactor notch Radiometric dating (K/Ar) of clay Palynologic analysis of unconsolidated clay Fracture and fold geometric analysis	 2.5.1.2.3 2.5.1.2.3	Reference 94, Vol. I, Section 3.4 Dames & Moore, 1976 Reference 94, Vol. I, Section 3.4 Reference 94, Vol. I, Sections 3.2 and 3.4
Cooling Tower fault (geologic investigation)	9/76 through 4/78	Detailed mapping Exploratory excavations Pit 1 Trench 3 Trench 4 Trench 5 Existing excavations Circulating water piping trench Cooling tower excavation Drainage ditch Intake shaft Geometric analysis of deformational features All exposures listed above	2.5.1.2.3 Appendix 2H 2.5.1.2.3	 Reference 94, Vol. I, Section 4.0 Reference 94, Vol. I, Section 4.0 Reference 94, Vol. I, Section 4.0 Reference 94, Vol. I, Section 4.0 Reference 94, Vol. I, Section 4.0 Reference 94, Vol. I, Section 4.0 Reference 94, Vol. I, Section 3.3 Reference 94, Vol. I, Section 3.3 Reference 94, Vol. I, Section 3.4

TABLE 2.5-21
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Investigation	Date	Scope of Work	FSAR Section	Reference
Cooling Tower fault (geologic investigation)	9/76 through 4/78	Boring program	2.5.1.2.3	Reference 94, Vol. I, Section 5.0
		Subsurface stratigraphy and structure at two locations along Cooling Tower fault (total of 24 borings) T-3 and T-4 series		
		Standard boring and geophysical logs		Reference 94, Vol. I, Appendix I-A
		Downhole impression packer survey		Reference 94, Vol. I, Appendix I-B
		Mineralization studies	2.5.1.2.6	Reference 94, Vol. I, Section 6.0
		Fluid inclusion analysis of mineralization from surficial exposures and rock cores		Reference 94, Vol. I, Section 6.3 and Appendix I-E
		X-ray diffraction analysis of bedrock samples		Reference 94, Vol. I, Section 6.4 and Appendix I-F
		Uranium series age dating of calcite		Reference 94, Vol. I, Appendix I-G
Cooling Tower fault (geomorphology investigation)	9/76 through 4/78	Stratigraphy and age of unconsolidated sediments	2.5.1.2.2	Reference 94, Vol. II, Section 1.4
		C-14 age dating		Reference 94, Vol. II, Section 1.4 and Appendix II-A
		Palynologic analyses		Reference 94, Vol. I, Appendix I-A and Vol. II, Section 1.4 and Appendix II-B
		Grain size analyses		Reference 94, Vol. II, Section 1.4 and Appendix II-C
		Mineralogic and X-ray diffraction analyses		Reference 94, Vol. II, Section 1.4 and Appendix II-D
		Aerial photograph interpretation	2.5.1.2	Reference 94, Vol. II, Section 1.4.3
Cooling Tower fault (rock mechanics investigation)	1/77 through 4/78	Stress measurements by overcoring	2.5.4.1.4	
		Numerous measurements in borings OC-1 through OC-9		Reference 94, Vol. III, Section 3.0 and Appendix III-C
		Logs of borings OC-1 through OC-9		Reference 94, Vol. III, Appendix III-D
		Stressmeters installed in borings OC-1, OC-2, OC-5, and OC-7		
		Analysis of residual strain relief by undercoring	2.5.4.1.4	Reference 94, Vol. III, Section 4.0 and Appendix III-G
		Measurements from five cores		
		Measurements of unconfined swelling strain	2.5.4.1.4	
		Measurements of seventeen cores under variety of relative humidity conditions		Reference 94, Vol. III, Section 6.0 and Appendix III-K
		Samples from boring TD-1		Reference 94, Vol. III, Appendix III-D
		Measurement of groundwater levels in vicinity of Cooling Tower fault	2.4.13	Reference 94, Vol. III, Section 5.0 and Appendix III-J
		Standpipes installed in borings T-4-7, T-4-9, T-4-10, and T-4-11		
		Water levels monitored 9/30/77 to 1/19/78		

TABLE 2.5-21
Sheet 3 of 5

Investigation	Date	Scope of Work	FSAR Section	Reference
Additional rock mechanics investigation and initial rock monitoring program (cont'd.)	3/77 through 6/82	Inclinometer installation and monitoring I-1 through I-4 around circulating water piping trench and intake shaft Boring logs I-1 through I-4	2.5.4.13 Appendix 2K	
	4/77 through 11/78	Test excavation and monitoring of trench 5 along Cooling Tower fault Inclinometers IT-1 through IT-5 Tape extensometer measurement array Precise leveling survey	2.5.4.13 Appendix 2K	
	1985	Displacement monitoring of 12-line wall	2.5.4.13	
	6/77 through 6/82	Extensometer installation and monitoring MPX-1 and MPX-2 (multipoint wire extensometers) adjacent to intake shaft and reactor containment excavations Boring logs MPX-1 and MPX-2	2.5.4.13 Appendix 2K	
	11/77	Geophysical surveys Borings GP-1, GP-2, and RT-1 Standard logs of borings Geophysical logs (caliper, gamma, density, and 3-D velocity) Uphole compressional wave survey Crosshole compressional wave survey Crosshole shear wave velocity survey	2.5.4.4 Appendix 2K 2.5.4.4	
	11/77 through 2/78	Testing of rock samples from borings RT-1 and RT-2 Boring logs RT-1 and RT-2 Direct shear tests on bedding Ring tests; swelling strain, swelling stress, creep strain Triaxial swell and creep tests Uniaxial compression tests Triaxial strength tests Determination of elastic parameters: Young's modulus and Poisson's ratio parallel and perpendicular to bedding, and shear modulus	2.5.4.2 Appendix 2K	

TABLE 2.5-21
Sheet 4 of 5

Investigation	Date	Scope of Work	FSAR Section	Reference
Additional rock mechanics investigation and initial rock monitoring program (cont'd.)	11/77 through 2/78	Stress measurements by overcoring Stress determinations in Boring OC-100 Log of Boring OC-100	2.5.4.1.4 Appendix 2K	
Radwaste thrust structure (geologic investigation)	4/79 through 10/80	Detailed mapping of various foundation Excavations Circulating water piping excavation Relocated cooling tower excavation North radwaste trench East lake water intake tunnel Analysis of structural elements from mapped areas	2.5.1.2.3 Appendix 2H 2.5.1.2.3	Reference 154, Section 2.1 Reference 154, Section 2.1
	4/79 through 10/80	Boring program Subsurface stratigraphy and structure west Of surficial exposure of radwaste fault (total of 9 borings) Standard boring and geophysical logs, Borings 801 through 810 Downhole television survey	2.5.1.2.3	 Reference 154, Section 2.2 and Appendix A Reference 154, Appendix A
	4/79 through 10/80	Mineralization studies Fluid inclusion analysis of mineralization from exposures of thrust structures and rock cores C-13 and O-18 analysis of calcite Age dating C-14 analysis of calcite unconsolidated sediment from within thrust structures Uranium series dating of calcite Palynologic analysis of sediments Mineralogic analysis of unconsolidated sediments within thrust structures	2.5.1.2.6 2.5.1.2.3 2.5.1.2.6 2.5.1.2.3	Reference 154, Section 2.3 and Appendix B.1 Reference 154, Appendix B.2 Reference 154, Section 2.3 Reference 154, Appendix B.2 Reference 154, Appendix B.3 Reference 154, Appendix C.1 and C.2 Reference 154, Section 2.3, Appendix C.3

TABLE 2.5-21
Sheet 5 of 5

Investigation	Date	Scope of Work	FSAR Section	Reference
Rock mechanics (intake shaft)	11/79 through 1985	Installation of instrumentation in vicinity of intake shaft Inclinometers SI-1 through SI-7 Extensometers EX-1 through EX-4 (multipoint rod type) Piezometers PI-1 through PI-4 (2 transducers each) Boring logs of SI, EX, and PI borings Tape extensometer measurement arrays at four levels in shaft	2.5.4.13 2.4.13 Appendix 2K 2.5.4.2	
Radwaste thrust structure (rock mechanics investigation)	2/80 through 10/80	Stress measurements by overcoring numerous measurements in borings RS-1 through RS-4 Logs of Borings RS-1 through RS-4 Installation of inclinometers in thrust block Inclinometers installed in Borings 803, 805, 806, 810, and RS-2	2.5.4.1.4 2.5.4.2	Reference 154, Section 3.0 and Appendices E.1 and E.3 Reference 154, Appendix E.2
Geophysical survey	7/80	Ground magnetometer survey (across trace of high-angle faults)	2.5.1.2.3	Reference 154, Response to NRC Question Q361.26
Rock mass displacement monitoring program	8/81 through 1985	Installation and monitoring of instruments in and around site excavations Inclinometers Intake shaft, SI-8, SI-9, SI-10 Reactor containment SI-20 through SI-23 Radwaste thrust block 820, 821 Extensometers (multipoint rod type) Intake shaft EX-5 and EX-6 Reactor containment EX-20 and IIEX-1 Piezometers Intake shaft PI-5 Reactor containment PI-20 and PI-21 Boring logs of above boreholes Various displacement monitoring gauges within site engineering structures	2.5.4.13 Appendix 2K	

TABLE 2.5-22
Sheet 1 of 2
PARTICIPANTS OF POST-PSAR INVESTIGATIONS

Investigation	Date	Participant	Contribution
General excavation mapping	5/75 to 1983	Stone & Webster Engineering Corporation	Architect-engineer, all foundation excavation mapping
Thrust structures within the power block complex	8/76 through 9/76	Dames & Moore Dr. F. A. Donath, CGS, Inc. Krueger Enterprises, Inc., Cambridge, MA Dr. I. A. Sirkin - Adelphi University, NY	Principal investigator Assessment of geologic structures K/Ar age dating Palynologic analysis
Cooling tower fault	9/76 through 4/78	Dames & Moore Dr. F. A. Donath Dr. D. Coates, SUNY, Binghamton, NY Dr. S. Alexander, Pennsylvania State University Dr. H. L. Barnes, Pennsylvania State University Dr. A. H. Vassiliou, Rutgers, the State University Dr. Teh-Lung Ku - University of Southern California Dr. L. A. Sirkin Brock University, Toronto, Canada Teledyne Isotopes, Westwood, NJ Krueger Enterprises, Inc. Dr. N. J. Price, Imperial College, London, England	Principal investigator NMPC review panel member NMPC review panel member NMPC review panel member Fluid inclusion analysis, ore microscopy, X-ray diffraction Mineralogic analysis, X-ray diffraction, X-ray, fluorescence Uranium series age dating Palynologic analysis C-14 age dating C-14 age dating C-14 age dating Assessment of evolution of bedrock stresses
Additional rock mechanics investigation and initial monitoring program	11/77 through 2/78	Dames & Moore Birdwell Geophysical Company Franklin Trow Associates, Ltd., Toronto, Canada CGS, Inc., Urbana, IL Woodward Clyde Associates	Principal investigator Downhole geophysical survey Rock testing Rock testing Rock testing
Radwaste thrust structure	4/79 through 10/80	Dames & Moore Dr. F. A. Donath Dr. D. Coates Dr. S. Alexander Dr. S. S. Philbrick Dr. R. H. Johns Mr. W. W. Moore Dr. T. I. Pewe, Tempe, AZ Dr. C. Fairhurst, University of Minnesota, Minneapolis, MN	Principal investigator NMPC review panel member NMPC review panel member NMPC review panel member Dames & Moore review panel member Dames & Moore review panel member Dames & Moore review panel member Assessment of origin of clay deformation Assessment of potential for differential movement on radwaste structure

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Investigation	Date	Participant	Contribution
		Dr. N. J. Price Dr. I. A. Sirkin Dr. J. Terasmae, Brock University Dr. H. Krueger Dr. T. L. Ku Dr. H. L. Barnes Dr. A. H. Vassiliou	Rock mechanics assessment of radwaste structure Assessment of origin of clay deformation and palynologic analyses Palynologic analyses Isotopic and radiometric analyses Uranium series age dating Paragenetic and fluid inclusion analyses Mineralogical analyses
Rock mechanics investigation of intake shaft	11/79 through 1985	Dames & Moore Dr. C. Fairhurst	Principal investigator Reviewer
Rock mass displacement monitoring program	8/81 through 1985	Dames & Moore Dr. C. Fairhurst Dr. E. D. McKay, CGS, Inc. Dr. B. Sellers Mr. J. Dunnicliff, P.E. Dr. F. A. Donath Dr. S. Alexander Dr. D. Coates	Overall responsibility for installation and monitoring program Principal investigator Staff investigator providing coordination between Dames & Moore and principal investigator Instrumentation consultant Instrumentation consultant NMPC review panel member NMPC review panel member NMPC review panel member

TABLE 2.5-23
Sheet 1 of 1

SUMMARY OF BORINGS COMPLETED ON THE NINE MILE POINT SITE

Boring Series	Year(s) Completed	Total No. Borings	Range of Depth (ft)	Purpose	Core Sample (Yes/No)
100	1963	21	10-100	Foundation investigation Unit 1	Yes
200	1968	14	12-150	Foundation investigation proposed facility	Yes
300	1968	15	75-198	Foundation investigation JAF plant	Yes
400	1971-72	31	52-205	Foundation investigation Unit 2	Yes
T-3	1977	10	176-222	Cooling Tower fault subsurface investigation	Yes
T-4	1977	13	90-342	Cooling Tower fault subsurface investigation	Yes
A	1977	2	160-161	Cooling Tower fault subsurface investigation	Yes
OC	1977-78	10	22-116	Stress measurements	Yes
TD	1977	1	106	Free swell testing	Yes
GP	1978	2	360-361	Geophysical testing	Yes
RT	1977-78	2	40-350	Rock testing	Yes
L	1977	8	115-132	Foundation investigation lake water tunnels	Yes
RS	1980	4	94-125	Stress measurements	Yes
800	1980	11	90-300	Radwaste fault investigation and rock monitoring	Yes
MPX	1977-79	2	115-165	Rock monitoring Unit 2	Yes
EX	1979, 1981	7	151-162	Rock monitoring Unit 2	Yes
HEX	1981	1	155	Rock monitoring Unit 2	Yes
SI	1979, 1981	14	151-205	Rock monitoring Unit 2	Yes
PI	1979, 1981	7	88-127	Rock monitoring Unit 2	Yes
I	1977	4	50-150	Rock monitoring Unit 2	No
IT	1977	5	20	Rock monitoring Unit 2	No
P	1979	6	43-95	Piezometer installation Unit 2	No

TABLE 2.5-24
Sheet 1 of 1
SOURCES OF CATEGORY I STRUCTURAL FILL

Vendor	Pit	Location
Meany	Maiden Lane	Scriba
	Airport Road*	Scriba
	Engles Road	Hannibal
Chauvin	Island Road	Palermo

* This pit used for strength testing only.

NOTE: The total quantity of Category I structural fill placed is approximately 4,473 m^3 (5,850 yd^3).

TABLE 2.5-25
Sheet 1 of 1
TRIAXIAL SHEAR TEST RESULTS

Source: Meany Airport Road Pit

Confining Pressure: σ_3 = 69kn/m^2 (10 psi)

Axial Deviator			Induced Principal Pore			Normalized Data		Stress Path			
	Stress							τ45		Σ457	
Strain (%)	psi	kn/m^2	Pore Pressure (kn/m^2)	Stress Ratio	Pressure Parameter	Deviator Stress	Pore Pressure	kn/m^2	psi	psi	Kn/m^2
0.0	0.0	0.0	0.0	1.000	0.0	0.0	0.0	0.0	0.0	10.0	69.0
0.04	4.5	31.2	0.0	1.452	0.0	0.452	0.0	15.6	2.2	12.2	84.6
0.10	10.1	70.1	0.0	2.016	0.0	1.016	0.0	35.0	5.0	15.0	104.0
0.18	18.6	128.9	0.0	2.869	0.0	1.869	0.0	64.5	9.3	19.3	133.5
0.29	28.7	198.0	0.0	3.870	0.0	2.870	0.0	99.0	14.3	24.3	168.0
0.44	38.1	263.2	0.0	4.815	0.0	3.815	0.0	131.6	19.0	29.0	200.6
0.78	45.3	313.2	0.0	5.539	0.0	4.539	0.0	156.6	22.7	32.7	225.6
1.17	43.8	302.6	0.0	5.385	0.0	4.385	0.0	151.3	21.9	31.9	220.2
1.62	39.7	274.6	0.0	4.979	0.0	3.979	0.0	137.2	19.8	29.8	206.2
2.35	38.1	263.2	0.0	4.814	0.0	3.814	0.0	131.6	19.0	29.0	200.6
3.17	36.1	249.4	0.0	4.614	0.0	3.614	0.0	124.7	18.0	28.0	193.7
4.20	35.1	242.4	0.0	4.513	0.0	3.513	0.0	121.2	17.5	27.5	190.2
5.56	36.2	250.0	0.0	4.624	0.0	3.624	0.0	125.0	18.1	28.1	194.0
8.44	34.7	239.4	0.0	4.470	0.0	3.470	0.0	119.7	17.3	27.3	188.7
11.77	35.0	241.5	0.0	4.501	0.0	3.501	0.0	120.8	(17.5)	(27.5)	189.8
14.65	35.5	245.2	0.0	4.553	0.0	3.553	0.0	122.6	(17.7)	(27.7)	191.6

TABLE 2.5-26
Sheet 1 of 1
TRIAXIAL SHEAR TEST RESULTS

Source: Meany Airport Road Pit

Confining Pressure: σ_3 = 138 kn/m^2 (20 psi)

Axial Deviator			Induced Principal Pore			Normalized Data		Stress Path			
Strain (%)	Stress		Pore Pressure (kn/m^2)	Stress Ratio	Pressure Parameter	Deviator Stress	Pore Pressure	τ45		Σ45	
	psi	kn/m^2						kn/m^2	Psi	psi	kn/m^2
0.0	0.0	0.0	0.0	1.000	0.0	0.0	0.0	0.0	0.0	20.0	138.0
0.05	6.6	45.7	0.0	1.331	0.0	0.331	0.0	22.8	3.3	23.3	160.8
0.10	12.6	87.2	0.0	1.632	0.0	1.632	0.0	43.6	6.3	26.3	181.6
0.16	18.8	129.8	0.0	1.940	0.0	0.940	0.0	64.9	9.4	29.5	202.9
0.31	33.9	233.9	0.0	2.695	0.0	1.695	0.0	117.0	16.9	36.9	255.0
0.56	49.3	340.4	0.0	3.466	0.0	2.466	0.0	170.2	24.6	44.6	308.2
0.56	49.3	340.4	0.0	3.466	0.0	2.466	0.0	170.2	24.6	44.6	308.2
0.80	58.0	400.7	0.0	3.904	0.0	2.904	0.0	200.4	29.0	49.0	338.4
1.10	61.8	426.6	0.0	4.091	0.0	3.091	0.0	213.3	30.9	50.9	351.3
1.59	61.6	425.0	0.0	4.080	0.0	3.080	0.0	212.5	30.8	50.8	350.5
2.13	61.3	423.1	0.0	4.065	0.0	3.065	0.0	211.5	30.6	50.6	349.5
3.19	62.0	428.1	0.0	4.102	0.0	3.102	0.0	214.1	31.0	51.0	352.1
4.34	61.7	425.7	0.0	4.085	0.0	3.085	0.0	212.9	30.8	50.8	350.9
5.22	61.7	425.7	0.0	4.087	0.0	3.087	0.0	212.9	30.8	50.8	350.9
6.98	62.3	430.5	0.0	4.119	0.0	3.119	0.0	215.2	31.1	51.1	353.2
9.40	63.6	439.4	0.0	4.184	0.0	3.184	0.0	219.7	31.8	51.8	357.7
11.82	65.0	448.5	0.0	4.250	0.0	3.250	0.0	224.3	32.5	52.5	362.3
14.24	65.7	453.8	0.0	4.289	0.0	3.289	0.0	226.9	32.8	52.8	364.9

TABLE 2.5-27
Sheet 1 of 1
TRIAXIAL SHEAR TEST RESULTS

Source: Meany Airport Road Pit

Confining Pressure: σ_3 = 276.0 kn/m^2 (40 psi)

Axial Deviator			Induced Principal Pore			Normalized Data		Stress Path			
Strain (%)	Stress		Pore Pressure (kn/m^2)	Stress Ratio	Pressure Parameter	Deviator Stress	Pore Pressure	τ45		Σ45	
	psi	kn/m^2						kn/m^2	psi	psi	kn/m^2
0.0	0.0	0	0.0	1.000	0.0	0.0	0.0		0.0	40.0	276.0
0.04	8.6	59.8	0.0	1.216	0.0	0.216	0.0	29.9	4.3	44.3	305.9
0.10	19.0	131.1	0.0	1.475	0.0	0.475	0.0	65.6	9.5	49.5	341.6
0.21	31.4	216.9	0.0	1.786	0.0	0.786	0.0	108.5	15.7	55.7	384.5
0.36	47.2	325.9	0.0	2.181	0.0	1.181	0.0	162.9	23.6	63.6	438.9
0.63	64.9	448.4	0.0	2.624	0.0	1.624	0.0	224.2	32.4	72.4	500.2
1.03	79.9	551.8	0.0	2.999	0.0	1.999	0.0	275.9	39.9	79.9	551.9
1.54	89.3	616.3	0.0	3.233	0.0	2.233	0.0	308.2	44.6	84.6	584.2
2.36	97.8	675.1	0.0	3.446	0.0	2.446	0.0	337.6	48.9	88.9	613.6
3.30	104.3	719.9	0.0	3.608	0.0	2.608	0.0	359.9	52.1	92.1	635.9
4.49	110.5	762.8	0.0	3.764	0.0	2.764	0.0	381.4	55.2	95.2	657.4
6.01	116.8	806.1	0.0	3.921	0.0	2.921	0.0	403.1	58.4	98.4	679.1
7.78	121.5	838.9	0.0	4.040	0.0	3.040	0.0	419.5	60.7	100.7	695.5
10.81	128.8	889.3	0.0	4.222	0.0	3.222	0.0	444.7	64.4	104.4	720.7
13.39	133.2	919.6	0.0	4.332	0.0	3.332	0.0	459.8	66.6	106.6	735.8
15.06	134.3	926.9	0.0	4.358	0.0	3.358	0.0	463.4	67.1	107.1	739.4

TABLE 2.5-28
Sheet 1 of 1
INDEX AND MECHANICAL PROPERTIES OF
CATEGORY I STRUCTURAL FILL

Property	Value
Specific gravity	2.66
Limiting densities	
Maximum	2.09 g/cc (130.6 pcf)
Minimum	1.72 g/cc (107.6 pcf)
In-place density	2.14 g/cc (134.1 pcf)
Maximum dry density	2.17 g/cc (135.5 pcf)
Optimum moisture content	7.6%
Angle of internal friction	40°

TABLE 2.5-28A
Sheet 1 of 1
QUALITY CONTROL PROGRAM FOR CATEGORY I
STRUCTURAL FILL AND GRANULAR FILL

Test or Inspection	Test Designation	Minimum Frequency
Gradation test	ASTM C136-81	1/1,000 cu yd for structural fill 1/4,000 cu yd for granular fill
Moisture density test	ASTM D1557-78	1/1,000 cu yd for structural fill 1/2,000 cu yd for granular fill
Backfill conditions		Each lift
Backfill and compaction procedure		Each lift
Lift thickness		Each lift
Compaction equipment		Prior to each placement
Passes of compaction equipment		Each lift
In-place density test	ASTM D1556-74 ASTM D2922-81	For open areas, 1/500 cu yd[1] For confined areas, 1/100 cu yd[2]
Relative density test	ASTM D2049-69	Prior to placement, Category I structural fill only

(1) Perform at least one test every lift if the lift is less than 500 cu yd.
(2) Perform at least one test every other lift if the lift is less than 100 cu yd.

TABLE 2.5-29
Sheet 1 of 7
PIEZOMETER READINGS, P-SERIES

Date	Time	Water Level Elevation (ft)					
		P-1	P-2	P-3	P-4	P-5	P-6
8-14-78	3:30 PM	220.5	211.2	208.3	223.6	-	-
8-15-78	3:30 PM	220.5	211.2	208.2	223.6	-	-
8-16-78	3:30 PM	220.5	211.3	208.2	224.0	-	-
8-17-78	8:30 AM	220.6	211.2	208.3	224.0	-	-
8-17-78	3:45 PM	220.5	211.2	208.3	224.0	-	-
8-18-78	4:15 PM	220.6	211.2	208.3	224.0	-	-
8-21-78	4:00 PM	220.7	211.3	208.6	224.1	-	-
8-22-78	4:00 PM	220.7	211.3	208.8	224.1	-	-
8-23-78	3:45 PM	220.7	211.3	208.9	224.0	-	-
8-24-78	8:45 AM	220.8	211.4	209.0	224.0	-	-
8-24-78	4:00 PM	220.7	211.3	209.0	223.9	-	-
8-25-78	3:30 PM	220.7	211.3	209.0	223.9	-	-
8-28-78	3:00 PM	220.7	211.1	209.1	223.9	-	-
8-29-78	3:45 PM	220.6	211.0	209.1	223.9	-	-
8-30-78	8:40 AM	220.7	211.1	209.0	224.0	-	-
8-30-78	3:45 PM	220.7	211.2	209.1	223.9	-	-
8-31-78	3:40 PM	220.7	211.2	209.1	223.7	-	-
9-1-78	3:45 PM	220.8	211.2	209.1	223.5	-	-
9-5-78	3:15 PM	220.7	211.1	209.5	223.4	-	-
9-6-78	4:00 PM	220.7	210.9	209.6	223.5	-	-
9-7-78	8:30 AM	220.7	210.8	209.7	223.4	-	-
9-7-78	3:15 PM	220.8	210.9	209.7	223.3	-	-
9-8-78	3:30 PM	220.7	211.0	210.0	223.5	-	-
9-11-78	3:30 PM	220.7	211.0	210.1	223.5	-	-
9-12-78	4:45 PM	220.7	210.9	210.0	223.6	-	-
9-13-78	5:15 PM	220.7	210.9	210.3	224.0	-	-
9-14-78	9:00 AM	220.6	210.9	210.4	224.0	-	-
9-14-78	3:45 PM	220.7	211.0	210.6	224.1	-	-
9-15-78	4:15 PM	220.7	211.0	210.8	224.0	-	-
9-18-78	3:45 PM	220.7	211.1	210.6	224.0	-	-
9-19-78	3:55 PM	220.7	211.1	210.6	224.2	-	-
9-20-78	3:00 PM	220.7	211.1	210.9	224.6	-	-
9-21-78	8:30 AM	220.7	211.1	211.0	224.2	-	-
9-21-78	3:45 PM	220.7	211.1	210.9	224.7	-	-
9-22-78	4:45 PM	220.6	211.0	210.6	224.2	-	-
9-25-78	4:10 PM	220.6	211.1	210.7	224.0	-	-
9-26-78	3:30 PM	220.6	211.1	210.7	223.9	-	-
9-27-78	4:15 PM	220.6	211.1	210.9	224.1	-	-
9-28-78	8:30 AM	220.5	210.9	210.8	223.9	-	-
9-28-78	3:45 PM	220.5	211.0	210.8	223.8	-	-
9-29-78	3:45 PM	220.6	211.1	210.8	223.7	-	-

TABLE 2.5-29
Sheet 2 of 7

Date	Time	Water Level Elevation (ft)					
		P-1	P-2	P-3	P-4	P-5	P-6
10-2-78	3:15 PM	220.5	211.1	211.0	223.7	-	-
10-3-78	3:50 PM	220.5	211.1	211.0	223.8	-	-
10-5-78	8:30 AM	220.5	211.0	211.0	223.8	-	-
10-5-78	3:45 PM	220.5	211.1	211.1	224.0	-	-
10-6-78	3:15 PM	220.5	211.0	212.1	224.3	-	-
10-9-78	3:20 PM	220.5	211.0	210.9	224.1	-	-
10-10-78	4:00 PM	220.4	211.0	211.0	224.2	-	-
10-11-78	3:45 PM	220.4	211.1	210.9	224.2	-	-
10-12-78	3:45 PM	220.4	211.0	211.0	224.3	-	-
10-13-78	3:00 PM	220.4	211.1	210.9	224.1	-	-
10-16-78	4:00 PM	220.3	211.1	210.8	224.7	-	-
10-17-78	3:50 PM	220.3	211.1	210.8	224.5	-	-
10-19-78	8:20 AM	220.2	210.9	211.1	224.0	-	-
10-19-78	4:00 PM	220.2	210.9	211.0	223.8	-	-
10-20-78	3:45 PM	220.2	210.8	210.9	223.7	-	-
10-23-78	3:55 PM	220.2	210.8	210.8	223.8	-	-
10-24-78	4:00 PM	220.2	210.8	210.7	223.8	-	-
10-25-78	3:30 PM	220.1	210.6	210.8	223.9	-	-
10-26-78	8:35 AM	220.2	210.0	210.8	223.8	-	-
10-26-78	3:45 PM	220.2	209.9	210.8	223.8	-	-
10-27-78	4:00 PM	220.1	210.1	210.8	224.0	-	-
10-30-78	4:00 PM	221.1	210.5	210.6	223.9	-	-
10-31-78	3:35 PM	220.0	210.3	210.7	224.0	-	-
11-1-78	3:40 PM	220.1	210.4	210.7	223.9	-	-
11-2-78	8:20 AM	220.0	210.3	210.7	223.9	-	-
11-2-78	3:30 PM	220.0	210.2	210.8	223.9	-	-
11-3-78	3:45 PM	220.0	210.3	210.7	223.8	-	-
11-6-78	3:30 PM	220.0	210.0	210.7	223.8	-	-
11-7-78	4:20 PM	220.0	210.1	210.7	223.8	-	-
11-8-78	3:15 PM	220.0	210.1	210.7	223.9	-	-
11-9-78	9:10 AM	219.9	210.0	210.7	223.9	-	-
11-10-78	3:40 PM	219.9	210.0	210.7	223.9	-	-
11-11-78	3:10 PM	219.9	210.1	210.7	223.7	-	-
11-13-78	3:45 PM	219.8	210.1	210.8	223.8	-	-
11-14-78	3:50 PM	219.9	209.9	210.8	223.8	-	-
11-15-78	3:55 PM	219.8	209.9	210.8	223.7	-	-
11-16-78	8:20 AM	219.9	210.1	210.7	223.7	-	-
11-16-78	4:30 PM	219.8	210.1	210.7	223.7	-	-
11-17-78	3:45 PM	219.7	210.0	210.7	224.0	-	-
11-20-78	3:45 PM	219.8	210.1	210.5	224.0	-	-
11-21-78	3:45 PM	219.8	210.0	210.5	224.0	-	-
11-22-78	3:50 PM	219.7	209.8	210.5	224.0	-	-
11-24-78	3:00 PM	219.7	208.6	210.8	224.4	-	-
11-27-78	3:40 PM	219.6	208.9	210.5	224.4	-	-
11-28-78	3:45 PM	219.7	208.6	210.4	224.2	-	-
11-29-78	3:40 PM	219.6	207.9	210.5	224.4	-	-
11-30-78	4:00 PM	219.6	208.6	210.5	224.3	-	-
12-1-78	3:30 PM	219.7	208.8	210.5	224.5	-	-

TABLE 2.5-29
Sheet 3 of 7

Date	Time	Water Level Elevation (ft)					
		P-1	P-2	P-3	P-4	P-5	P-6
12-4-78	3:30 PM	219.7	208.3	210.8	224.7	-	-
12-5-78	3:40 PM	219.6	208.5	210.7	224.6	-	-
12-6-78	3:55 PM	219.7	208.7	210.5	224.6	-	-
12-7-78	8:35 AM	219.7	208.9	210.5	224.5	-	-
12-7-78	3:40 PM	219.6	209.1	210.6	224.6	-	-
12-8-78	11:15 AM	219.7	208.9	210.7	224.6	-	-
12-11-78	3:25 PM	219.5	208.9	210.5	224.7	-	-
12-12-78	3:30 PM	219.5	208.9	210.7	224.7	-	-
12-13-78	3:35 PM	219.5	208.6	210.8	224.7	-	-
12-14-78	8:35 AM	219.5	208.5	210.7	224.6	-	-
12-14-78	3:30 PM	219.5	208.6	210.7	224.7	-	-
12-15-78	3:15 PM	219.5	208.6	210.9	224.7	-	-
12-18-78	4:05 PM	219.6	208.6	210.8	224.9	-	-
12-19-78	4:00 PM	219.6	208.6	210.8	224.9	-	-
12-20-78	3:15 PM	219.5	208.7	210.9	225.1	-	-
12-21-78	8:20 AM	219.6	208.4	210.9	225.0	-	-
12-21-78	2:45 PM	219.6	208.3	210.9	224.8	-	-
12-22-78	11:00 AM	219.6	208.7	210.7	224.9	-	-
12-27-78	4:00 PM	219.5	-	-	-	-	-
12-28-78	11:00 AM	219.5	208.8	210.4	224.7	-	-
12-29-78	2:45 PM	219.6	209.1	210.5	224.6	-	-
1-2-79	4:00 PM	219.7	208.8	211.6	225.8	-	-
1-3-79	3:55 PM	219.6	208.8	211.1	225.6	-	-
1-11-79	3:45 PM	219.6	211.4	212.4	225.1	-	-
1-19-79	9:40 AM	219.6	211.3	212.5	225.1	-	-
1-25-79	8:30 AM	219.7	211.6	213.0	225.9	-	-
2-1-79	9:00 AM	219.8	211.0	212.3	226.2	-	-
2-8-79	10:00 AM	219.7	211.0	212.5	225.8	-	-
2-15-79	8:45 AM	219.7	211.4	212.9	225.3	-	-
2-22-79	8:30 AM	219.6	211.1	213.0	225.0	-	-
3-2-79	9:30 AM	219.7	211.3	212.5	226.0	-	-
3-9-79	8:30 AM	219.9	210.4	212.1	226.6	-	-
3-16-79	11:00 AM	220.1	210.6	210.8	226.5	-	-
3-30-79	11:00 AM	220.0	211.0	210.6	226.3	-	-
4-13-79	8:45 AM	220.4	211.5	210.3	226.2	-	-
4-27-79	9:30 AM	220.4	211.4	210.2	226.5	-	-
5-11-79	4:00 PM	220.5	211.6	209.9	226.9	-	-
5-18-79	9:00 PM	220.6	211.6	209.5	226.9	-	-
5-25-79	9:00 PM	220.7	211.6	210.1	227.0	-	-
6-1-79	9:30 PM	220.8	211.7	210.9	227.7	-	-
6-18-79	8:45 AM	220.6	211.5	210.3	227.1	-	-
6-25-79	2:30 PM	220.6	211.5	209.9	226.4	-	-
7-2-79	4:15 PM	220.8	211.5	210.5	226.2	-	-
7-9-79	4:15 PM	220.8	211.6	210.6	226.2	-	-
7-10-79	10:15 AM	220.8	211.5	210.6	225.7	-	-
8-6-79	8:45 AM	221.1	211.7	210.7	226.3	-	-
8-14-79	3:45 PM	-	-	-	-	210.22	210.10
8-15-79	9:30 AM	-	-	-	-	209.75	209.75

TABLE 2.5-29
Sheet 4 of 7

Date	Time	Water Level Elevation (ft)					
		P-1	P-2	P-3	P-4	P-5	P-6
8-15-79	4:00 PM	-	-	-	-	209.62	209.65
8-16-79	4:15 PM	-	-	-	-	209.56	209.58
8-17-79	4:30 PM	-	-	-	-	209.37	209.35
8-20-79	4:00 PM	221.1	211.7	211.0	226.0	209.67	209.68
8-21-79	2:45 PM	-	-	-	-	209.67	209.67
8-22-79	3:20 PM	-	-	-	-	209.55	209.53
8-23-79	3:45 PM	-	-	-	-	209.85	209.89
8-24-79	3:30 PM	221.0	211.6	211.1	226.1	210.72	210.72
8-27-79	5:10 PM	-	-	-	-	209.72	214.25
8-28-79	4:15 PM	-	-	-	-	211.87	211.86
8-29-79	5:00 PM	-	-	-	-	213.79	213.81
8-30-79	3:15 PM	-	-	-	-	211.88	211.88
8-31-79	3:30 PM	220.8	211.3	211.2	226.6	210.28	210.29
9-4-79	3:15 PM	221.0	211.7	211.4	226.5	212.64	212.64
9-5-79	4:45 PM	-	-	-	-	209.63	209.64
9-6-79	4:00 PM	-	-	-	-	214.18	214.18
9-7-79	4:30 PM	-	-	-	-	211.89	211.90
9-10-79	4:30 PM	-	-	-	-	210.94	210.93
9-11-79	4:45 PM	-	-	-	-	209.77	209.77
10-29-79	3:30 PM	220.4	211.0	211.2	227.3	-	-
11-13-79	4:00 PM	220.5	211.3	211.3	227.3	-	-
12-4-79	2:15 PM	220.0	210.0	210.7	227.0	-	-
12-11-79	8:15 AM	220.3	211.0	210.8	226.2	-	-
12-24-79	11:00 AM	220.0	210.9	211.3	226.3	-	-
1-11-80	10:00 AM	219.5	210.5	211.3	226.4	-	-
1-13-80	1:30 PM	219.8	210.6	211.2	226.5	-	-
2-20-80	2:15 PM	218.4	210.4	211.1	226.1	-	-
2-28-80	2:15 PM	218.5	210.1	211.3	-	-	-
3-12-80	1:00 PM	217.2	210.0	211.0	-	-	-
3-20-80	1:45 PM	217.6	210.2	211.4	-	-	-
3-27-80	1:30 PM	218.1	210.2	211.0	-	-	-
4-3-80	3:30 PM	217.9	210.1	211.1	-	-	-
4-17-80	3:00 PM	218.7	210.0	211.4	-	-	-
4-23-80	2:15 PM	218.6	209.4	211.6	-	-	-
5-1-80	2:45 PM	218.7	209.7	211.9	-	-	-
5-8-80	1:00 PM	218.4	209.4	211.8	-	-	-
5-16-80	2:00 PM	219.0	211.5	211.7	-	-	-
5-22-80	2:30 PM	219.0	211.4	212.0	-	-	-
6-5-80	2:00 PM	218.9	211.4	210.8	-	-	-
6-12-80	3:15 PM	219.0	211.4	-	-	-	-
6-18-80	10:15 AM	219.0	211.4	-	-	-	-
7-25-80	1:15 PM	219.3	211.5	-	228.8	-	-
8-1-80	1:30 PM	219.3	211.5	-	229.1	-	-

TABLE 2.5-29
Sheet 5 of 7

Date	Time	Water Level Elevation (ft)					
		P-1	P-2	P-3	P-4	P-5	P-6
9-4-80	1:30 PM	219.8	211.7	-	228.3	-	-
10-3-80	-	219.0	-	-	228.3	-	-
10-21-80	11:30 AM	219.1	211.1	210.8	228.8	-	-
10-31-80	1:00 PM	219.1	211.2	212.8	228.5	-	-
11-7-80	1:00 PM	219.5	211.5	212.8	228.9	-	-
11-17-80	1:00 PM	218.3	211.1	212.5	228.7	-	-
11-21-80	2:30 PM	218.9	211.0	212.4	229.0	-	-
12-10-80	3:00 PM	219.2	211.3	212.5	229.9	-	-
12-17-80	1:00 PM	218.9	211.2	212.3	230.0	-	-
12-23-80	1:00 PM	219.0	211.2	212.3	230.0	-	-
12-30-80	1:00 PM	219.0	211.2	212.4	229.1	-	-
1-7-81	10:00 AM	218.9	211.2	212.3	230.7	-	-
1-15-81	1:00 PM	218.8	211.1	212.2	230.3	-	-
1-26-81	8:30 AM	218.8	211.1	212.1	230.5	-	-
2-4-81	10:00 AM	218.7	211.3	211.9	-	-	-
2-11-81	8:30 AM	220.0	212.1	212.0	-	-	-
2-18-81	10:30 AM	219.5	211.8	212.1	230.9	-	-
2-24-81	1:30 PM	220.0	212.0	212.3	230.9	-	-
3-4-81	1:00 PM	219.0	211.0	211.7	-	-	-
3-11-81	9:00 AM	218.1	210.6	210.8	230.7	-	-
3-18-81	11:30 AM	217.9	210.6	209.9	-	-	-
3-30-81	11:15 AM	218.8	211.1	210.5	230.3	-	-
4-6-81	3:00 PM	218.8	211.1	210.1	229.7	-	-
4-13-81	3:00 PM	218.8	211.2	210.2	229.5	-	-
4-20-81	1:00 PM	219.3	211.4	210.5	229.9	-	-
4-28-81	9:00 AM	218.4	211.4	210.6	230.0	-	-
5-5-81	11:30 AM	218.5	210.1	210.9	229.7	-	-
5-13-81	1:00 PM	218.5	210.0	210.6	229.5	-	-
5-26-81	2:00 PM	219.0	211.0	211.3	230.0	-	-
6-1-81	2:00 PM	218.8	210.6	210.8	230.2	-	-
6-10-81	11:30 AM	219.3	211.1	211.3	230.6		
6-16-81	3:30 PM	219.6	211.3	211.3	229.7		
6-24-81	3:00 PM	219.9	211.4	212.3	230.6		
7-1-81	2:00 PM	218.6	211.7	212.2	230.2		
7-8-81	9:30 AM	218.9	211.4	212.0	234.1		
7-14-81	11:00 AM	219.8	211.4	211.0	232.8		
7-22-81	10:30 AM	221.0	-	214.0	231.0		
7-27-81	10:00 AM	219.1	211.3	212.5	230.5		
8-4-81	1:00 PM	219.3	210.9	213.0	231.0		
8-13-81	4:15 PM	220.3	212.5	215.9	231.5		
8-20-81	10:00 AM	219.6	211.2	213.8	231.3		
8-28-81	2:45 PM	219.6	211.3	212.6	231.3		
9-4-81	9:00 AM	220.3	212.1	213.5	231.5		
9-16-81	3:00 PM	219.9	211.2	213.2	231.8		
9-29-81	2:30 PM	220.0	211.5	213.5	231.8		
10-7-81	11:00 AM	219.7	211.4	207.4	232.1		
10-21-81	9:30 AM	219.4	211.2	212.9	231.5		
10-28-81	3:00 PM	220.0	211.2	213.5	231.5		

TABLE 2.5-29

Date	Time	Water Level Elevation (ft)					
		P-1	P-2	P-3	P-4	P-5	P-6
11-4-81	8:30 AM	219.8	211.4	213.7	231.8		
11-12-81	9:30 AM	219.5	211.2	213.2	231.4		
11-18-81	10:00 AM	219.4	211.2	213.4	231.7		
11-24-81	2:30 PM	219.8	210.9	213.4	231.3		
12-3-81	1:30 PM	219.4	211.5	213.4	231.4		
12-9-81	11:00 AM	219.3	211.7	213.3	-		
12-18-81	10:00 AM	219.4	211.7	213.3	-		
12-23-81	11:00 AM	219.3	-	213.3	-		
1-7-82	11:00 AM	220.0	212.3	214.7	231.1		
1-15-82	9:30 AM	219.2	211.6	213.9	231.3		
1-21-82	9:30 AM	219.1	211.7	213.4	230.7		
2-3-82	1:00 PM	219.4	-	214.0	231.0		
2-15-82	10:00 AM	220.3	-	213.9	231.1		
2-23-82	9:10 AM	220.1	-	213.9	231.2		
3-8-82	1:30 PM	219.2	-	213.8	230.7		
3-16-82	11:30 AM	220.2	-	214.8	231.3		
3-30-82	2:30 PM	219.3	212.0	213.9	230.8		
7-2-82	3:30 PM	220.1	-	213.7	231.4		
7-12-82	12:45 PM	219.9	-	213.3	231.5		
7-20-82	8:45 AM	219.8	212.9	206.7	230.2		
7-27-82	1:30 PM	219.7	212.9	212.3	229.0		
8-3-82	1:00 PM	219.4	212.8	212.4	228.2		
8-11-82	9:00 AM	219.8	-	212.3	227.7		
8-18-82	2:00 PM	220.0	212.7	212.3	227.2		
8-25-82	2:30 PM	220.2	-	214.1	227.3		
8-31-82	1:00 PM	220.0	212.9	213.0	227.2		
9-9-82	9:45 AM	219.9	212.7	212.3	227.1		
9-15-82	10:00 AM	219.9	213.9	212.3	227.2		
9-22-82	1:30 PM	219.8	212.9	212.2	226.2		
9-30-82	9:15 AM	219.9	213.0	213.9	226.5		
10-6-82	2:30 PM	219.7	212.7	212.8	226.7		
11-4-82	11:00 AM	220.0	214.0	214.0	228.2		
11-12-82	3:30 PM	219.9	-	214.2	228.0		
11-18-82	1:30 PM	220.1	215.3	213.8	227.2		
11-30-82	9:30 AM	219.8	212.9	214.0	228.5		
12-22-82	2:30 PM	219.6	212.6	213.7	228.9		
1-3-83	12:30 PM	220.0	213.1	214.2	228.4		
1-17-83	10:00 AM	220.9	213.7	214.3	228.9		
1-24-83	1:00 PM	221.1	214.5	214.3	228.6		
1-31-83	9:15 AM	222.2	214.2	214.6	228.6		
2-16-83	9:30 AM	221.7	-	215.1	230.0		
3-17-83	1:30 PM	226.0	218.2	217.0	230.9		
3-25-83	2:15 PM	226.2	218.4	217.1	230.3		
4-1-83	1:30 PM	222.1	215.1	216.6	231.0		
4-13-83	9:30 AM	225.9	218.2	216.84	230.8		
4-21-83	1:15 PM	226.3	220.6	217.4	230.5		
5-10-83	1:00 PM	225.20	217.1	216.0	230.9		
5-18-83	1:00 PM	226.2	218.2	216.1	231.2		

TABLE 2.5-29
Sheet 7 of 7

Date	Time	Water Level Elevation (ft)					
		P-1	P-2	P-3	P-4	P-5	P-6
5-24-83	1:00 PM	226.9	219.1	216.8	231.8		
6-24-83	9:45 AM	226.9	219.0	217.2	231.5		
7-1-83	1:15 PM	221.1	-	215.2	231.5		
7-22-83	9:00 AM	220.9	213.2	213.3	230.6		
8-15-83	3:00 PM	221.5	213.8	213.4	228.9		
9-12-83	3:00 PM	221.5	213.9	213.1	231.5		

NOTE: Dash (-) = No reading.

TABLE 2.5-30
Sheet 1 of 1
PIEZOMETER READINGS, PI-SERIES

Date	Shaft Water Level Elev. (ft)	Water Level Elevation (ft)							
		PI-1		PI-2		PI-3		PI-4	
		A Sensor	B Sensor	A Sensor	B Sensor	A Sensor	B Sensor	A Sensor	B Sensor
11/12/79	NR	184.1	190.2	188.6	196.9	NR	NR	204.3	202.8
12/09/79	186.0	184.5	189.7	184.9	197.3	184.9	184.2	197.9	--
12/12/79	177.0	178.3	185.6	182.1	197.8	180.8	185.6	207.8	198.6
12/17/79	143.5	151.3	184.2	164.6	196.9	155.8	184.2	189.1	--
12/19/79	142.0	150.4	184.2	162.2	196.4	155.4	184.7	189.1	--
12/21/79	140.0	149.0	184.2	164.2	196.9	154.5	184.7	187.7	--
12/28/79	140.0	149.5	183.8	164.2	195.5	156.5	184.7	188.2	--
01/03/80	140.0	148.5	183.3	163.7	195.0	156.3	184.2	185.4	--
01/17/80	136.0	138.8	--	--	195.5	141.5	184.7	185.4	--
01/29/80	135.0	--	--	--	206.1	136.9	--	188.2	--
02/11/80	130.0	--	--	--	209.6	139.6	--	NR	NR
03/17/80	130.0	--	--	--	198.7	137.8	--	184.5	--
04/14/80	130.0	--	--	--	198.3	136.0	--	184.0	--
05/21/80	130.0	--	--	--	198.3	135.5	--	182.6	--
06/18/80	130.0	--	--	--	201.0	136.0	--	183.1	--
07/21/80	130.0	--	--	--	202.3	136.0	--	NR	NR
09/04/80	130.0	--	--	--	202.0	135.5	--	183.1	--
10/17/80	123.0	--	--	--	202.9	134.2	--	182.6	--
12/01/80	123.0	--	--	--	202.0	132.3	--	181.3	195.8

KEY: -- = Water level below sensor level
NR = No reading

NOTE: Sensor elevations are:

PI-1A	131.0	PI-1B	181.0
PI-2A	160.0	PI-2B	189.5
PI-3A	130.0	PI-3B	181.0
PI-4A	160.5	PI-4B	194.0

TABLE 2.5-31
Sheet 1 of 1
SUMMARY OF CYCLIC TRIAXIAL TESTS
BORROW SOURCE: CHAUVIN PIT

Initial			After Consolidation			Effective Consolidation Pressure		B at End of Consolidation	Cyclic Deviator Stress		Number of Cycles to:			
Dry Unit Weight		Relative Density (%)	Dry Unit Weight		Relative Density (%)						Double Amplitude Strain			Failure in Extension
kN/m^3	pcf		kN/m^3	pcf		kNm^2	psi		kN/m^{-2}	psi	2.5%	5%	10%	
17.9	(113.7)	68	18.0	(113.8)	69	49.0	(7.11)	0.84	34.3	(4.98)	46	48	50	-
18.0	(113.8)	69	18.0	(113.9)	70	49.0	(7.11)	0.97	36.3	(5.26)	27	28	29	-
18.0	(114.3)	73	18.0	(114.4)	73	49.0	(7.11)	0.97	47.1	(6.83)	10	11	-	12
18.0	(114.3)	73	18.0	(114.5)	74	49.0	(7.11)	0.97	43.2	(6.26)	10	11	12	14
18.0	(113.9)	70	18.0	(114.4)	73	147.1	(21.33)	0.96	85.3	(12.38)	18	19	20	21
18.0	(114.4)	73	18.1	(114.9)	76	147.1	(21.33)	0.92	114.7	(16.64)	6	6	-	7
18.0	(113.9)	70	18.1	(114.8)	76	294.2	(42.66)	0.93	161.8	(23.47)	7	7	-	8
17.9	(113.7)	69	18.1	(114.6)	74	294.2	(42.66)	1.00	135.3	(19.63)	28	29	30	31
18.0	(114.1)	71	18.1	(115.0)	77	294.2	(42.66)	0.97	119.7	(17.26)	53	53	54	55
18.2	(116.1)	84	18.2	(116.2)	84	49.0	(7.11)	0.97	52.0	(7.54)	13	16	20	-
18.1	(115.1)	77	18.1	(115.4)	79	49.0	(7.11)	0.95	45.1	(6.54)	29	32	36	-
18.3	(116.5)	86	18.3	(116.6)	87	49.0	(7.11)	0.93	42.2	(6.12)	53	56	66	-
18.2	(115.9)	82	18.3	(116.4)	85	147.1	(21.33)	0.95	85.3	(12.38)	22	23	25	27
18.2	(116.2)	84	18.3	(116.7)	87	147.1	(21.33)	0.93	114.7	(28.03)	7	8	8	9
18.1	(115.7)	81	18.3	(116.6)	87	294.2	(42.66)	0.99	116.7	(16.93)	57	58	59	-
18.2	(115.8)	82	18.3	(116.6)	87	294.2	(42.66)	0.94	143.2	(20.77)	28	29	30	-
18.2	(115.9)	82	18.3	(116.7)	87	294.2	(42.66)	0.94	173.6	(25.18)	11	-	-	-
18.4	(117.1)	90	18.4	(117.2)	90	49.0	(7.11)	0.98	39.2	(5.69)	146	169	304	-
18.5	(118.0)	95	18.5	(118.1)	95	49.0		0.90	42.2		66	73	82	-
18.3	(117.0)	89	18.4	(117.1)	90	49.0		0.96	52.0		26	30	35	-
18.3	(117.0)	89	18.4	(117.1)	90	49.0		0.95	68.7		9	11	13	-
18.4	(117.1)	90	18.4	(117.5)	92	147.1		0.92	100.0		33	36	39	41
18.4	(117.5)	92	18.5	(118.0)	95	147.1		0.88	85.3		172	175	180	-
18.4	(117.2)	90	18.5	(117.7)	93	147.1		0.96	120.6		11	13	15	-
18.4	(117.3)	91	18.5	(118.1)	95	294.2		0.94	146.1		43	44	-	-
18.4	(117.1)	90	18.5	(117.8)	94	294.2		0.93	173.6		27	29	30	-
18.4	(117.3)	91	18.5	(118.1)	95	294.2		0.96	201.0		7	-	-	-

SOURCE: Stone & Webster Geotechnical Laboratory, Reference 208.

TABLE 2.5-32
Sheet 1 of 1
SUMMARY OF CYCLIC TRIAXIAL TESTS
BORROW SOURCE: MEANY ENGLES ROAD PIT

Initial			After Consolidation			Effective Consolidation Pressure		B at End of Consolidation	Cyclic Deviator Stress		Number of Cycles to:			
Dry Unit Weight		Relative Density (%)	Dry Unit Weight		Relative Density (%)						Double Amplitude Strain			Failure in Extension
kN/m^3	pcf		kN/m^3	pcf		kNm^2	psi		kN/m^2	psi	2.5%	5%	10%	
18.1	(115.2)	50	18.1	(115.2)	50	147.1	(21.33)	0.96	82.4	(11.95)	5	6	6	7
18.1	(115.3)	50	18.1	(115.3)	50	147.1	(21.33)	0.98	60.8	(8.82)	19	20	21	23
18.1	(115.3)	50	18.1	(115.3)	50	147.1	(21.33)	0.96	47.1	(6.82)	90	92	93	97
18.1	(115.2)	50	18.1	(115.2)	50	294.2	(42.66)	0.95	163.8	(23.75)	6	6	7	9
18.2	(115.5)	51	18.2	(115.5)	51	294.2	(42.66)	0.95	138.3	(20.05)	10	11	12	14
18.1	(115.4)	51	18.1	(115.4)	51	294.2	(42.66)	0.95	111.8	(16.21)	15	15	16	18
18.1	(115.2)	50	18.2	(116.0)	52	440.3	(63.99)	0.95	242.2	(35.13)	5	5	6	7
18.2	(115.5)	51	18.2	(115.5)	51	440.3	(63.99)	0.95	182.4	(26.46)	10	11	11	11
18.2	(115.5)	51	18.2	(115.5)	51	440.3	(63.99)	0.95	124.6	(18.06)	71	73	73	73
18.6	(118.5)	60	18.6	(118.5)	60	147.1	(21.33)	0.95	61.8	(8.96)	16	18	20	24
18.7	(118.7)	61	18.7	(118.7)	61	147.1	(21.33)	0.95	88.3	(12.80)	8	10	11	12
18.6	(118.4)	60	18.6	(118.4)	60	147.1	(21.33)	0.95	51.0	(7.40)	55	58	62	82
18.6	(118.5)	60	18.6	(118.5)	60	294.2	(42.66)	0.93	166.7	(24.18)	8	9	11	12
18.6	(118.5)	60	18.6	(118.5)	60	294.2	(42.66)	0.95	127.5	(18.49)	22	23	25	26
18.6	(118.5)	60	18.6	(118.5)	60	294.2	(42.66)	0.94	98.1	(14.22)	82	85	89	96
18.6	(118.5)	60	18.6	(118.5)	60	440.3	(63.99)	0.90	257.9	(37.40)	6	7	9	9
18.7	(118.8)	61	18.7	(118.8)	61	440.3	(63.99)	0.94	197.1	(28.59)	18	20	24	25
18.6	(118.6)	60	18.7	(119.4)	62	440.3	(63.99)	0.95	163.8	(23.75)	39	41	42	42
19.2	(122.1)	70	19.2	(122.1)	70	147.1	(21.33)	0.95	78.5	(11.37)	26	32	45	48
19.2	(122.0)	70	19.2	(122.0)	70	147.1	(21.33)	0.95	62.8	(9.10)	84	93	151	175
19.3	(122.6)	72	19.3	(122.6)	72	147.1	(21.33)	0.98	108.9	(15.79)	9	13	16	18
19.2	(122.1)	70	19.2	(122.1)	70	147.1	(21.33)	0.95	175.6	(25.46)	8	12	17	20
19.2	(122.1)	70	19.2	(122.1)	70	147.1	(21.33)	0.95	369.7	(63.62)	2	4	5	8
19.2	(122.1)	70	19.2	(122.1)	70	294.2	(42.66)	0.96	128.5	(18.63)	27	31	36	37
19.2	(122.2)	71	19.2	(122.2)	71	294.2	(42.66)	0.95	112.8	(16.36)	86	92	109	141
19.2	(122.0)	70	19.2	(122.0)	70	294.2	(42.66)	0.93	136.3	(19.77)	26	29	35	36
19.2	(121.9)	70	19.2	(121.9)	70	294.2	(42.66)	0.96	272.6	(39.54)	8	12	16	17
19.2	(122.1)	70	19.2	(122.1)	70	294.2	(42.66)	0.95	303.0	(43.95)	5	7	9	9
19.2	(122.2)	71	19.2	(122.2)	71	294.2	(42.66)	0.96	363.8	(52.76)	4	7	9	9
19.2	(122.2)	71	19.2	(122.2)	71	440.3	(63.99)	0.91	317.8	(46.08)	9	13	18	19
19.2	(122.3)	71	19.2	(122.3)	71	440.3	(63.99)	0.91	386.4	(56.03)	7	11	15	15
19.2	(122.0)	70	19.2	(122.0)	70	440.3	(63.99)	0.91	257.9	(37.48)	16	21	29	30
19.2	(122.0)	70	19.2	(122.0)	70	440.3	(63.99)	0.95	447.2	(64.85)	5	8	10	11

SOURCE: Goldberg-Zoino-Dunnicliff & Associates, Inc., Reference 209.

TABLE 2.5-33
Sheet 1 of 5
INSTRUMENTATION

Instrument	Location (Plant Grid Coordinates)	Elevation (ft)	Date of First Reading	Reading Frequency	Status
Stressmeters					
OC-1	5610.91S 1896.30W	198 - 196	September 1977	Monthly	Ongoing
OC-2	2599.45S 1877.52E	159 - 157	September 1977	Monthly	Ongoing
OC-5	357.00N 281.50W	141	July 1977	Monthly	Destroyed December 1979
OC-7	351.92N 295.00W	141	July 1977	Monthly	Failed September 1980
Inclinometers					
I-1	523.01S 382.77W	Top, 260 Bottom, 196	March 1977	Monthly	Ongoing
I-2	226.17N 392.92W	Top, 253 Bottom, 200	March 1977	Monthly	Discontinued June 1981
I-3	318.42N 439.75W	Top, 250 Bottom, 103	March 1977	Monthly	Ongoing
I-4	603.76S 535.77W	Top, 263 Bottom, 183	March 1977	Monthly	Ongoing
SI-1	332.42N 254.79W	Top, 220 Bottom, 74	December 1979	Monthly	Destroyed below el 146 in February 1980
SI-2	338.67N 366.83W	Top, 246 Bottom, 93	January 1980	Monthly	Ongoing
SI-3	334.17N 310.83W	Top, 232 Bottom, 78	January 1980	Monthly	Ongoing
SI-4	334.00N 300.00W	Top 232 Bottom, 72	January 1980	Monthly	Ongoing

TABLE 2.5-33
Sheet 2 of 5

Instrument	Location (Plant Grid Coordinates)	Elevation (ft)	Date of First Reading	Reading Frequency	Status
SI-5	360.17N 300.00W	Top, 232 Bottom, 71	January 1980	Monthly	Ongoing
SI-6	395.00N 329.92W	Top, 246 Bottom, 70	January 1980	Monthly	Ongoing
SI-7	342.5N 265.5W	Top, 220 Bottom, 71	January 1980	Monthly	Blocked at el 158 in January 1980
SI-8	355.79N 261.82W	Top, 216 Bottom, 72	November 1981	Monthly	Ongoing
SI-9	355.34N 252.0W	Top, 216 Bottom, 74	November 1981	Monthly	Ongoing
SI-10	379.8N 252.0W	Top, 242 Bottom, 76	November 1981	Monthly	Ongoing
SI-10	7.26N 113.85E	Top, 248 Bottom, 53	November 1981	Monthly	Ongoing
SI-21	62.0N 138.0E	Top, 246 Bottom, 56	November 1981	Monthly	Ongoing
SI-22	15.59N 230.0E	Top, 250 Bottom, 60	November 1981	Monthly	Ongoing
SI-23	59.93S 80.23W	Top, 203 Bottom, 51	November 1981	Monthly	Ongoing
803	191.1N 6.8E	Top, 243 Bottom, 117	May 1980	Monthly	Ongoing
805	55.7N 318.7E	Top, 246 Bottom, 88	May 1980	Monthly	Ongoing
806	73.1N 468.4E	Top, 249 Bottom, -37	June 1980	Monthly	Ongoing
810	87.3S 707.7E	Top, 247 Bottom, -37	May 1980	Monthly	Ongoing
820	289N 50E	Top, 238 Bottom, 52	November 1981	Monthly	Ongoing
821	263N 106E	Top, 240 Bottom, 52	November 1981	Monthly	Ongoing

TABLE 2.5-33
Sheet 3 of 5

Instrument	Location (Plant Grid Coordinates)	Elevation (ft)	Date of First Reading	Reading Frequency	Status
RS-2	1057.3S 493.7E	Top, 250 Bottom, 172	June 1980	Monthly	Ongoing
<u>Extensometers</u>					
MPX-1	344.90N 353.54W	221, 216, 211 201, 171, 141	June 1977 August 1981	Monthly	Discontinued August 1981
MPX-2	166.53N 55.63E	245, 235, 225 205, 165, 125	September 1977 June 1981	Monthly	Discontinued June 1981
EX-1	335.08N 367.86W	247, 227, 213 196, 172, 130 97	November 1979	Monthly	Ongoing
EX-2	339.08N 310.58W	232, 213, 196 172, 130, 110 72	December 1979	Monthly	Ongoing
EX-3	330.42N 229.92W	232, 213, 196 172, 113, 72	December 1979	Monthly	Ongoing
EX-4	360.92N 304.08W	232, 213, 196 172, 113	December 1979	Monthly	Ongoing
EX-5	346.96N 261.64W	220, 213, 196 172, 132, 110 72	October 1981	Monthly	Ongoing
EX-6	348.0N 252.78W	220, 218, 196 172, 132, 110 72	October 1981	Monthly	Ongoing
EX-20	2.16N 113.94E	252, 237, 222 202, 177, 156 127, 92, 54	October 1981	Monthly	Ongoing
HEX-1	54.2N 88.4E	(Horizontal depths) 10, 20, 40 60, 112, 150	October 1981	Monthly	Ongoing

TABLE 2.5-33
Sheet 4 of 5

Instrument	Location (Plant Grid Coordinates)	Elevation (ft)	Date of First Reading	Reading Frequency	Status
Piezometers					
PI-1	332.50N 265.50W	A - 131 B - 181	November 1979	Monthly	Ongoing
PI-2	335.25N 372.53W	A - 160 B - 190	November 1979	Monthly	Ongoing
PI-3	332.33N 304.92W	A - 130 B - 181	November 1979	Monthly	Ongoing
PI-4	415.83N 355.00W	A - 161 B - 194	November 1979	Monthly	Ongoing
PI-5	355.0N 225.0W	A - 123 B - 202	November 1981	Monthly	Ongoing
PI-20	5.53N 117.63E	157	November 1981	Monthly	Ongoing
PI-21	4.37N 229.87E	156	November 1981	Monthly	Ongoing
Gap Instruments					
G1-A	--	--	--	--	Planned
G1-B	63.5S 64.5E	214.5	November 1981	Monthly	Ongoing
G1-C	--	--	--	--	Planned
G1-D	--	--	--	--	Planned
G2-A	--	--	--	--	Planned
G2-B	72S 49E	214.5	November 1981	Monthly	Ongoing
G3	Interface between control building and south auxiliary bay				Planned

TABLE 2.5-33
Sheet 5 of 5

Instrument	Location (Plant Grid Coordinates)	Elevation (ft)	Date of First Reading	Reading Frequency	Status
G4	Interface between pipe chase and reactor building				Planned
G5-A	131N 6E	217.75	November 1981	Monthly	Ongoing
G5-B	125S 9W	217.75	November 1981	Monthly	Ongoing
G6	128S 62W	214.5	November 1981	Monthly	Ongoing

TABLE 2.5-34
Sheet 1 of 1
GRADATION LIMITS AND WEIGHT REQUIREMENTS
FOR BACKSLOPE ARMOR AND UNDERLAYERS

	Gradation and Weight Requirements		
Layer Type	Minimum	Maximum	Median
Backslope armor, tons	5	8	6
1st Underlayer, tons	1	2.5	1.5
2nd Underlayer, lb	75	250	150
3rd Underlayer, lb	2.3	15	7.5

TABLE 2.5-35
Sheet 1 of 1
GRADATION LIMITS FOR GRANULAR FILTERS

Filter Type	Sieve Designation*	Percent Finer by Weight
Filter No. 1 – fine material	1/4 in	100
	No. 4	95-100
	No. 8	86-91
	No. 10	71-87
	No. 16	56-72
	No. 20	48-63
	No. 40	30-44
	No. 50	20-35
	No. 80	5-16
	No. 100	0-16
	No. 140	0-5
Filter No. 2 – coarse material	2 in	100
	1 1/2 in	92-100
	1 in	75-97
	3/4 in	65-82
	1/2 in	50-68
	3/8 in	42-60
	1/4 in	28-46
	No. 4	18-37
	No. 8	0-14
	No. 10	0-8
	No. 16	0

* Size of opening in inches, or number of mesh/inch, U.S. standard.

TABLE 2.5-36
Sheet 1 of 1
STANDARD VERMICULITE GRADES

Vermiculite Grade	U.S.S. Sieve Size	Cumulative Percent Retained Maximum	Cumulative Percent Retained Minimum
No. 2, Density: 4 to 8 lb/cu ft (sieve analysis by weight)	4	5	0
	8	80	20
	16	99	75
	30	100	90
No. 4, Density: 6 to 10 lb/cu ft (sieve analysis by weight)	16	5	0
	30	65	15
	50	98	60
	100	100	90

TABLE 2.5-37
Sheet 1 of 1
CONCRETE SAND GRADATION REQUIREMENTS

U.S.S. Sieve Size	Percent Passing
3/8 in	100
No. 4	95 to 100
No. 8	80 to 100
No. 16	50 to 85
No. 30	25 to 60
No. 50	10 to 30
No. 100	2 to 10

TABLE 2.5-38
Sheet 1 of 1
QUALITY CONTROL PROGRAM FOR VERMICULITE

Inspection	Minimum Frequency
Vermiculite material	Each shipment[1]
Conditions of backfilled area and backfill material	Each placement[2]
Wetting and placing of vermiculite backfill	Each placement[3]

[1] Vermiculite is a standard industrial material and was shipped in bags. At each shipment, the material was subjected to normal receipt inspection to verify that it is Zonolite brand expanded vermiculite, Grade No. 2, manufactured by W. R. Grace and Company.

[2] Prior to placement, each specific area and wall to be backfilled was inspected for proper spacing and extraneous materials such as rock, mud, concrete, or construction rubble.

[3] During placement, it was verified that the vermiculite backfill was loosely placed without any compaction, and wetting of vermiculite backfill was properly done without flooding of the backfill space.

TABLE 2.5-39
Sheet 1 of 1
QUALITY CONTROL PROGRAM FOR VERMICULITE – CONCRETE

Test or Inspection	Test Designation	Minimum Frequency
Cement type	ASTM C150-70	Each shipment[1]
Vermiculite aggregate tests	ASTM C332-77 ASTM D75-73	Each shipment[2]
Vermiculite concrete tests	ASTM C39-72	5 sets of tests/ 50 cu yd[3]
Vermiculite concrete curing		Each placement[4]
Vermiculite concrete placement		Each placement[5]
Vermiculite concrete batching		Prior to each placement

(1) Verify that certified copies of mill test reports were in conformance with ASTM C150-70, Type II cement.

(2) Gradation, fineness modulus, and loose dry unit weight tests are required to show conformance with ASTM C332-77. Sampling of the aggregate is in conformance with ASTM D75-73.

(3) During placement, at least 5 sets of tests are required for each 50 cu yd placed or for each section placed that is greater than 15 ft in length. Each set of tests consists of 2 unit weights, air and concrete temperatures, wet density of compression test cylinder prior to test, 6 compression test cylinders (6 in by 12 in) for 7-day and 28-day strengths.

(4) Verify that the placed vermiculite concrete is cured at or above 40°F surface temperature for 72 hr.

(5) Verify that vermiculite concrete is placed on a base free from standing water, and the placed sections are protected against rain or falling water.

TABLE 2.5-40
Sheet 1 of 1
QUALITY CONTROL PROGRAM FOR VERMICULITE - SAND

Test or Inspection	Test Designation	Minimum Frequency
Vermiculite material		Each shipment[1]
Fine concrete aggregate gradation test	ASTM C136-71	1 per stockpile
Compaction equipment		Prior to each placement
Backfill condition		Each lift
Lift thickness		Each lift
Passes of compaction equipment		Each lift
In-place density	ASTM D1556-74	1 per alternate lift[2], if placed volume is less than 50 cu yd
Vermiculite-Sand batching		Prior to each placement
Vermiculite-Sand placement		Each placement[3]

(1) Verify that the material is Zonolite brand expanded vermiculite Grade No. 2, as manufactured by W. R. Grace and Co., by normal receipt inspection.

(2) Perform in-place density tests at least once for every 50 cu yd if the volume of each lift placed is more than 50 cu yd.

(3) Verify that the vermiculite-sand backfill does not drop in free fall a distance more than 6 ft.

TABLE 2.5-41
Sheet 1 of 1
QUALITY CONTROL PROGRAM FOR NUFOAM/RODAFOAM

Inspection	Minimum Frequency
Materials	Each shipment*
Placement of materials	Each placement

* Verify certification of compressible materials which are either Nufoam Type I as manufactured by Republic Systems Corporation, or Rodafoam II as manufactured by W. R. Grace & Company.

TABLE 2.5-42
Sheet 1 of 1
DESIGN SOIL AND ROCK PARAMETERS USED IN THE DESIGN
OF CATEGORY I STRUCTURES

Parameter	Bedrock		Category I Structural Fill	
	Value	Reference Section	Value	Reference Section
Bearing capacity	10 tsf – 1st 10 ft 20 tsf – Below 10 ft	2.5.4.10.1	4,000 psf	2.5.4.10.1
Gradation			(See Figure 2.5-102)	2.5.4.5.2
Cohesion			0	2.5.4.5.2
Angle of internal friction			35°	2.5.4.10.2
Saturated unit weight	165 pcf	2.5.4.2.1	140 pcf	2.5.4.10.2
Young's modulus				
Static	3.3×10^6 psi	2.5.4.2.2		
Dynamic	5.0×10^6 psi	2.5.4.7		
Shear modulus				
Dynamic	2.1×10^6 psi	2.5.4.7		
Rock squeeze	1-in inward movement of excavated rock walls	2.5.4.11	0.02 percent maximum strain at reactor excavation	2.5.4.11
Rock swell	Vertical strains of 0.005 percent	2.5.4.11		
Fault movement	1 in along radwaste fault in N70W direction	2.5.4.11	1 in along radwaste fault in N70W direction	2.5.4.11
Fault movement	1/4 in along thrust structures in screenwell shaft and lake water tunnels	2.5.4.11		

TABLE 2.5-43
Sheet 1 of 2
FOUNDATION DESIGN PARAMETERS FOR MAJOR STRUCTURES

Major Structure	Bearing Pressures[1]		Description of Foundation Material (Section 2.5.1.2.2)	Design Displacement[2] (in) Due to			Estimated Settlement[3] (in)	
	Actual Maximum	Maximum Allowable		Rock Squeeze	Rock Swell	Faulting	Total	Differential
Control building	12.85 kip/sq ft	20 kip/sq ft for el 242'-0" 40 kip/sq ft for el 209'-6"	Pulaski Formation - Sandstone interbedded with shale and siltstone	1.0	0.018	0	0	0
Diesel generator building	19.10 kip/sq ft	20 kip/sq ft	Oswego Formation - Sandstone	1.0	0.0014	0	0	0
Electrical tunnels	14.00 kip/sq ft	40 kip/sq ft	Oswego Formation - Sandstone	1.0	0.035	0	0	0
Main stack	9.80 kip/sq ft	20 kip/sq ft	Oswego Formation (Transition Zone) - Alternating sandstone, siltstone and shale	1.0	0	0	0	0
Reactor building, including auxiliary bays	12.54 kip/sq ft	40 kip/sq ft	Pulaski Formation - Sandstone interbedded with shale and siltstone	2.0	0	0	0	0
Screenwell building	12.27 kip/sq ft	40 kip/sq ft	Oswego Formation (Transition Zone) - Alternating sandstone, siltstone and shale	1.0	0.009	0	0	0
Standby gas treatment building and railroad access lock	10.24 kip/sq ft	20 kip/sq ft	Oswego Formation - Sandstone	1.0	0.05	0	0	0
Electrical ductline 907, manhole No. 1 CL-1E, portion of electrical ductline 922	1.84	4.0 kip/sq ft	Category I structural fill	NA	NA	NA	0.15	0.15
Radwaste building[4]	9.80 kip/sq ft	10 kip/sq ft	Oswego Formation - Sandstone	1.0	0	1.0	0.1	0.1

TABLE 2.5-43
Sheet 2 of 2

Major Structure	Bearing Pressures[1]		Description of Foundation Material (Section 2.5.1.2.2)	Design Displacement[2] (in) Due to			Estimated Settlement[3] (in)	
	Actual Maximum	Maximum Allowable		Rock Squeeze	Rock Swell	Faulting	Total	Differential
Turbine building[5]	6.27 kip/sq ft	20 kip/sq ft	Oswego Formation - Sandstone	0	0	0	0	0
Intake structures	6.00 kip/sq ft	20 kip/sq ft	Oswego Formation (Transition Zone) - Alternating sandstone, siltstone and shale	0	0	0	0	0
Intake tunnels	23.00 kip/sq ft	40 kip/sq ft	Pulaski Formation - Sandstone interbedded with shale and siltstone	1.0	0	0.25	0	0
Offgas, regeneration, and condensate demineralizer area[5]	12.60 kip/sq ft	20 kip/sq ft	Oswego Formation - Sandstone	0	0	0	0	0
Auxiliary service building (substructure only)	15.80 kip/sq ft	40 kip/sq ft	Pulaski Formation - Sandstone interbedded with shale and siltstone	1.0	0.035	0	0	0

(1) Bearing pressures listed represent most critical static or dynamic loading conditions. The unconfined compression strength of the rock ranges from 634 kips/sq ft (310 kg/sq cm) to 4,133 kips/sq ft (2,020 kg/sq cm) (Section 2.5.4.2.3). The bearing capacity safety factor for structural fill is 3.

(2) Estimated displacement along the fault plane is discussed in Section 2.5.4.11. No Category I structures are founded on a fault plane.

(3) In light of sound rock subgrade and zero settlement estimates, no settlements have been monitored.

(4) Radwaste building is not a Category I structure, and it has been determined that the upper boundary of the radwaste thrust zone is below the radwaste building foundation. However, the foundation has been very conservatively designed for rock squeeze and fault movement prior to the final determination of the boundary of the fault zone.

(5) Non-Category I structures.

TABLE 2.5-44
Sheet 1 of 1
RESULTS OF LIQUEFACTION ANALYSIS

Structure	Depth of Burial (ft)	Cyclic Shear Stress Induced by SSE		Cyclic Shear Stress Required to Cause Liquefaction		τ (psf)	Safety Factor
		γ_T of Structural Fill (pcf)	τ_{avg} (psf)	Relative Density (%)	Double Amplitude Strain (%)		
Ductline #907, #922	15	145	195.1	70	2.5	402.92	2.06
	20	145	258.2	70	2.5	518.6	2.01
	25	145	322.0	70	2.5	611.6	1.90
Manhole #1	17	-	149.2	70	2.5	232.1	1.56

NOTE: γ_τ = Total unit weight.

γ_{avg} = Average shear stress induced by the SSE.

γ = Cyclic shear stress required to cause liquefaction.

TABLE 2.5-45
Sheet 1 of 1
MAXIMUM RESPONSES OF INPUT EARTHQUAKES TO
DUCTLINES 907 AND 922 AND MANHOLE NO. 1, CL-1E

	Maximum Acceleration						Avg. Max. Acceleration of Three Earthquakes
	Helene		Santa Anita		Lake Hughes		
	Range	Avg.	Range	Avg.	Range	Avg.	
Ductlines 907 and 922, depth at 10'-14'	0.17 - 0.184 (g)	0.18 g	0.247 - 0.292 (g)	0.27 g	0.247 - 0.3 (g)	0.27 g	0.24 g
Manhole No. 1, CL-1E, depth at 0'-17'	0.16 - 0.205 (g)	0.18 g	0.208 - 0.328	0.27 g	0.208 - 0.338	0.27 g	0.24 g

NOTES:

1. Based on Figure 2.5-151.
2. The corresponding frequency for the buried ductlines and manhole is at least 20 Hz for the maximum acceleration of 0.24 g.

TABLE 2.5-45A
Sheet 1 of 1
SUMMARY OF FIELD PERCOLATION TEST RESULTS

Boring No.	Depth of Test (ft)	Soil Description	Field Permeability, K (cm/sec)
BB-3	2.0 - 5.0	Glacial Till: Sandy Silt	8.6×10^{-4}
BB-5	2.5 - 5.7	Lacustrine Clay, Silt, and Sand	0
BB-6	6.0 - 9.3	Glacial Till: Silty Gravel	4.6×10^{-4}
BB-7	0.0 - 4.5	Construction Fill: Crushed Stone	Very High
BB-7	8.3 - 9.4	Buried Topsoil: Organic Silt	1.0×10^{-4}
BB-8	0.0 - 4.5	Construction Fill: Crushed Stone	3.6×10^{-1}
BB-8	4.5 - 11.5	Construction Fill: Silty Sand	0
BB-9	19.5 - 24.7	Glacial Till: Sandy Silt	1.3×10^{-6}
BB-11	12.5 - 18.8	Lacustrine Silt	1.3×10^{-6}
BB-13	12.5 - 18.5	Lacustrine Silt and Clay	1.0×10^{-6}

TABLE 2.5-46
Sheet 1 of 1
SUMMARY OF FIELD PERCOLATION TEST RESULTS

Boring No.	Depth of Test (ft)	Soil Description	Field Permeability, K (cm/sec)
BB-3	2.0 - 5.0	Glacial Till: Sandy Silt	8.6×10^{-4}
BB-5	2.5 - 5.7	Lacustrine Clay, Silt, and Sand	0
BB-6	6.0 - 9.3	Glacial Till: Silty Gravel	4.6×10^{-4}
BB-7	0.0 - 4.5	Construction Fill: Crushed Stone	Very High
BB-7	8.3 - 9.4	Buried Topsoil: Organic Silt	1.0×10^{-4}
BB-8	0.0 - 4.5	Construction Fill: Crushed Stone	3.6×10^{-1}
BB-8	4.5 - 11.5	Construction Fill: Silty Sand	0
BB-9	19.5 - 24.7	Glacial Till: Sandy Silt	1.3×10^{-6}
BB-11	12.5 - 18.8	Lacustrine Silt	1.3×10^{-6}
BB-13	12.5 - 18.5	Lacustrine Silt and Clay	1.0×10^{-6}

TABLE 2.5-46A
Sheet 1 of 1
TRIAXIAL COMPRESSION TEST DATA SUMMARY
FOR WHELSKY PIT SPECIMENS

Type of Test: Consolidated Undrained

Type of Specimens: Compacted

Size of Specimens: 2.8 in Diameter by 6.8 in Height

Soil Description: Silty Sand

Sample Number	4	5	6
Depth (ft)	-	-	-
Specimen Properties			
Initial			
w (%)	9.0	8.0	8.0
r_d (pcf)	128.5	128.1	127.3
e	0.302	0.306	0.314
After Consolidation			
w (%)	9.0	7.9	8.0
r_d (pcf)	128.5	128.4	127.4
e	0.302	0.303	0.313

TABLE 2.5-46B
Sheet 1 of 1
TRIAXIAL COMPRESSION TEST DATA SUMMARY
FOR KELLER PIT SPECIMENS

Type of Test:	Consolidated Undrained		
Type of Specimens:	Compacted		
Size of Specimens:	2.8 in Diameter by 6.9 in Height		
Soil Description:	Gravelly Sand		
Sample Number	4	5	6
Specimen Properties			
Depth (ft)	-	-	-
Initial			
w (%)	9.0	9.0	9.0
r_d (pcf)	124.1	123.6	123.2
e	0.348	0.353	0.358
After Consolidation			
w (%)	8.9	8.9	9.0
r_d (pcf)	124.3	123.8	123.2
e	0.346	0.351	0.358

TABLE 2.5-46C
Sheet 1 of 1
TRIAXIAL COMPRESSION TEST DATA SUMMARY
FOR MEANY PIT SPECIMENS

Type of Test: Consolidated Undrained

Type of Specimens: Compacted

Size of Specimens: 2.8 in Diameter by 6.8 in Height

Soil Description: Silty Sand

Sample Number	2	3	4	5	6	7
Depth (ft)	-	-	-	-	-	-
Specimen Properties						
Initial						
w (%)	9.0	9.0	9.0	9.0	9.0	9.0
r_d (pcf)	127.2	127.0	126.9	126.9	126.9	126.7
e	0.316	0.317	0.318	0.318	0.318	0.321
After Consolidation						
w (%)	8.9	8.9	9.0	9.0	8.8	8.7
r_d (pcf)	127.6	127.2	127.0	127.0	127.5	127.3
e	0.297	0.315	0.318	0.318	0.313	0.315

TABLE 2.5-47
Sheet 1 of 1
SUMMARY OF ATTERBERG LIMITS TESTS ON
BOREHOLD SAMPLES

Parameter	Boring No.				
	BB-5	BB-8	BB-11		BB-19
Depth (ft)	2.5-4.0	12.0-13.5	12.5-13.5	15.0-16.5	5.0-6.5
Soil Description	Lacustrine Clay, Silt, and Sand	Lacustrine Clay and Silt	Lacustrine Silt	Lacustrine Silt	Lacustrine Clay
Unified Soil Classification	cl	cl	ml/cl	ml	cl
Natural Water Content (%)	28.5	22.2	12.2	10.1	18.5
Liquid Limit (%)	32.8	32.5	17.8	17.4	22.2
Plastic Limit (%)	20.7	16.5	13.4	13.5	13.2
Plasticity Index	12.1	16.0	4.4	3.9	9.0
Liquidity Index	64	36	-27	-87	59

TABLE 2.5-48A
Sheet 1 of 1
TRIAXIAL COMPRESSION TEST DATA SUMMARY
FOR LACUSTRINE SILTY CLAY SPECIMENS

Type of Test:	Consolidated Undrained		
Type of Specimens:	Undisturbed Block Samples		
Size of Specimens:	1.4 in Diameter by 3.5 in Height		
Soil Description:	Silty Clay, Moderately Plastic, Yellow-Brown		
Sample Number	82A	82B	82C
Depth (ft)	-	-	-
Specimen Properties			
Initial			
w (%)	21.9	23.7	22.4
r_d (pcf)	102.0	100.1	100.3
e	0.652	0.683	0.681
After Consolidation			
w (%)	25.1	25.1	26.2
r_d (pcf)	101.8	101.3	102.4
e	0.655	0.664	0.647

TABLE 2.5-48B
Sheet 1 of 1
TRIAXIAL COMPRESSION TEST DATA SUMMARY
FOR LACUSTRINE SILTY CLAY SPECIMENS

Type of Test:	Consolidated Undrained		
Type of Specimens:	Undisturbed Block Samples		
Size of Specimens:	1.4 in Diameter by 3.5 in Height		
Soil Description:	Silty Clay, Moderately Plastic, Gray with Yellow		
Sample Number	81G	81H	81I
Depth (ft)	-	-	-
Specimen Properties			
Initial			
w (%)	18.2	19.5	21.3
r_d (pcf)	108.1	106.1	102.3
e	0.560	0.578	0.646
After Consolidation			
w (%)	20.0	20.3	22.2
r_d (pcf)	108.1	108.0	104.6
e	0.560	0.561	0.611

TABLE 2.5-48C
Sheet 1 of 1
TRIAXIAL COMPRESSION TEST DATA SUMMARY
FOR ORGANIC SILT SPECIMENS

Type of Test:	Consolidated Undrained		
Type of Specimens:	Undisturbed Block Samples		
Size of Specimens:	1.4 in Diameter by 3.5 in Height		
Soil Description:	Organic Silt, Highly Plastic, Dark Gray to Black		
Sample Number	81A	81B	81C
Depth (ft)	-	-	-
Specimen Properties			
Initial			
w (%)	46.7	47.1	45.1
r_d (pcf)	67.0	65.7	69.0
e	1.423	1.469	1.352
After Consolidation			
w (%)	51.3	51.6	48.6
r_d (pcf)	67.1	67.3	71.4
e	1.417	1.413	1.275

TABLE 2.5-49
Sheet 1 of 1
SUMMARY OF LIQUEFACTION ANALYSIS

Parameter/ Boring	Ground El (ft)	Bedrock El (ft)	Min SPT Count	Total Vertical Stress, σ_v (psf)	Effective Vertical Stress, σ_v (psf)	Stress Req Coef, r_d	Stress Ratio Caused by Earthquake τ_{avg/σ_v}	Stress Ratio Req to Cause Liq. τ_{avg/σ_v}	Corr. Ratio to Cause Liq. 1.32 (τ_{avg/σ_v})	Safety Factor
BB-1	257.4	250.73	23	867.1	600.65	0.99	0.14	0.24	0.32	2.3
BB-2	261.0	253.3	20	1007.5	898.3	0.99	0.11	0.20	0.26	2.4
BB-6	263.5	254.2	82							
BB-7	266.0	246.1	22	1809.6	1315.4	0.97	0.13	0.22	0.29	2.2
BB-8	267.8	247.3	3	1755	1399.3	0.97	0.12	0.12	0.15	1.3
BB-9	270.8	246.1	>40							
BB-19	261.4	254.4	26	910.0	560.6	0.99	0.16	0.25	0.33	2.1
BB-20	276.8	252.3	>40							
BB-22	276.5	254.0	>40							
BB-23	273.0	253.9	>40							

GENERAL NOTES:

1. The total density of the in situ material is 130 pcf ±.
2. GWL is at el 255 ft-0 in.
3. For blow counts >40, no liquefaction will occur.

TABLE 2.5-50
Sheet 1 of 1
SUMMARY OF TESTING, INSPECTION, AND DOCUMENTATION REQUIREMENTS DURING EMBANKMENT BACKFILLING OPERATIONS

Quality Control Program for Category I Structural Fill

Test or Inspection	Test Designation	Minimum Frequency
Gradation test	ASTM C136-81	1/2,500 cu yd(3)
Moisture density test	ASTM D1557-78	1/2,500 cu yd(3)
Backfill conditions		Each lift
Backfill and compaction procedure		Each lift
Lift thickness		Each lift
Compaction equipment		Prior to each placement
Passes of compaction equipment		Each lift
In-place density test	ASTM D1556-74 ASTM D2922-81	For open areas, 1/500 cu yd(1) For confined areas, 1/100 cu yd(2)
Relative density test	ASTM D2049-69	Prior to placement

NOTES:

(1) Perform at least one test every lift if the lift is less than 500 cu yd.

(2) Perform at least one test every other lift if the lift is less than 100 cu yd.

(3) Initial test frequency was 1/1,000 cu yd; later changed to 1/2,500 cu yd when confidence in fill quality was attained.