EF3 COL 2.0-29-A

2.5.4 Stability of Subsurface Materials and Foundations

The site specific information provided in the following subsections addresses COL Item 2.0-29-A in the ESBWR Design Control Document (DCD). This section was developed following the guidance of Regulatory Guide 1.206 and Section 2.5.4 of NUREG-0800.

An extensive subsurface investigation was performed at the Fermi 3 site to characterize the site for potential siting of a new nuclear power plant. The site characteristics and subsurface conditions that could affect the safe design and siting of the plant were evaluated. Information concerning the properties and stability of all soils and rocks that may affect nuclear power plant facilities, under both static and dynamic conditions is presented in this section. Properties necessary for evaluation of vibratory ground motions associated with Ground Motion Response Spectra (GMRS) are included.

This section is organized as follows, as presented in Regulatory Guide 1.206:

- Geologic Features (2.5.4.1)
- Properties of Subsurface Materials (2.5.4.2)
- Foundation Interface (2.5.4.3)
- Geophysical Surveys (2.5.4.4)
- Excavations and Backfill (2.5.4.5)
- Groundwater Conditions (2.5.4.6)
- Response of Soil and Rock to Dynamic Loadings (2.5.4.7)
- Liquefaction Potential (2.5.4.8)
- Earthquake Design Basis (2.5.4.9)
- Static Stability (2.5.4.10)
- Design Criteria (2.5.4.11)
- Techniques to Improve Subsurface Conditions (2.5.4.12)

2.5.4.1 Geologic Features

Subsection 2.5.1.1 describes the physiographic, geologic, and tectonic setting of the 320 km (200 mi) radius site region and Subsection 2.5.1.2 describes the stratigraphy, structural geology, and engineering geology of the 40 km (25 mi) radius site vicinity to the 1 km (0.6 mi) radius site location.

Areas of potential surface or subsurface subsidence, solution activity, and uplift or collapse are discussed in Subsection 2.5.1.1.5, 2.5.1.2.4 and 2.5.1.2.5. Potential for zones of alteration or irregular weathering profiles, and structural weakness are discussed in Subsection 2.5.1.2.6.2.

Subsection 2.5.1.2.6.3 discusses the potential for unrelieved residual stresses in bedrock. Bedrock or soils that might be unstable are discussed in Subsection 2.5.1.2.6.4. Rock joints and discontinuities are discussed in Subsection 2.5.1.2.4.3.

Depositional and erosion history are presented in Subsection 2.5.1.1.2.3, 2.5.1.2.2, and 2.5.1.2.3.

2.5.4.2 Properties of Subsurface Materials

This section presents engineering properties of subsurface materials, together with their potential variability. The properties of subsurface materials are presented in Subsection 2.5.4.2.1 and are based on the field investigation and sampling program discussed in Subsection 2.5.4.2.2, and laboratory testing presented in Subsection 2.5.4.2.3.

2.5.4.2.1 Engineering Properties of Subsurface Materials

The subsurface materials encountered at Fermi 3 consist of approximately 9.0 m (30 ft) of overburden overlying bedrock. The overburden is comprised of fill, lacustrine deposits, and glacial till. The bedrock units below the overburden consist of Bass Islands Group, and Salina Group Units F, E, C and B. A detailed description of the site stratigraphy is presented in Subsection 2.5.1.2.3.

The depths to the top of each soil and bedrock layer encountered during the geotechnical investigation are presented in Subsection 2.5.4.2.2. The existing ground surface elevation at Fermi 3 ranges from approximately 176.5 to 177.4 m (579 to 582 ft) NAVD 88, with an average of approximately 177.1 m (581 ft). The approximate elevation ranges and average thickness for each subsurface material type, encountered at Fermi 3, are summarized in Table 2.5.4-201.

The following sections discuss development of static and dynamic engineering properties of the subsurface materials. The static and dynamic engineering properties are summarized in Table 2.5.4-202.

2.5.4.2.1.1 **Engineering Properties of Soils**

This section discusses the engineering properties of soils encountered at Fermi 3 including fill, lacustrine deposits and glacial till. Fill, lacustrine deposits and glacial till will be fully excavated under and adjacent to all Seismic Category I structures.

2.5.4.2.1.1.1 **Fill**

The surface deposits at the Fermi 3 site (elevation 177.7 m (583.0 ft) plant grade datum) consist of a permeable artificial fill that overlies the lacustrine deposits. A detailed description and classification of fill are provided in Subsection 2.5.1.2.3.2.3.3. The fill was used during construction of Fermi 2 to establish the current grade at the Fermi 3 site. Fill material was encountered from the ground surface to approximately 4.0 m (13 ft) below ground surface at Fermi 3, including a wide range of particle sizes from fine-grained material to cobble. It is classified as cobbles, well graded gravel (GW), poorly graded gravel (GP), well graded gravel with silt (GW-GM), and boulders.

During the subsurface investigation at Fermi 3, nine standard penetration tests (SPT) were performed within fill material. N-values from two tests were not included in calculating average values as the measured SPT N-values were over 50 blows per 30.5 cm (blows per foot) which might be due to the presence of cobbles. A limited number of SPT were performed due to large material size in the fill, and the top 1.5 to 1.8 m (5 to 6 ft) of the fill was vacuum excavated to check for underground utilities.

The measured N-values were corrected for effects from hammer efficiency, rod length, borehole size, and sampler type. The corrected N_{60} values ranged between 5 and 16 blows per 30.5 cm (blows per foot), with an average and a standard deviation of 11 and 4 blows per 30.5 cm (blows per foot), respectively. A total unit weight, γ_t, of 19.6 kN/m^3 (125 pcf) was assumed for fill material. Based on correlation with SPT N-value and average vertical effective stress, the relative density of fill material is estimated to be 65 percent with an effective angle of internal friction, ϕ', of 36 degrees. No laboratory tests were performed on fill material.

The current gradation of fill material is not suitable for foundation support or structural backfill for Fermi 3. Therefore, fill material will be excavated in Fermi 3 area. If desired, the fill material can be processed by crushing and sieving to produce a gradation suitable for use as engineered granular backfill for Fermi 3.

The static engineering properties of fill presented herein are suitable for stability analysis and design of temporary excavation support systems and slopes, where applicable.

Since fill material will be excavated in the Fermi 3 area and is not considered as competent material due to variability in the gradation, the dynamic engineering properties of the fill material are not needed for ground motion response analysis.

2.5.4.2.1.1.2 **Lacustrine Deposits**

A detailed description and classification of lacustrine deposits are provided in Subsection 2.5.1.2.3.2.3.2. A thin layer of lacustrine deposits was encountered from approximately elevation 173.1 to 171.6 m (568 to 563 ft) NAVD 88. It is classified as lean to fat clay with a minimum of 82 percent fines. The plasticity index of lacustrine deposits ranges from 17 to 37 percent, with an average of 27 percent. Its liquid limit ranges from 34 to 54 percent, with an average of 44 percent.

During the subsurface investigation at Fermi 3, 15 SPT were performed within the lacustrine deposits. In addition, laboratory tests were performed to characterize the properties of lacustrine deposits as shown in Subsection 2.5.4.2.3. The results of the field and laboratory tests together with their variability are summarized in Table 2.5.4-203.

The average undrained shear strength, S_U, measured from one unconfined compression (UC) and two unconsolidated-undrained triaxial compression (UU) tests is 24.4 and 38.8 kPa (0.51 and 0.81 ksf), respectively. In addition, consolidated-undrained triaxial compression tests with pore pressure measurements ($\overline{CU}$ tests) were performed on two samples, isotropically consolidated to their in-situ vertical effective stress. The average S_U measured from two $\overline{CU}$ tests is 55.5 kPa (1.16 ksf). An S_U of 43.1 kPa (0.9 ksf) was chosen for design based on the average S_U determined from the above three methods. The modulus of elasticity, E, was computed from plots of axial stress versus axial strain based on results from UU tests. The average calculated E is 5.6 MN/m^2 (116 ksf).

Six $\overline{CU}$ tests were performed on the lacustrine deposits. Two failure criteria, the maximum principal stress difference criterion and the peak principal stress ratio criterion, were considered when determining the effective shear strength parameters. The ϕ' based on the maximum principal stress difference criterion and the peak principal stress ratio

criterion is 29.3 and 31.0 degrees, respectively. The effective cohesion intercept, c', was neglected. Conservative estimates of the Mohr-Coulomb parameters with $\phi' = 29^{\circ}$ and c' = 0 are used for lacustrine deposits. Based on the pore pressure response of the lacustrine deposits from $\overline{CU}$ tests, lacustrine deposits are considered slightly overconsolidated soil.

Unit weight and moisture content were measured in the laboratory for lacustrine deposits. Average dry unit weight of the lacustrine deposits is approximately 16.5 kN/m^3 (105 pcf), with an average natural moisture content of 27 percent.

The lacustrine deposits are not considered suitable for foundation support or structural backfill for Fermi 3 due to low undrained shear strength. Lacustrine deposits material will be removed in the Fermi 3 area and consolidation characteristics of lacustrine clay are not needed.

The static engineering properties of lacustrine deposits presented herein are suitable for stability analysis and design of temporary excavation support systems and slopes, where applicable.

Since lacustrine deposits will be excavated in the Fermi 3 area and are not considered as competent material due low shear strength, the dynamic engineering properties of the lacustrine deposits are not needed for ground motion response analysis.

2.5.4.2.1.1.3 **Glacial Till**

A detailed description and classification of glacial till is provided in Subsection 2.5.1.2.3.2.3.1 Glacial till was encountered from approximately elevation 171.6 to 168.2 m (563 to 552 ft) NAVD 88. It is classified as lean with an average of 68 percent fines. The plasticity index of glacial till ranges from 7 to 27 percent, with an average of 14 percent. Its liquid limit ranges from 18 to 47 percent, with an average of 29 percent. In general, it is observed that the gravel content increases with increasing depth in the glacial till.

During the subsurface investigation at Fermi 3, 72 SPT were performed within the glacial till. In addition, laboratory tests were performed to characterize the properties of glacial till as discussed in Subsection 2.5.4.2.3. The results of the field and laboratory tests together with their variability are summarized in Table 2.5.4-204.

The average S_U measured from three UC and two UU tests is 124.5 and 76.6 kPa (2.6 and 1.6 ksf), respectively. In addition, the average S_U measured from three $\overline{CU}$ tests, isotropically consolidated to their in-situ vertical effective stress, is 167.6 kPa (3.5 ksf). Based on the above three methods, an average S_U of 129.3 kPa (2.7 ksf) was chosen for design.

Twelve $\overline{CU}$ tests were performed on the glacial till. The ϕ' and c' values, based on the maximum principal stress difference criteria, are 30.6 degrees and 0, respectively. The ϕ' and c' values, based on the peak principal stress ratio failure criterion, are 31.3 degrees and 14.4 kPa (0.30 ksf), respectively. In addition to the $\overline{CU}$ tests, a set of three direct shear tests was performed. The results indicated a ϕ' of 37 degrees and c' of approximately 0 for glacial till. Conservative estimates of the Mohr-Coulomb parameters, with $\phi' = 31^o$ and c' = 0 are used for glacial till. Based on the pore pressure response of glacial till from $\overline{CU}$ tests, the till is considered as heavily overconsolidated soil.

Unit weight and moisture content were measured in the laboratory for glacial till. Average dry unit weight of the till is approximately 17.9 kN/m^3 (114 pcf), with an average natural moisture content of 15 percent.

E was computed from plots of axial stress versus axial strain based on UU and $\overline{CU}$ laboratory tests results. The average calculated E is approximately 28.7 MN/m^2 (600 ksf).

The glacial till will be removed from under Seismic Category I structures. However, based on the characteristic of glacial till, it may be used to support Non-Seismic Category I structures.

The static engineering properties of glacial till presented herein are suitable for stability analysis and design of temporary excavation support systems and slopes, and foundation support, where applicable.

Subsection 2.5.4.4.1 discusses the techniques used to measure shear wave velocity (V_s) and compression wave velocity (V_p) and the results of the testing. The measured V_s ranges from 244 to 351 m/s (800 to 1,150 fps) based on the spectra analysis of surface waves (SASW) method. The measured V_s is used to calculate the low-strain shear modulus of glacial till. Subsection 2.5.4.7 discusses the shear modulus behavior at larger strain levels.

Based on static and dynamic engineering properties presented above, glacial till is considered as the upper most competent material at Fermi 3.

The dynamic engineering properties of the till are suitable for ground motion response analysis for Fermi 3.

2.5.4.2.1.2 **Engineering Properties of Bedrock**

This section discusses the engineering properties of bedrock units encountered at Fermi 3 including Bass Islands Group, and Salina Group Units F, E, C and B. Seismic Category I structures at Fermi 3 are directly founded on the Bass Islands Group or on fill concrete overlying the Bass Islands Group.

A detailed description and classification of the Bass Islands Group is provided in Subsection 2.5.1.2.3.1.2. Subsection 2.5.1.2.3.1.1 presents a detailed description and classification of Salina Group Units F, E, C and B.

In each of the following sections, the properties of each bedrock unit based on field and laboratory testing results are presented with their variability. The strength and deformation characteristics of bedrock units were also estimated using Hoek-Brown criterion (Reference 2.5.4-201), which uses the following five input parameters to estimate rock mass strength:

1. q_u of intact rock core samples.
2. Material index (mi) related to rock mineralogy, cementation, and origin.
3. Geological strength index (GSI) that factors the intensity and surface characteristics of rock mass discontinuities.
4. Disturbance factor (D) related to the level of the rock mass disturbance due to construction excavation and blasting.
5. Laboratory measured E of the intact rock core samples.

The input parameters, for each bedrock unit, used to estimate rock mass strength based on Hoek-Brown criterion are summarized in Table 2.5.4-205.

Finally, measured mean V_s and V_p are presented based on the results presented in Subsection 2.5.4.4.1. Subsection 2.5.4.4.1 discusses the techniques used to measure V_s and V_p and the results of the testing.

2.5.4.2.1.2.1 Bass Islands Group

Bass Islands Group is the uppermost bedrock unit encountered during Fermi 3 subsurface investigation. The approximate elevation of the bedrock unit ranges from elevation 168.2 to 140.8 m (552 to 462 ft) NAVD 88.

The results of the field and laboratory tests together with their variability are summarized in Table 2.5.4-206. The average percent recovery throughout this rock unit was 94 percent with an average rock quality designation (RQD) of 54 percent. The RQD is a measure of rock integrity determined by taking the cumulative length of pieces of intact rock greater than 4 inches long for the length of a core sampler advance and dividing by the length of the core sampler advance, expressed as a percentage.

Unconfined compressive strength, q_u, and E of the intact bedrock were determined by laboratory UC tests based on testing 20 intact rock samples. The q_u ranges from 46.0 to 153.7 MPa (960 to 3,210 ksf), with an average of 89.5 MPa (1,870 ksf). The E ranges from 15,900 to 78,600 MPa (331,200 to 1,641,600 ksf), with an average of 43,000 MPa (898,600 ksf). Twelve rock direct shear tests were performed along sample discontinuities to provide the residual friction angle along the discontinuities presented in Table 2.5.4-206. The residual friction angle along discontinuities ranges between 33 and 74 degrees, with a mean of 52 degrees.

The rock mass properties and Mohr-Coulomb parameters for the Bass Islands Group, based on Hoek-Brown criterion are presented in Table 2.5.4-207 and Table 2.5.4-208, respectively. The upper bound, mean, and lower bound are presented for each property.

Table 2.5.4-209 summarizes the statistical analysis of the measured velocities using the P-S suspension logger for the Bass Islands Group. The mean V_p for the Bass Islands Group varies from 4,023 to 4,389 m/s (13,200 to 14,400 fps), and the mean V_s varies from 2,012 to 2,225 m/s (6,600 to 7,300 fps). The Poisson's ratio of the Bass Islands Group varies from 0.33 to 0.34, based on the mean V_p and V_s.

2.5.4.2.1.2.2 Salina Group Unit F

The approximate elevation of Unit F ranges from elevation 140.8 to 103.3 m (462 to 339 ft) NAVD 88.

The results of the field and laboratory tests together with their variability are summarized in Table 2.5.4-210. The average percent recovery throughout this rock unit was 59 percent with an average RQD of 13 percent. The q_u and E of the intact bedrock were determined by laboratory UC tests based on 13 intact bedrock samples. The q_u ranges from 2 to 147 MPa (45 to 3,070 ksf), with an average of 45 MPa (940 ksf). The E of the bedrock ranges from 766 to 51,710 MPa (16,000 to 1,080,000 ksf), with an average of 25,343 MPa (529,300 ksf).

In-situ pressuremeter testing was performed at one boring location, RB-C6, within Unit F to characterize the in-situ E of the bedrock unit. Detailed discussion of the pressuremeter testing results is presented in Subsection 2.5.4.2.2.2.5. The E value estimated from pressuremeter testing ranges between 276 and 2,758 MPa (5,760 and 57,600 ksf), with an average of 996 MPa (20,800 ksf).

The rock mass properties and Mohr-Coulomb parameters for Unit F, based on Hoek-Brown criterion are presented in Table 2.5.4-207 and Table 2.5.4-208, respectively. The upper bound, mean, and lower bound are presented for each property.

Table 2.5.4-211 summarizes the statistical analysis of the measured velocities using the P-S suspension logger for Unit F. Based on the P-S suspension logger, the mean V_p in Unit F varies from 2,438 to 2,865 m/s (8,000 to 9,400 fps), and the mean V_s varies from 975 to 1,219 m/s (3,200 to 4,000 fps). Both are based on Borings TB-C5 and CB-C3. Poisson's ratio of Unit F, calculated using the mean of V_p and $V_{s,}$ varies from 0.39 to 0.40.

2.5.4.2.1.2.3 **Salina Group Unit E**

The approximate elevation of the Unit E ranges from elevation 103.3 to 75.0 m (339 to 246 ft) NAVD 88.

The results of the field and laboratory tests are summarized in Table 2.5.4-212. The average percent recovery throughout Unit E is 94 percent, with an average RQD of 72 percent. The q_u and E of the bedrock were determined by laboratory rock UC tests performed on eight intact bedrock samples. The q_u ranges from 22 to 132 MPa (450 to 2,760 ksf), with an average of 84 MPa (1,750 ksf). The E of the bedrock ranges from 13,100 to 64,121 MPa (273,600 to 1,339,200 ksf), with an average of 32,147 MPa (671,400 ksf).

The rock mass properties and Mohr-Coulomb parameters for Unit E, based on Hoek-Brown criterion are presented in Table 2.5.4-207 and Table 2.5.4-208, respectively. The upper bound, mean, and lower bound are presented for each property.

Table 2.5.4-213 summarizes the statistical analysis of the measured velocities using the P-S suspension logger for Unit E. The mean V_p in Unit E varies from 4,115 to 4,938 m/s (15,300 to 16,200 fps), and the mean V_s varies from 2,408 to 2,774 m/s (7,900 to 9,100 fps) based on deeper penetrating Borings TB-C5 and RB-C8. Poisson's ratio of Unit E, calculated using mean V_p and V_s, varies from 0.27 to 0.32.

2.5.4.2.1.2.4 **Salina Group Unit C**

The approximate elevation of the Unit C ranges from elevation 75.0 to 47.5 m (246 to 156 ft) NAVD 88.

Results of field and laboratory tests together with their variability are summarized in Table 2.5.4-214. The average percent recovery throughout Unit C was 99 percent, with an average RQD of 97 percent. The q_u and E of the bedrock were determined by laboratory UC tests on two intact rock samples. The q_u ranges from 67 to 105 MPa (1,390 to 2,200 ksf), with an average of 86 MPa (1,790 ksf). The E of the bedrock ranges from 32,405 to 40,697 MPa (676,800 to 849,600 ksf), with an average of 36,542 MPa (763,200 ksf). Excellent RQD was obtained for Unit C; therefore, the measured q_u and E, based on intact rock samples, are considered representative of the engineering behavior of the rock mass for Unit C.

The rock mass properties and Mohr-Coulomb parameters for Unit C, based on Hoek-Brown criterion are presented in Table 2.5.4-207 and Table 2.5.4-208, respectively. The upper bound, mean, and lower bound are presented for each property.

Table 2.5.4-215 summarizes the statistical analysis of the measured velocities using the P-S suspension logger for Unit C. Only Borings TB-C5 and RB-C8 penetrated Unit C. The mean V_p in Unit C varies from 4,846 to 4,907 m/s (15,900 to 16,100 fps) and the mean V_s varies from 2,713 to 2,743 m/s (8,900 fps to 9,000 fps). Poisson's ratio of Unit C, calculated using mean V_p and V_s, varies from 0.26 to 0.28.

2.5.4.2.1.2.5 Salina Group Unit B

The top of Unit B is approximately at elevation 47.5 m (156 ft) NAVD 88. The bottom of Unit B was not encountered during the subsurface investigation.

Results of field and laboratory tests together with their variability are summarized in Table 2.5.4-216. The average percent recovery throughout this bedrock unit was approximately 100 percent, with an average RQD of 97 percent. The q_u and E of the bedrock were determined by laboratory UC tests on two intact bedrock samples. The q_u ranges from 54 to 93 MPa (1,130 to 1,940 ksf), with an average of 74 MPa (1,540 ksf). The E ranges from 68,900 to 75,200 MPa (1,440,000 to 1,569,600 ksf), with an average of 72,000 MPa (1,504,800 ksf). An excellent RQD was obtained for Unit B; therefore, the measured q_u and E, based on intact rock samples, are considered representative of the engineering behavior of the rock mass for Unit B.

Rock mass properties and parameters for Mohr-Coulomb criterion for Unit B based on Hoek-Brown criterion, are presented in Table 2.5.4-207 and Table 2.5.4-208, respectively. The upper bound, mean and lower bound are presented for each property.

Table 2.5.4-217 summarizes the statistical analysis of the measured velocities using the P-S suspension logger for Unit B. Only Borings TB-C5 and RB-C8 penetrated into Unit B. The mean V_p in Unit B varies from 5,334 to 5,578 m/s (17,500 to 18,300 fps) and the mean V_s varies from 2,896 to 3,018 m/s (9,500 to 9,900 fps). The Poisson's ratio of the unit, calculated using the mean V_p and V_s, is 0.29.

2.5.4.2.2 Field Investigations

The field investigations consisted of a hydrogeological phase and a geotechnical phase. The hydrogeological investigation program is presented in Subsection 2.5.4.2.2.1 and the geotechnical investigation program in Subsection 2.5.4.2.2.2.

Both investigations were supervised by geologists/geotechnical engineers, who directed all aspects of the investigation programs and prepared detailed geologic logs for each boring. The investigations were conducted in accordance with an approved nuclear quality assurance program developed for the project.

2.5.4.2.2.1 Hydrogeological Investigation Program

The hydrogeological investigation was performed following the data collection and work plans developed specifically for the project. The hydrogeological investigation consists of piezometers and monitoring wells installation, packer and slug testing, downhole geophysics, and sampling and testing groundwater.

The site hydrogeologic characterization addresses the overall Fermi site, with additional focus on the area of Fermi 3. The hydrogeological investigation was conducted from April to June 2007.

The investigation focused on the following:

- The unconfined surficial groundwater located above the confining glacial till layer (Subsection 2.5.4).
- The confined Bass Islands Group aquifer.

Borings for piezometers and monitoring wells were used to collect information on the subsurface conditions, water level information, and hydraulic properties. Groundwater quality samples were only collected from monitoring wells, while groundwater levels were measured in both piezometers and monitoring wells. When a monitoring well was installed, all equipment used to drill and test borings and all equipment used to construct monitoring wells were cleaned to prevent the introduction of foreign material into the monitoring well that could affect the water quality data.

2.5.4.2.2.1.1 Piezometers and Monitoring Wells

The locations of piezometers and monitoring wells are shown on Figure 2.5.1-235 and Figure 2.5.1-236. Seventeen shallow and eleven deep piezometers and monitoring wells were installed during this program. Shallow piezometers and monitoring wells were installed to monitor the surficial unconfined groundwater. Deep piezometers and monitoring wells are screened within the Bass Islands Group to monitor the confined Bass Islands Group aquifer.

At most locations, piezometers and monitoring wells installed within the Bass Islands Group confined aquifer were paired with a piezometer or monitoring well installed in the surficial unconfined groundwater to allow comparison of the head between the two. This information was used to evaluate the hydraulic head difference between the two groundwater locations and confirm the artesian nature of the confined aquifer.

The shallow piezometers and monitoring wells are distributed across the site to allow flow evaluation of the unconfined groundwater. The following two areas of the surficial groundwater are of interest:

- Between Lake Erie and the drainage channel west of the existing Fermi units (overflow canal as shown on Figure 2.5.1-235).
- West of the overflow canal.

To develop an understanding of the relationship between the surficial groundwater and the overflow canal, shallow piezometers and monitoring wells are installed east and west of the overflow canal. To characterize the flow west of the overflow canal, six piezometers and monitoring wells are located at two distances from the channel and are distributed at approximately uniform distances north-south along the site boundary. East of the overflow canal, shallow piezometers and monitoring wells are distributed near water bodies surrounding the site and in the interior of the site to allow flow gradients to be determined. The screens for the shallow wells were installed above glacial tills, within lacustrine silts and clays, and/or within rock fill used to establish the plant grade. The shallow piezometers and monitoring wells are listed below, where “P-“ designates a piezometer and “MW-“ a monitoring well.

Piezometers and Monitoring Wells West of the Overflow Canal	**Piezometers and Monitoring Wells East of the Overflow Canal**
MW-381S	MW-383S
P-382S	MW -384S
MW-388S	P-385S
P-389S	MW -386S
MW-393S	MW-387S
	MW-390S
	MW-391S
	P-392S
	MW-395S
	P-396S
	P-397S
	P-398S

To complement the water levels obtained from shallow piezometers and monitoring wells, surface water level gauging stations were installed at locations adjacent to shallow piezometers and monitoring wells, as shown on Figure 2.5.1-235. The existing gauging station at the plant near the Fermi 2 intake in Lake Erie was used to establish the water level in Lake Erie.

The piezometers and monitoring wells in the Bass Islands Group confined aquifer are distributed broadly across the Fermi site. This distribution allowed evaluation of the flow of the confined aquifer below the site. Piezometer P-399D in the Bass Islands Group confined aquifer is located near the south boundary of the Fermi site, north of Pointe aux Peaux Road, to provide coverage to the south of Fermi 3; thereby providing a broader understanding of the bedrock groundwater flow.

The piezometers and monitoring wells within the bedrock were screened in more highly fractured zones to ensure that water samples and water levels were obtained. Visual inspection of core and in-situ packer testing was used to select screened intervals. For piezometers and monitoring wells not installed to the bottom of a boring, the open hole below the piezometer or monitoring well was backfilled with bentonite chips. The deep piezometers and monitoring wells are listed below.

Bedrock Piezometers and Monitoring Wells	
MW-381D	MW-391D
MW-383D	MW-393D
MW -384D	MW-395D
P-385D	P-398D
MW -386D	P-399D
MW-387D	

Existing Fermi piezometers and monitoring wells were used to supplement Fermi 3 installations, as discussed in Subsection 2.4.12.

2.5.4.2.2.1.2 **Soil/Bedrock Sampling**

Soil sampling for paired deep and shallow piezometers and monitoring wells were performed as follows:

- At each piezometer and monitoring well location, the soil was sampled continuously using sonic drilling, or split-barrel and/or thin-walled tubes.
- Where fill material at the site could not be sampled effectively with split-barrel samplers due to the particle size of the material, it was either sampled with the sonic rig or not sampled until the boring reached the bottom of the fill, where sampling was resumed using split-barrel and/or thin-walled tubes.

The split-barrel samplers and thin-walled tubes were used for soil sampling as discussed in Subsection 2.5.4.2.2.2.1. The SPT hammer energy measurements are also discussed in Subsection 2.5.4.2.2.2.1.

The bedrock was sampled continuously by rock coring with a triple-tube, swivel-type core barrel (Reference 2.5.4-202) using PQ size core bit. The bedrock core was placed in core boxes.

In accordance with Regulatory Guide 1.132, color photographs of all samples were taken after their removal from the borehole.

2.5.4.2.2.1.3 **Groundwater/Fluid Levels**

In accordance with Regulatory Guide 1.132, groundwater levels were measured in the boreholes during the course of the field investigation. The groundwater or drilling fluid level was recorded during the following times:

- Generally, at the start of each workday for borings in progress.
- At the completion of drilling.

Groundwater levels in piezometers and monitoring wells were measured monthly for a period of one year from June 29, 2007 to May 29, 2008. Concurrent with groundwater level measurements in piezometers and monitoring wells, the levels of surface water at gauging stations indicated on Figure 2.5.1-235 were also measured. The groundwater elevations in piezometers, and monitoring wells, and surface water elevations at the gauging stations were generally measured on the same work day.

2.5.4.2.2.1.4 **Downhole Logging**

Where poor bedrock core recovery was obtained, optical televiewer logging was performed to gather information on the bedrock where the core was not recovered. In borings MW-384D, MW-393D, P-385D, P-398D, and P-399D, additional geophysical testing was performed to

provide additional characterization information. At these locations, downhole logging consisted of the following:

- Natural gamma.
- Long & short normal resistivity.
- Single point resistance.
- Spontaneous potential.
- Fluid temperature.
- Fluid resistivity.
- Natural gamma.
- Caliper.
- Heat pulse flowmeter

Information from these tests was used to aid in selecting packer test zones, understand the hydrogeology, and correlate the bedrock geology across the site. If good core recovery was obtained, downhole geophysical logging of the core hole was not performed.

2.5.4.2.2.1.5 **Packer and Slug Testing**

Packer and slug testing were performed to estimate the hydraulic conductivity of bedrock and soil.

Packer testing was performed to estimate the permeability of selected intervals of bedrock. The intervals tested were selected based on visual inspection of bedrock core recovered and review of downhole logging results. Intervals with expected high and low conductivity were tested to provide a range of hydraulic conductivities for bedrock.

Slug testing was performed to estimate hydraulic conductivity in the overburden. Slug testing mechanically induces an instantaneous change in water level; pressure transducers then monitor the rate of recovery of groundwater level back to static level. Slug tests were performed in piezometers and monitoring wells installed within the unconfined surficial groundwater. The test results provide an estimate of hydraulic conductivity of the soil stratum in the vicinity of the screen zone.

The results of packer and slug testing are presented in Subsection 2.4.12.

2.5.4.2.2.1.6 **Piezometer and Monitoring Well Development**

Following installation, a piezometer or monitoring well was developed by air lifting or pumping until the discharge water was clear, as determined by the field personnel, and soundings indicated that all loose material had been removed from the piezometer or monitoring well.

2.5.4.2.2.1.7 **Chemical Testing of Groundwater and Surface Water**

Chemical testing of groundwater was performed to establish baseline conditions at the site. The groundwater samples for chemical testing were collected from all the shallow and deep monitoring wells installed as part of the Fermi 3 investigation. Each monitoring well was sampled once.

Surface water samples were also collected from Lake Erie in the area of the plant gauging station, and from the location of GS-1 in the overflow canal as shown on Figure 2.5.1-235.

The groundwater and surface water samples were tested for the following:

- Hardness
- Sulfate
- Silica, Dissolved
- Turbidity
- Phosphorous, Total
- Sodium
- Iron
- Phosphorous Orthophosphate
- Potassium
- Color
- Ammonia
- Calcium
- Bicarbonate
- Odor
- Nitrate
- Magnesium
- Arsenic (III)
- Nitrite
- Total Coliform
- Cadmium
- Organic Nitrogen
- Fecal Coliform
- Chromium, Total
- ORP/Eh
- Alkalinity
- Fecal Streptococci
- Chromium (VI)
- Total Suspended Solids
- Chloride
- Silica, Total
- Copper
- Lead
- Nickel
- Silver
- Biological Oxygen Demand
- Mercury
- Selenium
- Zinc
- Total Dissolved Solids
- pH
- Conductivity
- Temperature
- Phytoplankton (surface water only)
- Carbon dioxide (groundwater only)
- Dissolved oxygen
- Chemical Oxygen Demand

2.5.4.2.2.2 Geotechnical Investigation Program

A geotechnical site investigation was performed at the Fermi 3 site to achieve the following:

- Obtain subsurface information for understanding the site geology and estimating the engineering properties of subsurface materials.
- Characterize site conditions and develop site-specific seismic design criteria.
- Evaluate potential for seismically induced ground failure and other geological or geotechnical hazards.

Exploration activities were specifically developed to comply with requirements of 10 CFR 52, 10 CFR 50, Appendix S, and 10 CFR 100.23, using guidance provided in the following:

- NRC Regulatory Guide 1.132, Site Investigations for Foundations of Nuclear Power Plants.
- NRC Regulatory Guide 1.208, A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion.

The geotechnical investigation was performed from June to September 2007 in accordance with the data collection and work plans developed specifically for the project. Data collection in the geotechnical phase consists of soil borings, soil sampling, rock coring, test pits, surface and downhole geophysical testing for shear wave velocity measurements, rock pressuremeter testing, and other downhole geophysical loggings including optical televiewer, natural gamma, 3-arm caliper and heat pulse flowmeter logs. Soil borings and a test pit were completed at the locations shown on Figure 2.5.1-235. Table 2.5.4-218 shows the elevations, boring depths and depths to the top of each soil/bedrock layer observed from each boring.

2.5.4.2.2.2.1 Drilling and Sampling

Five drilling methods were used during the geotechnical subsurface investigation as follows:

1. Air vacuum excavation,
2. Sonic drilling per ASTM D6914 (Reference 2.5.4-203),
3. Rotary wash drilling (Reference 2.5.4-204),

4. Hollow-stem auger drilling per ASTM D6151 (Reference 2.5.4-205), and
5. Triple-tube wireline core barrel per ASTM D2113 (Reference 2.5.4-202).

Where required to check for underground utilities, the upper portion of the boring was advanced by removal of soil using vacuum excavation.

Rotary wash, hollow-stem auger and sonic drilling were performed in overburden to the top of the bedrock. The type of method used on each boring depended on the field observations of the subsurface conditions. The triple-tube wireline core barrel method was used for rock coring.

Sampling methods used in the field include the following:

- Continuous sampler using sonic drilling per ASTM D6914 (Reference 2.5.4-203).
- Two-inch split-barrel sampler per ASTM D1586 (Reference 2.5.4-206).
- Three-inch split-barrel sampler per ASTM D3550-01 (Reference 2.5.4-207).
- Three-inch diameter thin-walled tube sampler per ASTM D1587 (Reference 2.5.4-208).
- Pitcher sampler with a three-inch thin-walled tube (Reference 2.5.4-204).
- Bedrock coring per ASTM D2113 (Reference 2.5.4-202).

In accordance with RG 1.132, soil samples were collected at depth intervals no greater than 1.5 m (5 ft). Additional soil samples were collected as directed by the field personnel. The field personnel selected an appropriate sampling method based on his/her judgment and the ground condition encountered at the time of drilling.

Energy measurements (Reference 2.5.4-250) were performed to compute the energy transfer efficiency for hammers used for SPT during soil sampling. The energy measurements were performed prior to the beginning of the hydrogeological investigation. The average energy transfer ratio (ETR) from individual sample depths ranged from 89.7 to 91.5 percent for the Diedrich D-50, and from 57.3 to 74.5 percent for the Braynard Kilman 81 (BK-81). The overall transfer ratio was 90.5 percent for the Diedrich D-50, and 69.8 percent for the BK-81. It was noted that

the hammer efficiency of the BK-81 is low compared to a typical automatic hammer. In addition, the variability of the efficiency is large. Therefore, the SPT N-values obtained using the BK-81 were not used to correlate the material properties of subsurface materials encountered at the site. The efficiency of the D-50 hammer is considered high; therefore, the efficiency of 80 percent was used for energy correction, which resulted in conservative estimates of properties based on SPT N-value correlations.

Split-barrel samplers were used to collect disturbed samples in both granular and cohesive soils, while the thin-walled tubes were used to collect undisturbed samples of cohesive soils. When 2-inch split-barrel samplers were used, standard penetration tests were performed. The 3-inch split-barrel sampler was used to collect samples in gravel that were too large to be collected using the 2-inch sampler. Split-barrel samples were placed in jars and were used for soil identification and classification, as well as index property testing that did not require undisturbed samples. Where fill could not be sampled effectively with split-barrel samplers, sonic sampling was used until the boring reached material that could be sampled with split-barrel or thin-walled tube samplers.

Undisturbed samples were collected in glacial till or other cohesive soils using thin-walled tubes. Where the soil was too stiff to push with a thin-walled tube, a pitcher sampler was used. The pitcher sampler has an outer barrel with a cutting bit that fits over the thin-walled tube. The thin-walled tube is mounted on a spring; therefore, if the soil is too stiff, the thin-walled tube retracts inside the inner barrel, allowing the cutting edge of the outer barrel to advance the sampler. The pitcher sampler operates in a manner similar to a core barrel, in that the thin-walled tube does not rotate as the outer barrel rotates. Thin-walled tube samples were left in the tube and sealed for future testing. Significant care in the transportation and handling of these samples was required to provide a sample with minimal disturbance as discussed in Subsection 2.5.4.2.2.3.

The bedrock was sampled continuously by coring with a triple-tube, swivel-type core barrel (ASTM D2113) using a PQ or HQ size core bit. The bedrock core was placed in wooden core boxes in accordance with ASTM D5079 (Reference 2.5.4-209).

In accordance with RG 1.132, at least one continuously sampled boring was used for each safety-related structure. Since all safety-related

structures at the Fermi 3 site are founded on bedrock or fill concrete over bedrock (Subsection 2.5.4.3), the continuous sampling requirement was satisfied by continuous sonic sampling from the ground surface to the top of bedrock and then continuous rock coring in bedrock.

2.5.4.2.2.2.2 **Piezometers**

During the geotechnical investigation two additional piezometers were installed at the location of Borings EB/TSC-C2 and CB-C5 to provide additional data for deep excavations.

The piezometer at the location of Boring EB/TSC-C2 was screened at elevations between 164.3 and 166.8 m (539 and 544 ft) NAVD 88 to obtain the piezometric surface of the groundwater in the upper portion of the Bass Islands Group. The piezometer at location of Boring CB-C5 was screened at elevations between 150.0 and 151.5 m (492 and 497 ft) NAVD 88 in the Bass Islands Group to obtain the water pressure measurements below the planned base of the reactor building.

Subsection 2.5.4.2.2.1.3 discusses the groundwater/fluid level measurements during the course of the field investigation and the groundwater level measurements in the piezometers after installation.

2.5.4.2.2.2.3 **Test Pit**

Due to the presence of cobbles in the fill material in the upper 3.0 to 4.6 m (10 to 15 ft) of overburden, split-barrel and thin-wall tube sampling techniques were not effective in obtaining subsurface information in this layer. Therefore, a test pit was performed at the location of FWS/ACB-C1 to aid in characterizing the fill material. The location of the test pit is shown on Figure 2.5.1-236.

2.5.4.2.2.2.4 **Geophysical Testing**

Geophysical testing consisted of the following:

- P-S suspension logging
- Downhole seismic testing
- SASW surface geophysics
- Natural Gamma logging
- 3-Arm Caliper logging
- Heat pulse floweter logging
- Optical televiewer logging

- Borehole deviation survey logging

The testing performed is summarized herein, with more detailed discussion in Subsection 2.5.4.4.

P-S suspension logging, downhole seismic testing and SASW surface geophysics were performed to obtain a V_s profile for use in site seismic response analysis. V_s data are used to help characterize low strain soil deformation characteristics and to address amplification and deamplification effects of soils/rocks between generic rock, ground surface, and other interfaces in between. In the central and eastern United States (CEUS), generic rock is typically defined as that material with a V_s of about 2,804 m/s (9,200 fps) (RG 1.208). At the Fermi site, it was necessary to penetrate into Salina Group Unit B, where the V_s of the formation is at least 2,804 m/s (9,200 fps).

P-S suspension logging and downhole seismic testing were performed at Borings CB-C3, RB-C4, RB-C8 and TB-C5. Borings RB-C8 and TB-C5 were drilled to depths extending a minimum of 12.2 m (40 ft) into the Salina Group Unit B, approximately 143.3 m (470 ft) deep, to reach the generic bedrock layer. Borings RB-C4 and CB-C3 were approximately 82.3 m (270 ft) deep and penetrated to Salina Group Unit E. The locations of Borings CB-C3, RB-C4, RB-C8, and TB-C5 are shown on Figure 2.5.1-236.

To maintain the borehole stability in fill and to facilitate coring, steel casing was required in the overburden. Downhole seismic testing and P-S suspension logging are not effective in steel cased holes. To facilitate measuring V_s within glacial till overlying bedrock, the casing was withdrawn at RB-C6 to immediately below the fill. P-S suspension logging was then performed within glacial till. The casing was installed using the sonic drilling technique, so there was a concern that the glacial till in the borehole wall may have been disturbed. Therefore, the SASW method was used to provide a second measurement of V_s within the glacial till.

To provide measurement of V_s in the fill and redundant glacial till measurements, SASW was performed at RB-C4, RW-C1, MW-393 and MW-381.

Optical televiewer logging was performed in all borings under safety-related structures and Borings RW-C1 and RW-C3 under the Radwaste Building, where the stability of deep cuts is of concern and in borings where downhole seismic testing was performed. For borings

under non-safety-related structures, if poor bedrock core recovery was obtained, then televiewer logging was performed.

In conjunction with the televiewer, 3-Arm caliper and natural gamma logging were performed. The caliper was used to measure the borehole size. Natural gamma was used for identifying alteration zones, identifying shale zones, and providing information on bedrock types.

Heat pulse flowmeter logging was performed in Borings RB-C8 and TB-C5, where downhole seismic testing was performed. The data obtained from a heat pulse flowmeter can be used to interpret vertical flow, as well as to identify higher conductivity zones in bedrock.

The results from geophysical surveys discussed above are presented in Subsection 2.5.4.4.

2.5.4.2.2.2.5 **Pressuremeter Testing**

Rock pressuremeter testing was performed in Salina Group Unit F to provide direct in-situ measurement of the E of the unit. The E of Unit F could also be estimated from V_s, Hoek-Brown Criterion, and laboratory testing; however, the extra testing was implemented for Unit F because of the variable nature of Unit F and the low RQD. Rock pressuremeter testing was performed at Boring RB-C6, at the location planned for the Reactor.

The material being tested was a very complex geological unit consisting of interbedded limestone/dolomite/claystone/siltstone/shale and breccias with varying degrees of induration. The bedding thickness ranges from much less than an inch to greater than 3.0 m (10 ft). These units also contain poorly indurated or weathered claystone that had soil-like consistencies that in some cases were soft enough to be penetrated by thumb pressure. The larger hole size produced, due to drilling effects in this unit, limits the range of the strain that the bedrock will experience during a test. The bedrock tested was not fully classified, as the core recovery was less than 59 percent, with an RQD of 0.

Even with the limitations noted, tests were successfully performed to provide acceptable estimates of E. As discussed herein, the E values are considered to represent low estimates due to the nature of the bedrock and physical limitations of the pressuremeter.

2.5.4.2.2.2.5.1 Pressuremeter Testing Procedure

Rock pressuremeter testing locations (herein called test pockets) in Borings RB-C6 were selected by examining the boring logs, percent recovery values, gamma, caliper, and optical televiewer logs, and photographs of cores from adjacent borings. The test pockets were selected to test a range of bedrock qualities and types to provide a range of E values for Unit F.

The borehole was advanced with PQ wireline with a triple-tube core barrel to the top of each 1.5 m (5 ft) long target test pocket. A triple-tube NQ core barrel was then inserted through the PQ wireline casing and drilled for 1.5 m (5 ft) to produce the test pocket for the pressuremeter test.

The pressuremeter used for this study was a monocell pressuremeter. Three electronic displacement sensors, spaced 120 degrees apart are located at the center of the pressuremeter. The flexible membrane is placed over the sensors, and clamped at each end. The membrane is covered by a protective sheet of stainless steel strips. The unit is pressurized using compressed nitrogen to deform the adjacent material. The electronic signals from displacement sensors and the pressure sensor are transmitted by cable to the surface. During the test, the average expansion versus pressure is displayed on a computer screen. The pressuremeter is expanded by regulating the flow of compressed nitrogen to the pressuremeter testing unit. The pressuremeter was expanded gradually and a first unload/reload cycle was performed once resistance was encountered. The pressure was then increased beyond the previous maximum pressure and another unload/reload cycle was performed.

2.5.4.2.2.2.5.2 Results from Pressuremeter Testing

The details of the test pockets and test results are presented in Table 2.5.4-219. All pressuremeter tests were performed in Boring RB-C6 within Salina Group Unit F in which the lowest bedrock V_s and V_p were measured. As indicated by the strain before testing on Table 2.5.4-219, drilling of Boring RB-C6 resulted in an oversized hole in the test pockets. Caliper tests performed in Salina Group Unit F in adjacent borehole locations indicated that the borehole diameter changed erratically with depth indicating an uneven borehole wall through the unit. Between 4 and 6 percent expansion of the pressuremeter was required (except for

one test) to provide contact with the borehole wall in Boring RB-C6; therefore, the strain that could be applied to the bedrock was limited, as the pressuremeter would reach its expansion limit before more stress could be applied to the bedrock.

Three unload/reload cycles were performed, except in two tests in which only two cycles were applied (Tests FMI-3Z and FMI-8Z) and one test (Test FMI-12Z) in which four cycles were applied.

The "basic" pressure versus radial strain curve (i.e., the curve that excludes unload/reload cycles) had a concave upwards shape showing an increase in the tangent modulus with strain. The E computed from this tangent modulus is referred to as initial elastic modulus E_o. Significant increases in the unload/reload modulus (E_{ur}) were observed from cycle to cycle, which was to be expected considering that the corresponding E_o was also increasing as each unload/reload cycle was started from a higher pressure than the preceding cycle. For example, the average increase in the modulus from cycle 2 to cycle 3 was by a factor of about 1.2 to 2.5 as shown in Table 2.5.4-219. The ratio of the last unload-reload modulus, $E_{ur,last}$ to E_o ranged from about 5 to 22 for all tests, with the higher ratios corresponding generally to the higher E values.

The straight line portion (pseudo-elastic response) of the stress-strain curve for the bedrock was not reached for pressuremeter tests in Unit F except possibly for tests FMI-3Z, FMI-8Z and FMI-10Z. The radial strains measured during the tests are mostly the result of closing of joints in the bedrock and of the pressuremeter membrane deforming to conform to an uneven borehole wall. Both of these conditions would lead to the stiffening type of test response that was observed. There are open joints in the in-situ bedrock as evidenced by the loss of the drilling fluid in the Salina Group Unit F, but most likely the joints opened more near the borehole as a result of drilling disturbance. Thus, the radial strains that were observed would be higher than the in-situ undisturbed bedrock would have shown in an ideal borehole with a smooth wall. In view of these considerations, the selection of E_{ur} from last cycle as an estimate of the in-situ modulus is reasonable because the condition of the bedrock at the highest pressure level is probably closer to the in-situ undisturbed bedrock than at the lower pressure levels and previous unload/reload cycles.

The results of tests FMI-1Z, FMI-11Z and FMI-12Z, where high E were measured, are excluded in determining the design modulus. The average

E from the remaining tests should give a conservative estimate of the in situ E.

2.5.4.2.2.2.6 **Boring Backfill**

Boring RW-C1 was the first geotechnical boring backfilled. Backfilling was initiated using cement/bentonite grout placed using the tremie method, with the tremie pipe discharge at the bottom of the boring. Approximately 1,079 liters (285 gallons) of grout (grout weight varied from 1,545 to 1,654 kilogram/m^3 (12.9 to 13.8 pounds per gallon) was pumped, resulting in the grout level only rising from a depth of 82.3 m (270 ft) to 65.2 m (214 ft) below ground surface. Theoretically, this volume of grout was sufficient to backfill the boring approximately four times; therefore, the remainder of the boring was backfilled with bentonite chips.

For borings that terminated within the overburden or the Bass Islands Group, the hole was backfilled with either bentonite chips, or cement/bentonite grout and bentonite chips to within 0.3 to 0.6 m (1 to 2 ft) of the ground surface. The top 0.3 to 0.6 m (1 to 2 ft) was backfilled with gravel. If a boring collapsed and blocked-off at depth above the bottom of the boring, then the boring was backfilled from the point of collapse.

For borings that extended into Salina Group Unit F or deeper, the borings were cleaned out using a wireline core barrel advanced to the bottom of the boring. The wireline drill rods acted as a tremie pipe for bentonite chip placement. For the deeper portions of the boring, the bentonite chips were screened to separate the fines from the coarse chips. The coarse chips were slowly poured into a tremie pipe to prevent blocking at the groundwater level. The boring was sounded using a weighted measuring tape to confirm the depth of the chips and that the chips did not bridge within the boring. When the chip level was within 0.3 to 0.6 m (1 to 2 ft) of the ground surface, the placement of chips was stopped and the boring was filled with gravel.

2.5.4.2.2.3 **Storage, Handling, and Transportation of Soil and Bedrock Samples**

Collected soil and bedrock samples were documented and stored in a manner that would allow future retrieval for examination and index testing. The following procedures were followed to preserve sample integrity:

- ASTM Standards D4220, Standard Practices for Preserving and Transporting Soil Samples (Reference 2.5.4-210), and D5079, Standard Practices for Preserving and Transporting Rock Core Samples (Reference 2.5.4-209) were implemented.
- Samples were clearly labeled with the job name, job number, borehole number, depth, and date collected.
- Soil and bedrock samples were prepared for storage and documented using a sample custody record form.
- Field samples were delivered to the temporary storage facility on a daily basis.
- The sample custody forms were completed by the field geologist/engineer (or other field professional) and submitted to and accepted by the Site Coordinator for storage of the field samples in the temporary storage facility.

The Site Coordinator, or designee, retained the copies of the sample custody record form after relinquishing sample control to the laboratory manager at the offsite facility.

2.5.4.2.3 Laboratory Testing

The purpose of the laboratory testing program is to identify and classify soils and bedrock and to evaluate their physical and engineering properties.

The laboratory testing program was specifically developed to comply fully with requirements in Regulatory Guide 1.138. The following items discussed in Regulatory Guide 1.138 were addressed:

1. Approved sample handling, storage and transportation protocol was followed prior to testing as discussed in Subsection 2.5.4.2.2.2. Sample custody forms were used for sample shipment.
2. Soil samples were initially identified and described based on a visual description in accordance with ASTM D2488-06 (Reference 2.5.4-211) at the field and recorded in boring logs.
3. Soil samples that were not tested immediately after arrival from the field to the laboratory facility were stored in a separate room with temperature and humidity control. The relative humidity was maintained at or near 100 percent.

4. Classification tests were performed on soil samples to define the various soil types present across the site using the Unified Soil Classification System in ASTM D 2487-06 (Reference 2.5.4-212).
5. The selection of the soil and rock specimens for laboratory testing was performed following careful examination of boring logs and by reviewing photographs of soil and bedrock samples.
6. Samples selected for testing were considered to be either representative of a given stratum or considered to represent upper or lower limits of material properties.
7. Laboratory testing was performed in accordance with standard test procedures using calibrated equipment. No deviations from standard test procedures were made.

Laboratory testing to determine static engineering properties of soil and bedrock was performed in accordance with standard test procedures. The scope of the static laboratory testing program included the following:

- Natural moisture content tests per ASTM D2216-05 (Reference 2.5.4-213)
- Specific gravity tests per ASTM D854-06 (Reference 2.5.4-214)
- Atterberg limits tests per ASTM D4318-05 (Reference 2.5.4-215)
- Mechanical sieve analysis ASTM D422-63 (Reference 2.5.4-216)
- Hydrometer analysis per ASTM D422-63 (Reference 2.5.4-216)
- Percent finer than No. 200 sieve per ASTM D1140-00 (Reference 2.5.4-217)
- Consolidated-undrained triaxial compression tests with pore pressure measurements ($\overline{CU}$) per ASTM D4767-04 (Reference 2.5.4-218)
- Unconsolidated-undrained triaxial compression tests (UU) per ASTM D2850-03a (Reference 2.5.4-219)
- Unconfined compression tests (UC) on soil per ASTM D2166 (Reference 2.5.4-220)
- Unconfined compression tests (UC) on rock per ASTM D7012-07 (Reference 2.5.4-221)
- One-dimensional consolidation tests per ASTM D2435-04 (Reference 2.5.4-222)
- Direct shear tests on soil per ASTM D3080-04 (Reference 2.5.4-223)

- Direct shear tests on rock per ASTM D5607 (Reference 2.5.4-224)
- Hydraulic conductivity using a flexible wall permeameter per ASTM D5084 (Reference 2.5.4-225)
- Chemical analysis of soils per ASTM G51, ASTM D512 and ASTM D516 (Reference 2.5.4-226 through Reference 2.5.4-228)

The results for index properties, gradation and chemical analysis of soil samples are summarized in Table 2.5.4-220. Table 2.5.4-221 shows the results for strength tests of soil samples from $\overline{CU}$, UU and UC tests. The unconfined compressive strength tests and direct shear tests on discontinuities of rock core samples are summarized in Table 2.5.4-222 and Table 2.5.4-223, respectively.

The mean V_s for the Bass Islands Group, Salina Groups Units E, C and B were greater or equal to 2,042 m/s (6,700 fps) as shown in Table 2.5.4-202; therefore, no dynamic testing is required for these bedrock units. The need to perform dynamic testing was investigated for Salina Group Unit F, with a mean V_s ranging from 975 to 1,219 m/s (3,200 to 4,000 fps). It was concluded that no dynamic testing is required for this bedrock unit based on the following:

1. The shear strain that would be induced in Salina Group Unit F during the postulated design earthquake was estimated. The calculation was performed using the assumption of peak ground acceleration of 0.25 g and minimum V_s = 549 m/s (1,800 fps) measured at Boring TB-C5 at a depth of approximately 73.2 m (240 ft). The estimated shear strain would be approximately 0.0252 percent, which would indicate a ratio of G/G_{max} of approximately 0.91. To approximate a worst case, this G/G_{max} is based on sand between depths of 36.6 to 76.2 m (120 to 250 ft) (EPRI, 1993, Reference 2.5.4-229). The actual G/G_{max} for bedrock would be larger, indicating negligible modulus reduction for the bedrock. The statistics for the level of effective strain computed in the analyses for the 10^{-4} and 10^{-5} input ground motions are shown on Figure 2.5.2-271 and Figure 2.5.2-272, respectively. Figure 2.5.2-271 and Figure 2.5.2-272 show that within the elevation range of the Salina Group Unit F (elevations of approximately 103 to 141 m [339 to 462 ft]) the computed shear strains in the randomized site profiles were all less than or equal to 0.03 percent. Therefore, these results

confirm the estimated shear strain level in Salina Unit F is less than 0.03 percent.

2. Core recovery and RQD in Salina Group Unit F was poor. Testable samples from Salina Group Unit F were collected and preserved. These samples likely represent the more intact portions of the bedrock and hence testing under static or dynamic loading conditions would possibly give high values not representative of the overall Unit F.

Using an estimated average V_S of 305 m/s (1,000 fps) for till, the strain levels induced in till during the design earthquake was estimated to be 0.03 percent, with a resultant modulus reduction that would not exceed 20 percent. Therefore, only Resonant Column and Torsional Shear (RCTS) Testing is needed to obtain the dynamic response of the till. The RCTS testing will provide the dynamic response of soils up to shear strain of approximately 0.5 percent. No cyclic triaxial and cyclic direct simple shear tests are required. Figure 2.5.2-271 and Figure 2.5.2-272 show that within the elevation range of the glacial till (elevations of approximately 168 to 172 m [552 to 563 ft]) the computed shear strains in the randomized site profiles were all less than or equal to 0.1 percent. The RCTS testing provides the modulus reduction characteristic of glacial till up to shear strain of approximately 0.3 percent as shown on Figure 2.5.4-226. Therefore, these results confirm that cyclic triaxial and cyclic direct simple shear tests were not necessary since RCTS testing provides the modulus reduction characteristic for glacial till up to approximately 0.3 percent.

A number of dynamic tests on samples of glacial till to obtain the modulus reduction and damping curves as a function of strain were performed. Four RCTS tests were performed on glacial till as presented in Subsection 2.5.4.7.3.

2.5.4.3 **Foundation Interface**

Figure 2.5.1-236 shows the locations of the site explorations including borings, monitoring wells, piezometers and the test pit at Fermi 3 for the geotechnical investigation. Figure 2.5.4-201 shows the plan view of the excavation (discussed in Subsection 2.5.4.5) for the following ESBWR technology structures:

- Reactor Building/Fuel Building (RB/FB), Seismic Category I
- Control Building (CB), Seismic Category I

- Firewater Service Complex (FWSC), Seismic Category I
- Turbine Building (TB), Seismic Category II
- Radwaste Building (RW), Nonseismic

Figure 2.5.4-202 through Figure 2.5.4-204 show geologic cross-sections through the excavation plan view on Figure 2.5.4-201. The geologic cross-sections show the relationship of the foundations of all Seismic Category I structures to the subsurface materials.

Table 2.5.4-224 provides the foundation elevations of the major structures in the Power Block area. The key dimensions of the foundations for the RB/FB, CB, and the FWSC are provided in the DCD Table 3.8-13. The finished ground level grade (finish grade) of elevation 179.6 m (589.3 ft) NAVD 88 was obtained from Subsection 2.4.1.

The RB/FB embedment depth is 20 m (65.6 ft) below finish grade. The base elevation of the RB/FB foundation is at 159.6 m (523.7 ft) NAVD 88. As shown on Figure 2.5.4-202 and Figure 2.5.4-203, the base of the RB/FB foundation lies on Bass Islands Group. The CB embedment depth is 14.9 m (48.9 ft) below finish grade resulting in a foundation base elevation of 164.7 m (540.4 ft) NAVD 88. As shown on Figure 2.5.4-202, the base of the CB foundation is also founded on Bass Islands Group. The embedment depth of the foundation base of the FWSC is 2.35 m (7.7 ft), at elevation 177.3 m (581.6 ft) NAVD 88. The FWSC foundation base is within fill material as shown on Figure 2.5.4-202; however, the existing subsurface materials including fill, lacustrine and glacial till are to be removed and backfill consisting of fill concrete will reestablish the foundation grade of the FWSC. Concrete is used to backfill the gap between the RB/FB and CB and excavated bedrock up to the top of the Bass Islands Group bedrock at Elevation 168.2 m (552.0 ft) NAVD 88. The gap between the RB/FB and the CB up to the top of the Bass Islands Group bedrock at Elevation 168.2 m (552.0 ft) NAVD 88 is also backfilled with fill concrete.

The static and dynamic engineering properties of the fill concrete under the FWCS are discussed in Subsection 2.5.4.5.4.2.

Figure 2.5.4-203 shows that the foundation base of the Radwaste Building (RW) is founded on Bass Islands Group, while the foundation base level of the Turbine Building (TB) is within glacial till as shown on Figure 2.5.4-203 and Figure 2.5.4-204. The glacial till will be removed underneath the TB and replaced with fill concrete to reduce the

interaction between the TB and RB since they are located in close proximity.

Logs of Fermi 3 borings, monitoring wells, piezometers and test pit are presented in Appendix 2.5DD.

2.5.4.4 Geophysical Surveys

Geophysical surveys performed are listed in Subsection 2.5.4.2.2.2.4. Details of the testing are discussed herein. The geophysical surveys performed to characterize the dynamic characteristics of soils and bedrock are discussed in detail in Subsection 2.5.4.4.1. Subsections 2.5.4.4.2 to Subsection 2.5.4.4.3 discuss the results of other geophysical surveys performed.

2.5.4.4.1 Geophysical Surveys for Dynamic Characteristics of Subsurface Materials

The dynamic characteristics of soil and bedrock were measured using downhole P-S suspension logging, downhole seismic testing, and surface SASW logging. P-S suspension logging was performed with an OYO Model 170 Suspension Logging system, serial number (S/N) 15014, manufactured by OYO Corporation. The P-S suspension logger obtained in-situ horizontal shear and compressional wave velocity measurements at 0.5 m (1.6 ft) intervals in uncased boreholes. Downhole seismic wave velocity measurements were performed, using Geostuff Model BHG-3 S/N B3015, and B3031 3-component borehole geophones, at 0.8 to 1.5 m (2.5 to 5 ft) intervals in uncased boreholes. SASW logging was performed using OYO Geospace 4.5 Hz geophones. (Reference 2.5.4-248)

P-S Suspension logging was used to obtain V_s and V_p of the soil and bedrock units. Downhole seismic testing was used to obtain V_s and V_p in the bedrock. SASW was used to obtain V_s in the soil. Overburden is removed underneath the FWSC and the foundation is placed on fill concrete over bedrock. Therefore, most of the effort for the geophysical surveys was exerted on characterizing the dynamic properties of bedrock units. However, effort was also exerted to characterize the dynamic properties of soil layers at the Fermi 3 site. P-S Suspension logging was performed at one borehole location to obtain V_s of the overburden mainly in glacial till. In addition, surface seismic wave velocity measurements were obtained at four locations using the SASW method. The purpose of

the SASW survey was to obtain V_s profiles in the upper 9.1 m (30 ft) for fill and glacial till layers presented at the site.

The results of all V_s and V_p measurements using various methods are presented in Reference 2.5.4-248. Detailed discussions of the geophysical surveys used for dynamic characterization of soils and bedrock are presented in the following subsections.

2.5.4.4.1.1 **P-S Suspension Logging and Downhole Seismic Testing in Bedrock Units**

Both the P-S suspension logger and downhole seismic testing procedures were used to obtain V_s and V_p of bedrock units at Fermi 3. The P-S Suspension method was considered as the primary method for obtaining the V_s and V_p profile, while the Downhole Seismic method was used to validate the results measured using P-S Suspension logging.

The procedure for P-S suspension seismic velocity logging (Reference 2.5.4-230) and the downhole seismic velocity logging procedure (Reference 2.5.4-231) were followed for P-S suspension logging and downhole seismic testing, respectively.

Repeated collapse of the boreholes in the 33.5 to 62.5 m (110 to 205 ft) depth range (Salina Group Unit F) was experienced and resulted in oversized borehole and irregular borehole shapes. Effectiveness of P-S suspension logging and downhole seismic testing can be limited in oversized sections of a borehole. Consideration was given to installing permanent PVC casing in the collapsing zones. However, based on the inability to grout Borings RW-C1 due to grout loss to the formation, the ability to grout the annulus outside the casing was considered doubtful. The problem was overcome by using the following methodology for V_s and V_p measurements:

- Use of temporary steel casing to below the borehole collapse zone.
- P-S suspension logging and downhole seismic testing below the temporary casing to the bottom of the boring.
- Removal of temporary steel casing and performing P-S suspension and downhole seismic logging in the Bass Islands Unit above the borehole collapse zone.
- Perform P-S suspension logging and/or downhole seismic testing at select locations within Salina Group Unit F (collapsing zone).

Table 2.5.4-225 provides a summary of testing locations, logging methods and depth ranges where measurements were obtained.

For downhole seismic testing, both V_s and V_p were measured in Borings CB-C3, RB-C4, and RB-C8, but only V_p was measured in a small portion of Boring TB-C5. Limited measurements were performed in Salina Group Unit F in any of the borings due to oversized holes and irregular hole shapes. However, arrival time of shear and compression waves above and below the interval of the oversized zones could be measured using the downhole seismic method; therefore, average V_s and V_p across the oversized zone were measured. The downhole measurements of V_p in Boring TB-C5 were performed only between depths 85.3 to 99.1 m (280 to 325 ft) due to equipment problems associated with attempting downhole testing in an open boring.

The quality of the velocity data measured using the P-S Suspension probe was judged based upon five criteria:

1. Consistency of data between velocities measured from receiver to receiver (R1–R2) and velocities measured from source to receiver (S–R1).
2. Consistency of relationship between V_p and V_s measured in a borehole (excluding transition to saturated soils).
3. Consistency of measured V_p and V_s between adjacent depth intervals in a borehole.
4. Clarity of compression wave and shear wave onset, as well as damping of later oscillations.
5. Consistency of measured velocity profiles between adjacent borings.

The evaluation of the quality of the velocity data measured using the P-S Suspension probe was performed based on the above criteria (Reference 2.5.4-248). Overall results obtained from P-S suspension logging are acceptable for all analysis purposes. The results are discussed in detail herein.

Evaluation of the quality of the velocity data measured using the downhole seismic method was performed mainly based in the waveforms received. In general, the quality of the compression waveforms obtained from seismic dowhhole testing were good and easy to interpret; therefore, the measured V_p is reliable. However, the quality of the shear

waveforms received from seismic dowhhole testing was poor and contaminated by noise; therefore, the measured V_s from the downhole seismic method was not considered reliable.

Analyses were performed to compare V_s and V_p measurements obtained with other subsurface information such as RQD, caliper, natural gamma, and optical televiewer logs. The study was mainly focused on the Bass Islands Group and Salina Group Unit F where RQD was low. The purpose of the analysis was to understand if the measured V_s and V_p were representative of the actual subsurface conditions. In addition, the analyses provided insight regarding why waveforms were highly variable between 9.1 and 36.6 m (30 and 120 ft) (in Bass Islands Group) in all boreholes.

Figure 2.5.4-205 through Figure 2.5.4-208 compare the percent RDQ and measured V_p and V_s for receiver to receiver (R1 to R2) measurements using the P-S suspension logging results for Borings TB-C5, RB-C8, CB-C3, and RB-C4, respectively. The measured discrete velocities typically increase with increasing RQD, and visa versa.

Irregular readings were obtained in the Bass Islands Group between the depths of 9.1 and 36.6 m (30 and 120 ft). The waveforms were difficult to interpret in this depth range in most boreholes. The variability observed in the measured V_p and V_s from P-S Suspension logs in the Bass Islands Group can be better explained based on optical televiewer logs. Figure 2.5.4-209 through Figure 2.5.4-212 compare the optical televiewer logs and the measured velocities in Borings TB-C5, RB-C8, CB-C3 and RB-C4, respectively. These figures indicate that the variability in the measured V_p and V_s within the Bass Islands Group is mainly caused by geologic features such as fractures, bedding planes, brecciation, oolitic rock, and pitting of the bedrock. At these features, the velocities tend to be lower.

For the P-S suspension instrumentation, the separation of R1–R2 is 1 m (3.3 ft) and the separation of S–R1 is 1.9 m (6.3 ft). The inconsistency between receiver to receiver (R1–R2) and source to receiver (S–R1) profiles in the Bass Islands Group was because the volume of bedrock sampled from near to far receivers (R1-R2) is less than the volume of bedrock sampled from the source to near receiver (S-R1); therefore, R1-R2 velocity will show greater variability due to the nature of discontinuities in Bass Islands Group (Subsection 2.5.1.2.4.3) as compared to the S-R1 velocity.

Understanding the variability observed in the measured V_p and V_s in the Salina Group Unit F can be aided using natural gamma logs. Figure 2.5.4-213 and Figure 2.5.4-214 show the comparison of the natural gamma logs and the measured velocities in Borings TB-C5 and CB-C3, respectively. Figure 2.5.4-213 and Figure 2.5.4-214 show that the variability in the measured V_p and V_s within the Salina Group Unit F correlates with the variability in the natural gamma value in Boring TB-C5 and CB-C3, respectively. The higher gamma value indicates the presence of shale or claystone and the lower gamma value indicates dolomite or limestone. The measured V_p and V_s increase in the areas where dolomite and/or limestone are present.

Based on the above observations, it is concluded that the variability of the measured V_s and V_p from P-S Suspension logs in the Bass Islands Group and Salina Group Unit F can be correlated directly with observed geologic features; therefore, the measured V_s and V_p are considered representative of the actual ground conditions.

Figure 2.5.4-215 shows all measured V_p using P-S suspension and downhole seismic methods in one plot. It is shown that all the V_p measurements at different borehole locations using both P-S and downhole seismic methods agree with each other, except for the V_s measured in Boring RB-C8 within the Bass Islands Group using the downhole seismic method.

Figure 2.5.4-216 shows all measured V_s using P-S suspension and downhole seismic methods in one plot. Although the arrival of shear waves for the downhole seismic method are difficult to interpret due to poor quality shear wave forms, the downhole V_s values in general agree with V_s obtained using P-S suspension logging. At Boring CB-C3, from approximately El. 167.6 to 143.3 m (550 to 470 ft) (in Bass Islands Group), the downhole V_s agrees with the measured V_s using P-S suspension logger. At Boring RB-C8, the V_s obtained, from approximately El. 167.6 to 143.3 m (550 to 470 ft) (in Bass Islands Group), using downhole seismic method is close to the lower bound of the measured V_s using P-S suspension logger. The V_s obtained, from approximately El. 143.3 to 94.5 m (470 to 310 ft) (in Salina Group Unit F and upper portion of Salina Group Unit E), using downhole seismic method at Boring RB-C8 agrees with the measured V_s using P-S suspension logger.

Since good quality compression wave forms are obtained from the downhole seismic method, the V_s in Boring RB-C8, from El. 167.6 to

143.3 m (550 to 470 ft), can be calculated using the following equation (Reference 2.5.4-232):

$$\nu = \frac{0.5\left(V_p / V_s\right)^2 - 1}{\left(V_p / V_s\right)^2 - 1} \qquad \text{[Eq. 1]}$$

where ν is the Poisson's ratio, and V_s and V_p are the shear and compression wave velocities, respectively. The Poisson's ratio of the bedrock was determined from P-S suspension data. The calculated V_s at RB-C8 using V_p obtained from downhole seismic method and a Poisson's ratio of 0.33 is 1,859 m/s (6,100 fps) which agrees with the P-S suspension data.

The measured V_p for the bedrock at Fermi 3 was compared to the measured V_p at Fermi 2. The measured V_p using the seismic refraction surveys at Fermi 2 site for Bass Islands Group, Salina Group Unit F and Salina Group Unit E are within the range of the measured V_p at Fermi 3. The measured V_p at Fermi 2 for the Salina Group Unit C and B were lower than the range of measured V_p at Fermi 3; the difference is less than 15 percent and 5 percent for Unit C and Unit B, respectively.

2.5.4.4.1.2 **P-S Suspension Logging and Spectral Analysis of Surface Wave in Soil Layers**

Seismic wave velocities were measured in the overburden at Boring RB-C6. Waveform consistency and clarity in this borehole are poor, but the results are considered acceptable, because soil shear wave velocities measured using the P-S Suspension method agree with those measured using SASW method. The measured V_p and V_s from the P-S method in Boring RB-C6 were compared to the measured N-values and to the gravel content in the all borings at Fermi 3. The V_s increases with increasing N-value, and with increasing gravel content as shown on Figure 2.5.4-217 and Figure 2.5.4-218, respectively. The minimum measured V_s for glacial till is approximately 305 m/s (1,000 fps).

The SASW method was used close to Borings RB-C4, RW-C1, MW-381, and MW-393 to obtain V_s of overburden. Borings RB-C4 and RW-C1 were located in the Fermi 3 power block area as shown on Figure 2.5.1-236, while Borings MW-381 and MW-393 were located at least 610 m (2000 ft) away from power block area as shown on Figure 2.5.1-235. At locations of Borings MW-381 and MW-393, the glacial till layer exists at or within 0.6 m (2 ft) from ground surface, and the top of the bedrock is

within 4.6 m (15 ft) of the ground surface. At Borings RB-C4 and RW-C1, the fill extends from the ground surface to a depth of about 4.6 m (15 ft), and the till extends from a depth of 4.6 m (15 ft) to the top of bedrock at a depth of approximately 9.8 m (32 ft). Where the till or fill are the surficial soil, SASW results are clearer to interpret, because for the initial readings near the ground surface there is no interference from other materials.

The measured V_s using the SASW method for Boring RB-C4, RW-C1, MW-381, and MW-393 is shown on Figure 2.5.4-219. In the fill, V_s in the Fermi 3 power block area generally decreases with depth from ground surface to approximate 4.6 m (15 ft) below the ground surface, then V_s increases when glacial till layer is encountered. The V_s near MW-381, increases with increasing depth. The V_s near MW-393 decreases from ground surface to approximately 1.8 m (6 ft) below ground and then increases to approximately 320 m/s (1050 fps) below 1.8 m (6 ft). The measured V_s ranges from approximately 244 to 351 m/s (800 to 1150 fps) for glacial till. Below 0.9 m (3 ft), V_s in the fill is approximately 244 m/s (800 fps).

2.5.4.4.2 Natural Gamma, 3-Arm Caliper, Heat Pulse Flowmeter, and OTV Logging

Natural gamma and 3-arm caliper logs were acquired using a MGX II digital logging system manufactured by Mount Sopris Instrument Company. The natural gamma and caliper loggings were performed concurrently. The optical televiewer (OTV) logs were acquired using a Robertson Geologging Micrologger 2 and digital optical televiewer probe. The results of the natural gamma, 3-arm caliper, heat pulse flowmeter and optical televiewer loggings are presented in Reference 2.5.4-249. The heat pulse flowmeter was performed in Borings RB-C8 and TB-C5. Natural gamma, 3-arm caliper, and optical televiewer logging was performed in the following boreholes:

CB-C2	CB-C3	CB-C4
CB-C5	FWS/ACB-C1	HM-E1
RB-C1	RB-C2	RB-C3
RB-C4	RB-C5	RB-C7
RB-C8	RB-C9	RB-C10
RB-C11	RB-C12	RW-C1
RW-C3	TB-C5	

The natural gamma log is a passive instrument that measures the amount of naturally occurring radioactivity from geologic units within the borehole. The natural gamma log is an excellent lithologic indicator because fine-grained clays and shales contain a higher radioelement concentration than limestones or sands. Gamma ray values are often used to assess the percentage of clay materials (indurated or non-indurated) that are present within a formation by using empirically derived equations and line information. The natural radioactivity trend for earth materials is as follows:

Shales
Clays
Marls
Siltstone
Sandstone
Limestone
Dolomite

Radioactivity
Increases

The caliper log measures variations in borehole size. The typical caliper response in a fractured, weathered, or karstic unit is a relatively abrupt increase in borehole size.

The OTV probe combines the axial view of a downward looking digital imaging system with a precision ground hyperbolic mirror to obtain an undistorted 360° view of the borehole wall. The probe records one 360° line of pixels at 0.9 mm (0.003 ft) depth intervals. The sample circle can be divided into 720 or 360 radial samples to give 0.5° or 1° radial resolution. For this investigation, the highest radial resolution (0.5°) was used. The line of pixels is aligned with respect to True North and digitally stacked to construct a complete, undistorted, and oriented image of borehole walls. The data are 24-bit true color and may be used for lithologic determination as part of interpretation. Since the acquired image is digitized and properly oriented with respect to borehole deviation and tool rotation, it allows data processing to provide accurate strike and dip information of structural features.

The heat-pulse flowmeter is used to measure low groundwater flow rates which may lie below threshold limits of conventional impeller tools. The heat-pulse flowmeter probe contains a horizontal wire-grid heating element and thermistors located above and below it. Apertures in the tool

permit the free flow of fluid through the assembly. Pulses of electric current are applied to the heating grid under surface command, warming fluid in the vicinity of the grid. The warm fluid front migrates towards thermistors where it is detected. Depending on the direction of the flow, either upper or lower thermistor detects the warm fluid front first. The time taken to reach the detector gives an indication of the flowrate.

2.5.4.4.3 Borehole Deviation Survey

A borehole deviation survey was performed. A maximum deviation of less than 1.5 degree was recorded in any of the borings surveyed. Borehole deviation surveys were performed in the following borings:

CB-C2	CB-C3	CB-C4
CB-C5	FO-E1	FWS/ACB-C1
RB-C1	RB-C2	RB-C3
RB-C4	RB-C5	RB-C6
RB-C7	RB-C8	RB-C9
RB-C10	RB-C11	RB-C12
RW-C3	RW-C4	TB-C5
TB/ESC-E3		

The deviation survey was performed in steel cased boreholes using the equipment EZ-Trac (EZ-BQ no. 5020) from REFLEX Corporation except in Borings RB-C8 and TB-C5. In Borings RB-C8 and TB-C5, the deviation survey was performed concurrently with the optical televiewer probe using the Robertson Geologging Micrologger 2 in the uncased borehole.

The EZ-Trac tool has a capacity for single shot, multi-shot, and hole orientation applications. The application of the tool used at Fermi 3 was the multi-shot function with particular attention to dip (angle of inclination) readings. The tolerance specifications of the dip measurements are +/- 0.25 degrees. The deviation survey was performed using the EZ-Trac tool inside the drilled stem to prevent borehole wall collapse. The dip is provided by three MEMS (micro-electro-mechanical systems) capacitive accelerometers aligned in orthogonal directions. By measuring the electrical capacitance with respect to acceleration a translation is made into a calibrated voltage output. This output is then translated into a dip function.

2.5.4.5 Excavations and Backfill

The Fermi 3 excavation system combines elements to provide side slope stability and those that limit or exclude groundwater entry. The excavation support and seepage control system could include various options such as a vertical reinforced concrete diaphragm wall extended into bedrock, ground freezing with vertical excavation faces, or sloped cut excavation with a sheetpile groundwater cutoff embedded into the glacial till combined with a grout curtain. During design, the specific excavation system type and configuration will be determined to develop an acceptable excavation approach that achieves groundwater control and excavation stability.

Subsurface conditions are addressed in Subsection 2.5.4.1. Details of bedrock units present at the Fermi site are provided in Subection Basic Geology and Seismic Information with engineering properties discussed in Subsection 2.5.4.2. Details of engineering granular backfill requirements at the site are provided in Subsection 2.5.4.5.4.2.

Details of bedrock units present at the Fermi 3 site are provided in Subection Basic Geology and Seismic Information with engineering properties discussed in Subsection 2.5.4.2. Details of engineered granular backfill requirements at the site are provided in Subsection 2.5.4.5.4.2.

The finished Fermi 3 site surface grade elevation is approximately elevation 179.6 m (589.3 ft) NAVD 88. Foundation elevations range from 177.3 m (581.6 ft) to 159.6 m (523.7 ft) NAVD 88. A list of the major structures in the power block area and their foundation levels are presented in Table 2.5.4-224. All excavation activities for the power block structures will commence from the existing ground surface elevation of approximately 177.1 m (581.0 ft) NAVD 88.

Excavated soil and/or rock may be used to fill some open water areas and to fill areas associated with temporary parking and construction laydown. Excavated material that meets requirements for use as engineered backfill will be segregated.

2.5.4.5.1 Source and Quantities of Backfill and Borrow Materials

The Fermi 3 project excavation generates approximately 313,468 m^3 (410,000 cubic yards) of excavated material. Excavated material that meets gradation requirements may be used as engineered granular

backfill as defined in Subsection 2.5.4.5.4.2. Backfill surrounding Seismic Category I or II structures is well-graded engineered granular material and fill concrete. Backfill underneath the FWSC and Seismic Category II structures from the base of the foundation to the top of the bedrock is fill concrete. The anticipated extent of engineered granular backfill and fill concrete is shown on the foundation cross-sections on Figure 2.5.4-202 through Figure 2.5.4-204.

Completing the Fermi 3 excavation using vertical sidewall excavation in soils and bedrock results in a total estimated cut volume (in-place volume) of about 313,000 m^3 (410,000 cubic yards), of which 256,000 m^3 (335,000 cubic yards) is soil excavation and 57,000 m^3 (75,000 cubic yards) is bedrock excavation. The total estimated backfill volume for full site development (in-place volume) is 344,000 m^3 (450,000 cubic yards). The volume of granular backfill from on-site excavation for Fermi 3 is estimated to be 180,000 m^3 (235,000 cubic yards). The quantity of the engineered granular backfill within the perimeter of the reinforced concrete diaphragm wall shown on Figure 2.5.4-201 is approximately 153,000 m^3 (200,000 cubic yards); therefore, the potential total on-site source of granular material is greater than the quantity required to backfill within the perimeter of the reinforced concrete diaphragm wall. As a result, the source used for backfill adjacent to the Seismic Category I structures will be from an on-site source. The on-site backfill source is investigated by borings, test pit, and laboratory and field testing, and the properties are discussed in Subsection 2.5.4.2.

Bulking and shrinkage factors have not been applied to the estimated excavation and backfill material volumes. Bulking and shrinkage factors are applied during the final design phase and are determined by specific material testing.

The excavated fill and bedrock may be processed to meet the required grading in accordance with Subsection 2.5.4.5.4. As an alternative or supplement to the onsite crushed rock, dense-graded aggregate from an off-site source may be used as engineered granular backfill material. Dense graded aggregate such as Size 21A or 21AA as specified by the Michigan Department of Transportation (Reference 2.5.4-233) is suitable material. These types of materials are available from local and regional quarry sources.

For any imported source material, the material(s) are sampled and tested to verify adherence to the required specifications for engineered granular

backfill. Laboratory tests including moisture content per ASTM D2216 (Reference 2.5.4-213), sieve analysis per ASTM D422, (Reference 2.5.4-216), standard Proctor per ASTM D698 (Reference 2.5.4-234), modified Proctor tests per ASTM D1557 (Reference 2.5.4-235), Relative Density test per ASTM D 4253 and 4254 (Reference 2.5.4-236, Reference 2.5.4-237) and Direct Shear Test per ASTM D3080 (Reference 2.5.4-223) are performed to verify design requirement compliance for engineered granular backfill. The soundness of aggregate is confirmed using sulfate soundness per ASTM C88 (Reference 2.5.4-238) and Los Angeles abrasion tests per ASTM C131 and ASTM C535 (Reference 2.5.4-239, Reference 2.5.4-240).

Testing for chemical and static properties are performed on all proposed engineering backfill material(s).

2.5.4.5.2 **Extent of Excavations, Fills and Slopes**

Vertical excavation faces within soil and bedrock could be achieved by using an excavation system consisting of a vertical cut-off, such as reinforced concrete diaphragm wall system around the entire excavation. In such a case, overburden soils would be excavated from ground surface to the estimated top of the bedrock surface at elevation 168.2 m (552 ft) NAVD 88. Bedrock would be excavated to reach the required foundation design elevations as shown in Table 2.5.4-224.

If the vertical cut-off wall were utilized, it would likely be installed from existing ground surface at elevation 177.1 m (581.0 ft) NAVD 88 to a depth determined during design to control seepage into the excavation, followed by excavation to the required foundation design elevations inside the cut-off walls through soil and bedrock. For this discussion, the Fermi 3 cut-off walls are assumed to be approximately 24.4 m (80 ft) deep with an embedment depth of approximately 15.2 m (50 ft) into bedrock, between elevations 168.2 and 153.5 m (552.0 and 503.7 ft) NAVD 88. Soil nails or rock bolts may be used to provide additional lateral support, as necessary, based upon analysis during the detailed design phase if such an excavation system were used.

The reinforced concrete diaphragm wall will act as the perimeter of the soil excavation and will provide vertical support for the portion of the excavation within the soil. Structural design of the concrete diaphragm wall will be in accordance with ACI 318. The reinforced concrete

diaphragm wall will be reinforced to resist lateral forces applied by the soils.

A plan view of the excavation for Fermi 3 using the vertical cut-off wall option in soil and bedrock is shown on Figure 2.5.4-201. Cross-sections of the excavation plan are shown on Figure 2.5.4-202, Figure 2.5.4-203, and Figure 2.5.4-204. These figures are intended to indicate the presence of the wall, but are not intended to establish distances between the wall and Seismic Category I structures. Considerations that will be taken into account regarding the distance between the wall and the Seismic Category I structures include the following:

- During design, the deflection of the concrete diaphragm wall will be estimated. The wall will be aligned to prevent the deflected wall from encroaching on the limits of Seismic Category I structures plus any construction limits.
- The wall will be aligned to allow sufficient space for placement of backfill outside the Seismic Category I structures.
- The wall will be aligned to allow sufficient space for performing inspections of the outside of the structures, as required, during construction.
- The distance from the Seismic Category I structures to the diaphragm wall will be established to provide sufficient space to facilitate erection of structures. Considerations for construction would include providing sufficient space for personnel and equipment.

Seismic Category I structures are designed to resist all static and dynamic soil and bedrock loads assuming the concrete diaphragm wall is not present. There are no impacts to the completed Seismic Category I structures due to the presence of the concrete diaphragm wall as the wall will not impact the foundation input response spectra (FIRS) and the diaphragm wall will be supported on both sides when structures are completed, as backfill will be placed in the gap between the structure and the wall. Therefore, presence of the diaphragm wall will not adversely impact the Seismic Category I structures.

2.5.4.5.3 Excavation Methods and Stability

2.5.4.5.3.1 Excavation in Soil

Conventional excavation methods (e.g. backhoe, front end loader, and dump truck) could be utilized to remove soil layers to the lines and grades

shown on Figure 2.5.4-201 through Figure 2.5.4-204 using reinforced concrete diaphragm wall option for the excavation support and seepage control system.

Slope Stability at Fermi 3 is discussed in Subection Stability of Slopes. During the project detailed design stage, stability analyses are conducted, as needed, to show that the excavated temporary slopes have an adequate factor of safety including the effect of surcharge loading from construction equipment and the effect of groundwater seepage control.

2.5.4.5.3.2 **Excavation in Bedrock**

Excavation of bedrock at Fermi 3 may be completed using blasting, mechanical excavation, or a combination of blasting and mechanical excavation. The bedrock stratum is excavated to the lines and grades shown on Figure 2.5.4-201 through Figure 2.5.4-204.

Any blasts would be designed by a qualified blasting professional and a vibration control specialist to ensure protection of all existing adjacent structures including Fermi 2 structures and utilities, and Fermi 3 components associated with the excavation support and seepage control system. Potential effects and mitigation activities from use of explosives on Fermi 2 are discussed in Section 1.12.

Controlled blasting techniques, including cushion blasting, pre-splitting and line drilling may be used, with dimensioned bench heights as required. Blasting would be designed and strictly controlled to preserve the integrity of exterior bedrock, to prevent damage to existing structures, equipment, and freshly placed concrete, and to prevent disruption of Fermi 2 operations. Peak particle velocity would be measured and kept within specified limits that are a function of the distance from blast and amount of explosives used.

Mechanical excavation may include the use of roadheaders, terrain levelers, rockwheels, rock trenchers, and other mechanical excavation techniques. The bedrock may be reinforced and supported to ensure adequate safety and stability. Rock excavation support (e.g. rock bolts, welded wire fabric, or similar reinforcement) may be used as needed to provide support of temporary rock faces during construction below El. 168.2 m (552 ft). Appropriate temporary rock face-support measures would be utilized.

During construction, excavated subgrades in bedrock for safety-related structures are mapped and photographed by qualified and experienced geologists. Geotechnical instrumentation such as extensometers, inclinometers, and other instrumentation, as required, are installed to monitor bedrock movements. Unforeseen geologic features are evaluated.

2.5.4.5.3.3 Foundation Bedrock Grouting

A foundation bedrock grouting program was completed for the Fermi 2 excavation and was successful in reducing groundwater flow through the rock mass into the excavation during construction (Reference 2.5.4-241). A similar approach to the foundation bedrock grouting program used for Fermi 2 may be used for Fermi 3 as part of the excavation support and seepage control system.

2.5.4.5.4 Compaction Specifications and Quality Control

This section describes the methods and procedures used for verification and quality control of foundation materials.

2.5.4.5.4.1 Foundation Bedrock

Properties of foundation materials are discussed in Subsection 2.5.4.2. This section describes methods and procedures used for verification and quality control of foundation materials.

Visual inspection of the final bedrock excavation surface is performed to confirm material is in general conformance with the expected foundation materials based on boring logs. Visual inspection is performed of exposed bedrock foundation subgrade to confirm that cleaning and surface preparations are properly completed. Fill concrete may be used to create a level, uniform surface for installation of concrete foundation slab.

Geologic mapping of the final exposed excavated bedrock surface is performed before placement of fill concrete and foundation concrete. The geologic mapping program includes photographic documentation of the exposed surface and documentation for significant geologic features.

The details of the quality control and quality assurance programs for foundation bedrock are addressed in the design specifications prepared during the detailed design phase of the project.

2.5.4.5.4.2 Backfill Materials and Quality Control

Backfill for Fermi 3 may consist of fill concrete or a sound, well graded engineered granular backfill. Subsection 3.7.2 presents the results of the Fermi 3 site-specific soil-structure interaction (SSI) analyses for the RB/FB and CB with fill concrete included as the backfill below the top of the Bass Islands Group bedrock, and engineered granular backfill above the top of the bedrock neglected. The Fermi 3 site-specific SSI results show that, with the engineered granular backfill neglected, the RB/FB and CB are within the Referenced DCD structural design and stable against sliding and overturning, as discussed in Subsection 3.8.5. The soil-structure interaction analyses in the Referenced DCD for the FWSC were performed as a surface structure, so the backfill surrounding the FWSC foundation basemat is not included in the Referenced DCD SSI. The Referenced DCD sliding analysis considers the backfill supporting and surrounding the basemat. For the FWSC, the supporting material below the FWSC at Fermi 3 is fill concrete with a mean compressive strength of 31 MPa (4,500 psi) versus the soil with an angle of internal friction of 35 degrees used in the Referenced DCD. As discussed in Subsection 3.8.5, sliding of the FWSC is not an issue when neglecting the engineered granular backfill surrounding the basemat. Therefore, the engineered granular backfill surrounding the basemat for the FWSC is not Seismic Category I backfill.

The Fermi 3 engineered granular backfill surrounding the Seismic Category I structures will meet the following Referenced DCD requirements:

i. Product of peak ground acceleration α (in g), Poisson's ratio ν and density γ

$\alpha(0.95\nu + 0.65)\gamma$: 1220 kg/m^3 (76 lbf/ft^3) maximum

ii. An angle of internal friction equal to or greater than 35 degrees when properly placed and compacted.

iii. Soil density

γ: 2000 kg/m^3 (125 lbf/ft^3) minimum

The DCD requirement $\alpha(0.95\nu + 0.65)\gamma$ is retained because it is associated with the dynamic lateral earth pressure of the engineered granular backfill on the embedded walls of Seismic Category I RB/FB and CB. The DCD requirements for angle of internal friction and density are retained to ensure a dense backfill.

The anticipated extent of fill concrete and engineered granular backfill is shown on Figure 2.5.4-202, Figure 2.5.4-203, and Figure 2.5.4-204.

Fill concrete mix designs are addressed in a design specification prepared during the detailed design phase of the project. Field observation is performed to verify that approved mixes are used and test specimens are obtained that verify that specified design parameters are reached. The foundation bedrock and fill concrete provide adequately high factors of safety against bearing capacity failure under both static and seismic structural loading. Quality Control testing requirements for bedrock include visual inspection and geologic mapping.

Engineered granular backfill sources are identified and tested for engineering properties, in accordance with recommendations from Subsection 2.5.4.5.1 and other testing as required by design specifications. The compaction effort required for the engineered granular backfill surrounding the Seismic Category I structures above the top of the Bass Islands Group bedrock will be a mean of 95 percent of the modified Proctor density or a mean of 75 percent of the maximum relative density. During detailed design, the laboratory testing in Subsection 2.5.4.5.1 is implemented to establish the required density to meet design requirements of the engineered granular backfill adjacent to Category I structures. To further confirm the density selected based on the laboratory testing results meets the design requirements, a program will be implemented to test the in-place engineered granular backfill, which could consist of construction of a test pad(s). Also during detailed design, a testing program will be implemented to confirm the engineered granular backfill placed during construction meets the design requirements. For liquefaction, the program could consist of performing standard penetration tests to confirm the fill has the minimum N_{60} in Subsection 2.5.4.8.

Engineered granular backfill is compacted to achieve a density that results in the backfill having a minimum ϕ' of 35 degrees. Based on correlations of strength characteristics for granular soils (Reference 2.5.4-242), the ϕ' of compacted granular soils can achieve 35 degrees. Engineered granular backfill materials are placed in controlled lifts and compacted. Within confined areas or close to foundation walls, smaller compactors are used to prevent excessive lateral pressures against the walls from stress caused by heavy compactors.

Evaluation and discussion of liquefaction issues related to soil backfill materials is provided in Subsection 2.5.4.8. Lateral pressures applied against foundation walls are evaluated and discussed in Subsection 2.5.4.10.

The gradation of the engineered granular backfill will be selected to approximate a hydraulic conductivity of 8.85 x 10^{-4} m/s (251 ft/day) (Subsection 2.4.12.2.4) or greater.

A quality control sampling and testing program is developed to verify that fill concrete and engineered granular backfill material properties conform to the specified design parameters. Sufficient laboratory compaction and grain size distribution tests are performed to account for variations in fill material. A test fill program may be included for the purposes of determining an optimum size of compaction equipment, number of passes, lift thickness, and other relevant data for achievement of the specified compaction.

Fill concrete used as fill under the FWSC, Seismic Category II structures, and surrounding the RB/FB and CB to the top of bedrock will be proportioned, tested and the placement controlled in accordance with Regulatory Guide 1.142. Additionally, ACI 349 requirements for concrete exposed to sulfate-containing solutions will be implemented. The fill concrete will have a mean 28-day compressive strength of equal to, or greater than, 31 MPa (4,500 psi) with a mean shear wave velocity of equal to, or greater than, 1,100 m/s (3600 ft/s). Compressive strength of the fill concrete will be tested in accordance with Regulatory Guide 1.142. The compressive strength of the fill concrete will be used to calculate shear wave velocity to ensure that the shear wave velocity of 1,100 m/s (3600 ft/s) is met. The mix design developed for the fill concrete will control erosion and leaching due to contact with site groundwater and limit settlement to specified tolerances (Table 2.0-201), including creep and shrinkage.

The quality control program for fill concrete includes requirements for compressive strength testing. Verification will be performed to confirm that compressive strength testing results comply with mix design, minimum strengths, and placement requirements. The details of the quality control program will be addressed in a design specification prepared during the detailed design phase of the project.

Aggregate for concrete for Fermi 3 Seismic Category I and RTNSS structures will be tested for deleterious expansive alkali-silica reaction in accordance with ASTM C1260 (Reference 2.5.4-253) and ASTM C1293 (Reference 2.5.4-254).

The quality control program for engineered granular backfill includes requirements for field in place density tests and index tests to confirm material classification and compaction characteristics are within the compliance range of materials specified and used for design. Granular backfill placement and compaction methods will be addressed in design specifications prepared in the detailed design stage of the project.

Test methods for index and static engineering properties of the backfill surrounding Seismic Category I structures are provided in Subsection 2.5.4.5.1.

The test methods, frequency, and location of testing on the backfill surrounding Seismic Category I structures are as follows:

- Direct Shear Test (ASTM D 3080) - Minimum of 3 tests per material type per borrow source.
- Maximum and Minimum Relative Density Test (ASTM D 4253, ASTM D 4254) [Minimum of 3 tests per material type per borrow source] or Modified Proctor Test (ASTM D 1557) [Minimum of 3 tests per material type per borrow source].
- In-Place Density Tests (ASTM D 6938) - test frequency and location determined during detailed design and provided in construction specifications.

Thermal cracking control of fill concrete adjacent to and underneath Seismic Category I and II structures will be addressed during detailed design by implementing ACI 207.1R, 207.2R, and 207.4R measures that address mass concrete, as follows:

- Implementing the following fill concrete mix design measures:
  - o Optimizing the amount of cement paste in the mix by using larger aggregate blends, and admixtures to improve placeability and rate of hydration.
  - o Using Type V cement and including pozzolans, such as fly ash, to reduce the heat of hydration of the cement component.
- Implementing the following fill concrete design measures:

  - o Evaluate the need for reinforcing for thermal cracking control.
  - o If reinforcing is needed for thermal cracking control, select the amount and configuration of reinforcing required.
- Implementing the following controls during placement of fill concrete:
  - o Controlling placement size and arrangement to limit heat gradients within the fill concrete.
  - o Providing a cool mix to limit heat of hydration and thermal gradients by using cold water and/or ice, and shading aggregates and sand.
  - o Using curing methods to facilitate heat reduction and possible use of insulation to reduce thermal differences in the concrete mass.

The details of the quality control and quality assurance programs for fill concrete and engineered granular backfill are addressed in the specifications prepared during the detailed design phase of the project.

2.5.4.5.5 Control of Groundwater during Excavation

Control of groundwater and dewatering during excavation is presented in Subsection 2.5.4.6.2.

2.5.4.5.6 Geotechnical Instrumentation

The Fermi 3 excavation support and seepage control system will be continually monitored during excavation activities for movement and/or deflection. Real time data acquisition techniques may be used for collection and graphical representation of the data. An instrumentation and monitoring program developed during the project detailed design phase may include inclinometers, piezometers, seismographs, survey points, and construction inspection documentation.

Rebound or heave, less than 12.7 mm (0.5 inch), as presented in Subsection 2.5.4.10, is expected from foundation excavation; therefore heave monitoring is not needed.

As discussed in Section Subsection 2.5.4.10.2, settlement is predicted to be well within the design limits in the ESBWR DCD. Settlement is expected to occur during the construction phases of the project instead of during post construction because the Seismic Category I structures are founded on bedrock, which will compress elastically as the loads are

applied. To confirm the settlement predictions, the following monitoring plan will be implemented.

- Benchmarks will be established at the corners of selected Seismic Category I structures as the foundation mats are constructed. These will be monitored before and periodically during construction of the basemats and sidewalls prior to placement of the backfill materials.
- Additional bench marks will be installed approximately 1 meter (3 feet) above site grade and connected to the sidewalls directly above the deeper bench marks locations described previously. These bench marks will be monitored during backfilling operations and, periodically, during and after construction.

Monitoring will be continued until at least 90% of expected settlement has occurred or the rate of settlement has virtually stopped. This will be evaluated by review of the settlement versus time curves at the bench mark locations. Post construction settlement monitoring would be included as part of the Maintenance Rule program.

2.5.4.6 Groundwater Conditions

This section includes information on the groundwater conditions at the site relative to foundation stability for the safety-related structures.

2.5.4.6.1 Groundwater Measurements

The field investigation program for groundwater measurements is presented in Subsection 2.5.4.2.2. The data from monitoring wells and piezometers are presented and discussed in Subsection 2.4.12.

2.5.4.6.2 Construction Dewatering and Impact of Dewatering

A excavation support and seepage control system system around the perimeter of the Fermi 3 excavation will control groundwater seepage through soils and bedrock. During excavation, localized sump pumping systems within the excavation may be used to supplement water control, as necessary. The sump pumping system would consist of pumps being placed at low points, with water pumped to a location outside of the excavation. Foundation bedrock grouting may be performed at the base of the Fermi 3 excavation to aid in controlling groundwater seepage into the excavation.

Following installation of the excavation support and seepage control system, a series of pump tests could be performed to evaluate the

effectiveness of the system in controlling groundwater flow towards the excavation. Observation wells to monitor the groundwater levels outside the reinforced concrete diaphragm wall and within the excavation footprint would be installed as required. The location and details of any pump tests will be determined during the detailed design phase of the project.

The pump test results would be used to evaluate the need for bedrock grouting prior to excavation. Otherwise, localized foundation bedrock grouting, as necessary, may be performed to control groundwater inflow from zones of high permeability within the rock mass during excavation. The thickness of the grouted zone will be based on the need to minimize inflow into the excavation and to resist any uplift pressures at the base of the excavations. The design of the foundation bedrock grouting program will be completed during the detailed design phase of the project.

The groundwater control measures maintain the groundwater at an elevation below the base of the excavation that precludes degradation of the foundation materials during foundation construction, and allows for proper placement and compaction of engineered granular backfill materials.

2.5.4.6.3 **Seepage during Construction**

The impact of seepage into the excavation and groundwater control measures during construction upon the existing groundwater conditions is discussed in Subsection 2.4.12.2.5. No potential exists for piping due to seepage in bedrock. The seepage into the excavation will be minimized by the excavation support and seepage control system.

All Fermi 2 Seismic Category I structures are founded on bedrock (Reference 2.5.4-241) and therefore the potential for settlement associated with Fermi 3 dewatering operations is negligible. During the project detailed design stage for Fermi 3, a monitoring program will be developed to assess groundwater levels and settlement at existing Fermi 2 structures. **[START COM 2.5.4-001]** A Contingency Plan will be developed for mitigation of any settlement prior to the start of Fermi 3 construction. **[END COM 2.5.4-001]**

2.5.4.6.4 **Permeability Testing**

Packer and slug testing, and laboratory hydraulic conductivity testing were performed to estimate the hydraulic conductivity of bedrock and

soil. The results of testing are presented in the detail in Subsections 2.4.12.

2.5.4.6.5 Impact of Groundwater Conditions on Foundation Stability

Seismic Category I structures will be founded on bedrock or fill concrete. Other major structures in the power block area will be founded either on bedrock or structural fill. During detailed design, the foundation stability of all Fermi 3 structures founded on either bedrock, concrete, or engineered granular backfill will be designed to account for the following:

- Short term construction conditions in dry or moist ground with a lowered groundwater elevation.
- Long term operational in-service condition of saturated or partially saturated ground with a rebounded natural groundwater elevation.

2.5.4.7 Response of Soil and Rock to Dynamic Loadings

This section presents the response of soil and bedrock to dynamic loading and the effect of past earthquakes.

2.5.4.7.1 Effect of Past Earthquakes

The historical earthquake events and their effects are discussed in Subsection 2.5.1.1.4.3. No reports or studies exist on liquefaction and paleoliquefaction in the (40 km [25 mi] radius) site vicinity as presented in Subsection 2.5.1.2.6.6.

2.5.4.7.2 Seismic Wave Velocity Profiles

The geophysical surveys used for dynamic characterization of soil and bedrock are: 1) P-S Suspension logger, 2) Downhole Seismic procedure, and 3) SASW. Detailed discussions of the results from geophysical surveys are presented in Subsection 2.5.4.4. Figure 2.5.4-220 through Figure 2.5.4-223 present V_s and V_p profiles measured in bedrock units. The V_s and V_p profiles in overburden from P-S suspension logging are shown on Figure 2.5.4-224. The V_s profiles of overburden from SASW survey are presented on Figure 2.5.4-225. The variability of seismic wave velocities is present in Subsection 2.5.4.2.2. The average values of seismic wave velocities are summarized in Table 2.5.4-202.

To consider variation and uncertainties in dynamic soil properties, a suite of 60 randomized soil profiles were generated for soil amplification analyses as discussed in Subsection 2.5.2.5.1.3. Soil amplification

analyses were performed for the RB/FB, CB and FWSC soil profiles and the response motions at the foundation level were obtained for the 10^{-4} and 10^{-5} input ground motions. At each of the loading levels for the input ground motions used in the SHAKE analyses, the iterated shear wave velocities for each layer of the 60 randomized profiles were sorted into rank order (from the lowest to highest value), and the 16^{th,} 50th and 84th percentiles shear wave velocity profiles at seismic strains were determined. The 16th percentiles of the randomized shear wave velocities at seismic strains represent mean minus one standard deviation (the lower bound soil properties) specified by the Reference DCD. The 16th percentiles of the randomized shear wave velocities at seismic strains for foundation materials below the RB/FB, CB and FWSC are greater than 300 m/s (1,000 fps), as required by the Reference DCD.

2.5.4.7.3 Dynamic Laboratory Testing

The laboratory testing program for dynamic properties is discussed in Subsection 2.5.4.2.3. No dynamic laboratory testing was performed in bedrock units. Some dynamic laboratory tests were performed on undisturbed glacial till samples; however, these results are not required for Seismic Category I structures, as these are all supported directly on bedrock, or on fill concrete extending to the bedrock.

Four RCTS tests were performed on glacial till after evaluating sample disturbance and quality by reviewing the results of X-ray radiography and one-dimensional consolidation tests for evaluating sample disturbance and quality. The RCTS tests were performed on undisturbed samples obtained using thin-wall tubes. Prior to the RCTS testing, the thin-wall tubes of all samples to be tested were subjected to X-ray radiography to evaluate the level of sample disturbance. Subsequently, good quality sample intervals were identified and selected for RCTS and one-dimensional consolidation testing. One-dimensional consolidation tests were first performed prior to RCTS testing for sample quality evaluation using the Specimen Quality Designation (SQD) (Reference 2.5.4-251). RCTS tests were then performed for samples with acceptable SQD (SQD of “A” or “B”), indicating relatively undisturbed samples.

2.5.4.7.4 Shear Modulus Reduction and Damping Curves for Rocks

Shear modulus reduction and damping curves for bedrock are discussed in Subsection 2.5.2.5.

2.5.4.7.5 Shear Modulus Reduction and Damping for Soils

No Seismic Category I structures are founded on soil. The RB/FB and CB are founded on bedrock. The FWSC is founded on fill concrete extending to bedrock. The fill and lacustrine deposits are removed under all foundations in the power block area; therefore, no shear modulus and damping curves are presented for these materials.

The modulus reduction and damping curves for glacial till are needed for developing the GMRS. The shear modulus and damping curves for glacial till are chosen from published correlations (Reference 2.5.4-229). As shown in Table 2.5.4-204, the plasticity index of glacial till ranged from 7 to 27 percent with a mean value of 14 percent. The shear modulus reduction and damping curves with plasticity index equal to 15 and 50 were selected for glacial till as discussed in Subsection 2.5.2.5.1.2. The modulus reduction and damping curves were then randomized as shown on Figure 2.5.2-259 and Figure 2.5.2-260 as discussed in Subsection 2.5.2.5.1.3.

Measured shear modulus reduction and damping data from RCTS testing and published curves for a range of plasticity index values are plotted for comparison on Figure 2.5.4-226. The measured modulus reduction and damping curves from the RCTS tests are well within the randomized plasticity index 15 and 50 curves as shown on Figure 2.5.2-259 and Figure 2.5.2-260.

2.5.4.7.6 Shear Modulus Reduction and Damping Curves for Granular Backfill and Fill Concrete

Engineered granular backfill is not used to support any Seismic Category I structures. Engineered granular backfill is mainly used as backfill surrounding the embedded walls of structures or to backfill beneath other structures with foundation levels above bedrock, except Seismic Category II structures, which are founded on fill concrete.

The shear modulus and damping curves for engineered granular backfill are discussed in Subsection 3.7.1.1.4.1.1.

Shear modulus reduction and damping curves for assumed lean concrete fill using a lower bound compressive strength of 2.1 MPa (300 psi) used for site response analysis are discussed in Subsection 2.5.2.5.

2.5.4.7.7 Ground Motion Response Spectra

The seismic velocity profiles are shown on Figure 2.5.4-220 through Figure 2.5.4-225. The GMRS and FIRS based on these velocity profiles and modulus reduction and damping curves are described in Subsection 2.5.2.6.

2.5.4.8 Liquefaction Potential

This section conforms to guidelines in RG 1.198.

All Seismic Category I structures are supported within the Bass Islands dolomite or on fill concrete extending to the top of bedrock. Neither the bedrock nor fill concrete are susceptible to liquefaction.

For engineered granular backfill adjacent to Seismic Category I structures, liquefaction considerations only apply below the groundwater table. Subsection 2.4.12.5 provides the maximum historical high groundwater level elevation of 175.6 m (576.11 ft) NAVD 88, which is approximately 4 m (13.2 ft) below the plant grade elevation of 179.6 m (589.3 ft) NAVD 88; therefore, liquefaction is not a consideration in the upper 4 m (13.2 ft) of the engineered granular backfill. Subsection 2.5.4.5.4.2 discusses placement of granular backfill adjacent to Seismic Category I structures in controlled lifts with compaction. This will result in a dense to very dense consistency engineered backfill surrounding the embedded walls of Seismic Category I structures; therefore, there is also no potential for liquefaction in the engineered granular backfill below the groundwater. For confirmation, a liquefaction analysis based on the SPT is provided to demonstrate that the engineered granular backfill is not susceptible to liquefaction.

Reference 2.5.4-251, Table 12.1 shows that for dense granular soils N_{60} is between 30 and 50 blows/foot, and for very dense granular soils N_{60} is greater than 50 blows/foot. N_{60} is the numbers of blows to drive a standard split barrel sampler the last 12 inches of the SPT using a 140 pound hammer falling 30 inches, where the hammer has a 60 percent energy efficiency. To evaluate liquefaction potential of soil, $(N_1)_{60}$ is needed, where $(N_1)_{60}$ is the N_{60} value normalized to an overburden pressure of approximately 100 kPa (1 ton per square foot) (Reference 2.5.4-252). Reference 2.5.4-252 shows that for historical data, no liquefaction was observed when $(N_1)_{60}$ is greater than 30 blows/ft.

For the engineered granular backfill, the N_{60}-value is estimated to be 30 blows/foot at the ground surface, and is increased linearly to 60

blows/foot at a depth of 65 feet. Using this distribution for N_{60} and a bounding groundwater level at 2 feet below finished ground level grade, at all engineered granular backfill depths for the full depth of the deepest Seismic Category I structure, $(N_1)_{60}$ is greater than 30 blows/foot. With the backfill placement approach and resultant $(N_1)_{60}$ greater than 30, it is concluded that the engineered granular backfill, adjacent to all Seismic Category I structures, is not susceptible to liquefaction. If $(N_1)_{60}$ of the in-place engineered granular backfill is less than 30, a more refined liquefaction analysis will be performed to confirm there is adequate resistance against liquefaction.

The existing fill, lacustrine deposits and glacial till are removed under and adjacent to all Seismic Category I structures; therefore, liquefaction analysis for these soils is not necessary.

Glacial till and/or engineered granular backfill will be used as foundation support under non-Category I structures that could strike a Seismic Category I structure if it were to fail during a seismic event. Glacial till is not susceptible to liquefaction based on its USCS classification as lean clay (CL) and fines content greater than 30 percent (Table 2.5.4-202). As described above, engineered granular backfill is not susceptible to liquefaction.

2.5.4.9 **Earthquake Design Basis**

The V_s values of soils and bedrock at the site were determined through the field exploration program using geophysical testing as described in Subsection 2.5.4.2 and Subsection 2.5.4.4. Subsection 2.5.4.7 presents the dynamic response of soil and bedrock under dynamic loading conditions. The top of generic bedrock is approximately 129.5 m (425 ft) below the existing ground surface where the V_s of bedrock (Salina Group Unit B) is greater than 2804 m/s (9200 fps). A site response analysis was performed using the above information to develop the GMRS for the site as described in Subsection 2.5.2.6.

2.5.4.10 **Static Stability**

In this section, the analyses performed to evaluate the stability of the safety-related structures under static loading conditions are presented. Specifically, this subsection addresses three Seismic Category I structures – RB/FB, CB and FWSC. This section includes analyses of foundation bearing capacity and settlement, excavation rebound, lateral earth pressures, and hydrostatic pressures.

DCD Figure 3G.1-6 and DCD Tables 2.0-1, 3.8-8, and 3.8-13 provide information on plan dimensions, embedment depths, and loads. The RB/FB mat foundation has plan dimensions of 49.0 by 70.0 m (161 by 230 ft), and bears 20.0 m (65.6 ft) below the Referenced DCD reference grade (4500 mm). As discussed in Subsection 2.5.4.5, the Referenced DCD reference grade is equivalent to a site elevation of 179.6 m (589.3 ft) NAVD 88. The base of the RB/FB foundation base is thus at elevation 159.6 m (523.7 ft) NAVD 88. The 4.0 m (13.1 ft) thick foundation is designed for soil pressures of 699 kPa (14,600 psf) (static) and 1,100 kPa (23,000 psf) (dynamic).

The CB mat foundation has plan dimensions of 23.8 by 30.3 m (78 by 99 ft) and bears 15.0 m (48.9 ft) below the final site elevation. The base of the CB foundation is thus at elevation 164.7 m (540.4 ft) NAVD 88. The 3.0 m (9.8 ft) thick CB mat is designed for allowable soil bearing pressures of 292 kPa (6,100 psf) (static) and 420 kPa (8,800 psf) (dynamic).

The FWSC mat foundation has plan dimensions of 20 by 52 m (65.6 by 171 ft) and is embedded 2.4 m (7.7 ft) below the final site elevation. The base of the FWSC foundation is thus at elevation 177.3 m (581.6 ft) NAVD 88. The 2.5 m (8.2 ft) thick FWSC mat is designed for allowable soil bearing pressures of 165 kPa (3,450 psf) (static) and 1,200 kPa (25,100 psf) (dynamic).

The stability of the RB/FB, CB, and FWSC foundations were evaluated for the various design conditions, which included Referenced DCD reference grade, maximum design groundwater elevation, and the total static dead plus live loads. Bearing capacity and foundation settlement potential were evaluated for the foundations using currently accepted methods and practices. Lateral earth pressures were calculated for the situation where compacted gravel backfill is placed against buried concrete walls (RB/FB and CB only). The lateral earth pressures were based on the at-rest lateral earth pressure condition.

Table 2.5.4-226 summarizes building sizes, depths, and loadings for buildings in the power block area. The information was used for stability analyses in the following sections.

2.5.4.10.1 Bearing Capacity

For bearing capacity analysis, it is assumed that the influence zone of the foundation level is taken to be one times the width of the foundation.

Therefore, the material properties important for the bearing capacity analysis are those of Bass Islands Group and Salina Group Unit F.

Table 2.5.4-208 shows the Mohr-Coulomb parameters, based on Hoek-Brown criterion. For the Bass Islands Group, the upper bound Hoek-Brown ϕ' of 53 degrees matches well with the mean residual friction angle of 52 degrees measured from rock direct shear tests on discontinuities (Table 2.5.4-206); therefore, ϕ' equal to 52 degrees is used for the Bass Islands formation. For the Salina Group Unit F the Hoek-Brown, the lower bound ϕ' of 28 degrees was used.

The bearing capacity was evaluated at each unit using the following two independent methods:

1. Ultimate Bearing Capacity using the Terzaghi approach based on strength of bedrock mass (Reference 2.5.4-243).
2. Allowable bearing pressure based on q_u of bedrock, based on Uniform Building Code (Reference 2.5.4-244).

For the FWSC, the ultimate bearing capacity using the Terzaghi approach (Method 1) is computed using Equation 2 shown below:

$$q_{ult} = cN_c + 0.5\gamma' BN_\gamma + \gamma' DN_q \qquad \text{[Eq. 2]}$$

where;

q_{ult} = ultimate bearing capacity
g' = effective unit weight of bedrock mass
B = width of foundation
D = depth of foundation below ground surface
c = cohesion intercept for the bedrock mass

The terms N_c, N_γ and N_q are bearing capacity factors given by the following equations:

$$N_c = 2N_\phi^{1/2}\left(N_\phi + 1\right) \qquad \text{[Eq. 3]}$$

$$N_\gamma = N_\phi^{1/2}\left(N_\phi^2 - 1\right) \qquad \text{[Eq. 4]}$$

$$N_q = N_\phi^2 \qquad \text{[Eq. 5]}$$

$$N_\phi = \tan^2\left(45 + \phi/2\right) \qquad \text{[Eq. 6]}$$

where:

ϕ = angle of internal friction for the bedrock mass.

However, in cases where the shear failure is likely to develop along planes of discontinuity or through highly fractured bedrock masses, cohesion is not relied upon to provide resistance to failure (Reference 2.5.4-243). As the bedrock contains fractures, this approach was used for evaluating the bearing capacity of the RB/FB and CB; therefore, the ultimate bearing capacity of the RB/FB and CB is computed using Equation 7 as follows:

$$q_{ult} = 0.5\gamma B N_\gamma + \gamma D N_q \qquad \text{[Eq. 7]}$$

All terms are as previously defined. The ultimate bearing capacity is estimated by using the foundation correction shape factor (Reference 2.5.4-243).

For large foundations that are founded at great depths below grade, these equations can give very large bearing capacity values, even when a factor of safety of 3 is included for allowable bearing value. In such situations, settlement considerations normally governs design.

The Uniform Building Code (Method 2) calculates the allowable bearing pressure on rock as 20 percent of $q_{u.}$

Table 2.5.4-227 shows the results of the bearing capacity analyses using methods 1 and 2. Both methods were used to check against the static bearing capacity requirement in the Referenced DCD. Using Terzaghi's approach, the allowable bearing capacity is estimated by dividing the ultimate bearing capacity by a factor of safety of 3. The allowable bearing capacity calculated based on both methods is greater than the maximum static bearing demand required in the Referenced DCD as shown in Table 2.5.4-227.

Method 1 was also used to check against the dynamic bearing capacity requirement. Using Terzaghi's approach, the calculated ultimate bearing capacity was divided by a factor of safety of 2.25 to obtain the allowable dynamic bearing pressure. The dynamic factor of safety is established by dividing the static factor of safety by 1.33. The allowable dynamic bearing pressure based on Terzaghi's approach is greater than the maximum dynamic bearing demand required in the Referenced DCD and the Fermi 3 site-specific SSI dynamic bearing demand as shown in Table 2.5.4-227.

2.5.4.10.2 Rebound Due to Excavation and Settlement Analysis

All Seismic Category I structures are founded on either bedrock or fill concrete overlying bedrock (Subsection 2.5.4.3); therefore, only linear

elastic deformation is considered for settlement analysis. The parameter of interest for linear elastic settlement in the bedrock is E, which is addressed herein.

The E values of bedrock units at the Fermi 3 site obtained by various methods are summarized in Table 2.5.4-228. The various methods used to determine the E of bedrock units are 1) stress-strain curve from laboratory unconfined compression tests, 2) wave equation obtained by solving 3-dimensional equations of motion (using mean V_s from P-S suspension), 3) empirical approach using the Hoek-Brown criterion, and 4) stress-strain curve from results of pressuremeter testing.

For the Bass Islands Group and Salina Group Unit F, the largest E is the average E obtained from laboratory tests, because the unconfined compression tests were performed on intact rock samples which do not take the fractured nature of the bedrock mass into consideration. The E calculated from average V_s is lower, because the average V_s is more representative of the bedrock mass. The ratio of the E, based on laboratory tests, to the E, based on average V_s, is approximately 1.6 for the Bass Islands Group (average RQD is 54 percent) and 4.0 for the Salina Group Unit F (average RQD is 13 percent). The E calculated from average V_s and laboratory tests are both greater than the upper bound E using the Hoek-Brown criterion. The average E, based on the pressuremeter tests in Salina Group Unit F, falls within the upper and lower bound E based on Hoek-Brown criterion.

For Salina Group Unit E (average RQD is 72 percent) and Unit C (average RQD is 97 percent), the E of bedrock based on the average V_s are greater than the average E measured from laboratory unconfined compression tests. The ratio of the E based on laboratory tests to the E from the average V_s are approximately 0.9 and 0.8 for Unit E and Unit C, respectively, which shows good agreement. The E calculated from average V_s and laboratory tests are greater than the upper bound E using the Hoek-Brown criterion.

For Salina Group Unit B (average RQD is 97 percent), the E using laboratory tests is greater than the E based on the average $V_{s,}$ with a ratio of approximately 1.3, which is in agreement. The calculated E based on the average V_s falls within the upper and lower bound E based on Hoek-Brown criterion.

For analysis of settlements, the lower bound E based on the Hoek-Brown criterion for each bedrock unit were selected. It is believed that the average E of the bedrock units will be greater than the lower bound E from the Hoek-Brown criterion; therefore, estimated rebound, and total and differential settlement will represent upper limit estimates. These lower bound E are used for settlement analysis.

The buildings in the power block area are in close proximity as shown on Figure 2.5.1-236. Furthermore, the arrangement of and loading conditions on the buildings are not symmetrical. Due to the complex loading condition, a three-dimensional finite element program, PLAXIS 3D Foundation, Version 2.1, was used to estimate the settlements of Seismic Category I structures. This software is capable of analyzing load-displacement behavior of subsurface materials under complex geometry and loading situations. The 3D finite element analysis was used to take into account settlement caused by non-symmetrical loadings caused by adjacent buildings in the power block area.

The Ancillary Diesel Building (ADB) and Hot Machine Shop (HMS) will have the greatest impact to settlement of the RB/FB if located immediately adjacent to the RB/FB, as this results in the greatest stress from these structures applied below the RB/FB. Therefore, in the finite element model, the ADB and HMS are located immediately adjacent to the RB/FB.

Subsurface material properties are presented in Subsection 2.5.4.2.1. The E of bedrock selected for rebound and settlement analyses are discussed in Subsection 2.5.4.2.1.2. Other parameters such as total unit weight and Poisson's ratio are presented in Table 2.5.4-202. In addition, E, Poisson's ratio, and total unit weight of fill concrete are needed since soils underneath the FWSC are removed and backfilled with fill concrete. As stated in Subsection 2.5.4.5.4.2 the mean compressive strength of the fill concrete is 31 MPa (4,500 psi). However, a lower bound E was calculated using a reduced lean concrete compressive strength of 300 psi.The parameters for the linear elastic model are summarized in Table 2.5.4-229.

Information from Table 2.5.4-226 and Table 2.5.4-229 was used as inputs for the finite element analysis. The settlement analysis for the Seismic Category I structures was performed in stages. The initial stage was used to define the initial states of stress in the ground. The second stage simulated the rebound associated with load removal when excavation

was performed to appropriate foundation elevations or to top of bedrock in the power block area. The remaining stages were simulated to estimate settlement after loadings were applied. Only elastic settlements are considered in the analysis and there is no long term (post-construction) settlement anticipated at the Fermi 3 site.

Figure 2.5.4-227 and Figure 2.5.4-228 show the graphical results from finite element analysis for excavation rebound at the completion of excavation, and for total settlements caused by structure and fill loads, respectively. The settlement analysis results are summarized in Table 2.5.4-230 and Table 2.5.4-231, respectively, for excavation rebound, and total (settlement from the rebounded position) foundation settlements. Only settlements under Seismic Category I structures are shown in these tables. The calculated total and differential settlements in Table 2.5.4-232 are within the acceptance criteria required in the Referenced DCD.

2.5.4.10.3 **Lateral Earth Pressures**

Static and seismic lateral earth pressures are addressed for Fermi 3 below-ground walls. From the Referenced DCD, the lateral soil pressure at rest is applied to external walls for RB/FB and CB. Therefore, the RB/FB and CB walls are assumed to not yield due to the lateral earth pressure applied to them. The at-rest pressure is the appropriate earth pressure to use for design of the walls per the Referenced DCD. For the Firewater Service Complex, the lateral soil pressure is not considered since it has no below-grade walls.

For a conservative analysis, the engineered granular backfill was assumed to be resting on the RB/FB and CB walls from finish grade to bottom of foundation. Therefore, properties of engineered granular backfill were used for calculating lateral earth pressure from plant grade to the bottom of foundation. It is expected that the ϕ' of the engineered granular backfill is a minimum of 35 degree; therefore $\phi' = 35°$ was used for lateral pressure analysis. The saturated and unsaturated unit weights of 21.2 and 20.4 kN/m^3 (135 and 130 pcf), respectively, was conservatively assumed for the engineered granular backfill.

Hydrostatic pressures are conservatively based on the groundwater table being 0.6 m (2 ft) below finished ground level grade [El. 179.0 m (587.3 ft), NAVD 88]. A surcharge pressure of 24 kPa (500 psf) is used. Considering the small to medium sized compaction equipment normally used for compaction of backfill behind rigid retaining walls, a 24 kPa (500

psf) compactive surcharge pressure is appropriate for the additional compaction lateral earth pressures that are developed (Reference 2.5.4-245).

2.5.4.10.3.1 Static Lateral Earth Pressures

The at-rest static lateral earth pressure σ_h for a given depth z is calculated as follows (Reference 2.5.4-246):

$$\sigma_h = K_0 \sigma_0^{'} + u \qquad \text{[Eq. 8]}$$

where:

K_0 = coefficient of at-rest earth pressure = $1 - \sin\phi$

u = pore water pressure

$\sigma_0^{'}$ = effective vertical subsurface stress = $q + \gamma z$ (q is surcharge load, γ is effective soil unit weight)

ϕ = angle of internal friction = 35 degree

2.5.4.10.3.2 Dynamic Lateral Earth Pressures

A method developed by Ostadan is used to compute seismic soil pressure on building walls (Reference 2.5.4-247). The peak response horizontal ground acceleration was used for the analyses of the seismic lateral earth pressure on the RB/FB and CB walls. The peak response horizontal ground acceleration is approximately 0.50g for both the RB/FB and CB based on site-specific FIRS as shown on Figure 2.5.2-289 and Figure 2.5.2-290.

2.5.4.10.3.3 Results of Lateral Earth Pressure Analyses

The results of the static soil lateral earth pressure and seismic soil lateral earth pressure for the RB/FB and CB are shown on Figure 2.5.4-229 and Figure 2.5.4-230, respectively.

2.5.4.11 Design Criteria

DCD Table 2.0-1 shows the envelope of ESBWR standard site parameters. Subsection 2.5.4 addresses specifically the following parameters listed in DCD Table 2.0-1:

- Minimum Static Bearing Capacity.
- Minimum Dynamic Bearing Capacity.

- Minimum Shear Wave Velocity.
- Liquefaction Potential.
- Angle of Internal Friction.
- Maximum Settlement Values for Seismic Category I Buildings.

The design criteria required for minimum static and dynamic bearing capacity is addressed in Subsection 2.5.4.10.1. The factor of safety for static bearing capacity is at least 3 while for the dynamic bearing capacity is at least 2.25. The selection of shear strength parameters used in the bearing capacity evaluation is discussed in Subsection 2.5.4.2.1.

Results of the geophysical surveys for shear wave velocity are presented in Subsection 2.5.4.4.1 and shear wave velocity profiles are summarized in Subsection 2.5.4.7.2. The minimum shear wave velocity of the supporting foundation material associated with seismic strains for lower bound soil properties at minus one sigma from the mean is greater than 300 m/s (1,000 fps) as discussed in Subsection 2.5.4.7.2. Fill concrete is used as the backfill surrounding RB/FB and CB embedded walls below the top of bedrock and below the FWSC. Fill concrete meets the shear wave velocity requirement. For backfill above the top of the bedrock surrounding Seismic Category I embedded walls, the Fermi 3 site-specific soil-structure interaction analyses (Subsection 3.7.2) show that the Referenced DCD requirements for backfill surrounding Seismic Category I RB/FB and CB are not required.

For the FWSC, the supporting material below the FWSC at Fermi 3 is fill concrete with a mean compressive strength of 31 MPa (4,500 psi) with 3000 mm (9.8 foot) deep shear keys extending into the fill concrete; therefore, sliding of the FWSC is not an issue as discussed in Subsection 3.8.5 when neglecting the engineered granular backfill surrounding the basemat. Therefore, the Referenced DCD requirements for backfill surrounding FWSC are not required.

The static stability analyses are presented in Subsection 2.5.4.10. The design criteria for static stability analyses are identified in Subsection 2.5.4.10 and are compared to site parameters in Table 2.0-201. Discussion of the assumptions and methods of analyses for the static stability analyses are provided in Subsection 2.5.4.10.

Subsection 2.5.4.8 discusses the liquefaction potential of soils encountered and fill at the site. It is concluded that there are no liquefiable soils under and adjacent to all Seismic Category I structures.

DCD Table 2.0-1 requires that $\phi' \geq 35°$. Seismic Category I structures are founded on bedrock or fill concrete extending to bedrock. The angle of internal friction of bedrock is greater than 35 degree based on laboratory direct shear tests performed on samples with discontinuities from the Bass Islands Group and empirical correlations using Hoek-Brown criterion. Engineered granular backfill is used to backfill adjacent to all Seismic Category I structures and based on compaction requirements the angle of internal friction of engineered granular backfill should be greater than 35 degrees.

The design criteria required for the foundation settlement for Seismic Category I structures are addressed in Subsection 2.5.4.10.2. The calculated foundation settlements of all Seismic Category I structures were demonstrated to be less than the maximum settlement values specified in the Referenced DCD.

The computer program used in the settlement analysis (Subsection 2.5.4.10.2) was validated by comparing the results obtained from computer program to solutions obtained from theoretical equations.

2.5.4.12 Techniques to Improve Subsurface Conditions

The RB/FB and CB are founded on bedrock. Based on the stability analysis presented on Subsection 2.5.4.10, no subsurface improvement is needed. The exposed foundation bedrock is sluiced with high-pressure water jets and carefully examined by a qualified geologist to ensure that no excessive natural fracturing or blasting back-break exists that might be unsuitable for foundation support. Any areas with open fractures are filled with concrete backfill.

For the FWSC and Seismic Category II structures, all soils are removed below the foundation to the top of bedrock and replaced with fill concrete to improve subsurface conditions.

2.5.4.13 References

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Table 2.5.4-201 Approximate Elevation Ranges for Each Subsurface Material Encountered at Fermi 3 [EF3 COL 2.0-29-A]

Subsurface Material	Approximate Ranges of Elevation in NAVD 88 (ft)			Average Thickness (ft)
Fill	581	to	568	13
Lacustrine Deposits	568	to	563	5
Glacial Till	563	to	552	11
Bass Islands Group	552	to	462	90
Salina Group Unit F	462	to	339	123
Salina Group Unit E	339	to	246	93
Salina Group Unit C	246	to	156	90
Salina Group Unit B	156	to	--	--

Note: the bottom of Salina Group Unit B was not encountered during the geotechnical investigation.
NAVD 88 = North American Vertical Datum 1988
ft = feet

Table 2.5.4-202 Summary Engineering Properties of Soils and Bedrock (Sheet 1 of 2)

Stratum	Quarry Fill	Lacustrine Deposits	Glacial Till	Bass Islands Group	Salina Group Unit F	Salina Group Unit E	Salina Group Unit C	Salina Group Unit B
USCS Symbol	GP/GW	CL/CH	CL	-	-	-	-	-
Total Unit Weight, γ (pcf)	125	130	135	150	150(1)	150(1)	160	160(1)
Fines Content (%)	-	93	68	-	-	-	-	-
Natural Water Content (%)	-	27	15	0.1	0.4	3.9	0.9	0.2
Atterberg Limits								
Liquid Limit, LL (%)	-	44	29	-	-	-	-	-
Plastic Limit, PL (%)	-	17	15	-	-	-	-	-
Plasticity Index, PI (%)	-	27	14	-	-	-	-	-
Adjusted SPT N_{60}-value (blows/foot)	11	7	47	-	-	-	-	-
Undrained Shear Strength, s_u (ksf)	-	0.9	2.7	-	-	-	-	-
Effective Shear Strength Parameters								
Effective Cohesion, c' (ksf)	0	0	0	-	-	-	-	-
Effective Friction Angle, ϕ' (degrees)	36	29	31	-	-	-	-	-
Rock Quality Designation, RQD (%)	-	-	-	54	13	72	97	97
Unconfined Compressive Strength of rock, q_u (ksf)	-	-	-	1,870	940	1,760	1,800	1,540
Poisson Ratio, υ	0.35(2)	0.35/0.49(3)	0.35/0.49(3)	0.33	0.39	0.30	0.28	0.29

Table 2.5.4-202 Summary Engineering Properties of Soils and Bedrock (Sheet 2 of 2) [EF3 COL 2.0-29-A]

Stratum	Quarry Fill	Lacustrine Deposits	Glacial Till	Bass Islands Group	Salina Group Unit F	Salina Group Unit E	Salina Group Unit C	Salina Group Unit B
Modulus of Elasticity based on Hoek-Brown Criterion								
Upper Bound Modulus of Elasticity (ksf)	-	-	-	109,500	31,700	492,100	623,000	1,324,700
Mean Modulus of Elasticity (ksf)	-	-	-	80,700	24,200	424,200	559,300	1,228,400
Lower Bound Modulus of Elasticity (ksf)	-	-	-	59,900	19,300	349,000	482,100	1,102,700
Modulus of Elasticity based on Laboratory Test (ksf)	-	-	-	898,600	529,200	671,500	763,200	1,504,800
Modulus of Elasticity based on Average V_s (ksf)	-	-	-	556,200	132,600	755,800	1,007,600	1,156,900
Average Shear Wave Velocity, V_s (fps) (4)	-	-	800 to 1,150(7)	6,700 to 7,300	3,200 to 4,000	7,900 to 9,100	8,900 to 9,000	9,500 to 9,900
Average Compression Wave Velocity, V_p (fps) (5)	-	-	-	13,200 to 14,400	8,000 to 9,400	15,300 to 16,200	15,900 to 16,100	17,500 to 18,300
Shear Modulus at very small strain levels, G_{max} (ksf)(6)	-	-	2,700	209,100	47,700	290,700	393,600	448,400

Notes:
1. The mean total unit weight was high; therefore, lower total unit weight was chosen.
2. Assumed Poisson's ratio for fill under drained loading condition.
3. Assumed Poisson's ratio under drained loading condition / assumed Poisson's ratio for under undrained loading condition.
4. Average V_s is range of mean V_s measured from P-S Suspension Logger in all borings.
5. Average V_p is range of mean V_p measured from P-S Suspension Logger in all borings.
6. G_{max} is calculated based on lowest mean V_s.
7. Vs is from SASW

pcf = pounds per cubic foot, ksf = kips per square foot,% = percent, fps = feet per second

Table 2.5.4-203 Statistical Analysis of Results from Field and Laboratory Test Performed for Lacustrine Deposits [EF3 COL 2.0-29-A]

Statistical Description	N_{60}	Natural Moisture Content	Dry Unit Weight	Liquid Limit	Plastic Limit	Plasticity Index	Fines	Undrained Shear Strength Measured from	
								UU Test(2)	UC Test(3)
	bpf	(%)	(pcf)	(%)	(%)	(%)	(%)	(ksf)	(ksf)
Minimum	0	23	103	34	16	17	82	0.28	0.51
Maximum	14	33	106	54	20	37	99	1.33	0.51
Median	7	27	106	46	17	29	94	0.81	0.51
Mean	7	27	105	44	17	27	93	0.81	0.51
Standard Deviation	4	4	1	7	1	7	6.6	--	--
Count(1)	15	7	3	8	8	8	5	2	1

Notes:

1. Count is the number samples obtained or tests performed and it is dimensionless.
2. UU test is the unconsolidated-undrained triaxial compression test.
3. UC test is the unconfined compression test.

bpf = blows per foot
pcf = pounds per cubic foot
ksf = kips per square foot
% = percent

Table 2.5.4-204 Statistical Analysis of Results from Field and Laboratory Test Performed for Glacial Till [EF3 COL 2.0-29-A]

Statistical Description	N_{60}	Natural Moisture Content	Dry Unit Weight	Liquid Limit	Plastic Limit	Plasticity Index	Fines	Undrained Shear Strength Measured from	
								UU Test(2)	UC Test(3)
	bpf	(%)	(pcf)	(%)	(%)	(%)	(%)	(ksf)	(ksf)
Minimum	9	9	105	18	11	7	17	1.3	2.3
Maximum	78	25	130	47	20	27	97	1.8	3.2
Median	52	13	110	26	14	13	71	1.6	2.3
Mean	47	15	114	29	15	14	68	1.6	2.6
Standard Deviation	19	6	10	9	3	6	23	0.3	0.5
Count(1)	72	20	8	22	22	22	17	2	3

Notes:

1. Count is the number samples obtained or tests performed and it is dimensionless.
2. UU test is the unconsolidated-undrained triaxial compression test.
3. UC test is the unconfined compression test.

bpf = blows per foot
pcf = pounds per cubic foot
ksf = kips per square foot
% = percent

Table 2.5.4-205 Input Parameters to Estimate Rock Mass Strength

[EF3 COL 2.0-29-A]

Rock Unit		Classification	Dominant Rock Type	GSI	mi	D	q_u (ksf)	E (ksf)
Bass Islands Group		between blocky and very blocky structure with fair to good surface condition	Dolomite	55 ± 5	9 ± 3	1(1)	1,870	898,600
Salina Group	Unit F	between blocky/disturbed/ seamy and disintegrated with poor to very poor surface condition	Claystone	20 ± 5	4 ± 2	0(2)	940	529,200
	Unit E	blocky structure with good surface condition	Dolomite	65 ± 5	9 ± 3	0(2)	1,760	671,500
	Unit C	between intact or massive and blocky structure with good surface condition	Shale	70 ± 5	6 ± 2	0(2)	1,800	763,200
	Unit B	intact or massive structure with good surface condition	Dolomite	75 ± 5	9 ± 3	0(2)	1540	1,504,800

Notes:

1. D = 1.0, which indicates significant disturbance in bedrock due to blasting and stress relief.
2. D = 0, which indicates undisturbed bedrock condition; it is reasonable that no blast damage exists or excavation disturbance for these bedrock units since they exists at least 110 feet below ground.

GSI = geological strength index
mi = material index
D = disturbance factor
ksf = kips per square foot
q_u = unconfined compressive strength
E = modulus of elasticity

Table 2.5.4-206 Statistical Analysis of Results from Field and Laboratory Test Performed for Bass Islands Group [EF3 COL 2.0-29-A]

Statistical Description	Percent Recovery	RQD	Measured Moisture Content	Total Unit Weight	Unconfined Compression Strength of Intact Rock	Modulus of Elasticity of Intact Rock	Residual Friction Angle along Rock Discontinuity
	(%)	(%)	(%)	(pcf)	(ksf)	(ksf)	(degree)
Minimum	0.0	0.0	0.0	125	960	331,200	33
Maximum	100.0	100.0	0.3	169	3,210	1,641,600	74
Median	100.0	58.0	0.1	152	1,650	842,400	51
Mean	94.0	53.7	0.1	151	1,870	898,600	52
Standard Deviation	14.7	26.1	0.1	11	620	318,800	12
Count(1)	490	490	20	20	20	20	12

Notes:

1. Count is the number samples obtained or tests performed and it is dimensionless.

pcf = pounds per cubic foot
ksf = kips per square foot
% = percent
RQD = rock quality designation

Table 2.5.4-207 Rock Mass Properties for Rock Units Encountered at Fermi 3 based on Hoek-Brown Criterion [EF3 COL 2.0-29-A]

		Uniaxial Compressive Strength			Global Compressive Strength			Rock Mass Modulus		
		Upper Bound	**Mean**	**Lower Bound**	**Upper Bound**	**Mean**	**Lower Bound**	**Upper Bound**	**Mean**	**Lower Bound**
Rock Unit		**(ksf)**	**(ksf)**	**(ksf)**	**(ksf)**	**(ksf)**	**(ksf)**	**(ksf)**	**(ksf)**	**(ksf)**
Bass Islands Group		66	43	28	210	150	101	109,500	80,700	59,900
Salina Group	Unit F	11.2	7.5	4.7	68	46	26	31,700	24,200	19,300
	Unit E	330	249	188	530	415	309	492,100	424,200	349,000
	Unit C	448	338	256	545	423	317	623,000	559,300	482,100
	Unit B	507	383	290	622	484	362	1,324,700	1,228,400	1,102,700

ksf = kips per square foot

Table 2.5.4-208 Mohr-Coulomb Parameters for Bedrock Units Encountered at Fermi 3 based on Hoek-Brown Criterion [EF3 COL 2.0-29-A]

Rock Unit		Friction Angle, ϕ'			Cohesion Intercept, c'		
		Upper Bound (degree)	Mean (degree)	Lower Bound (degree)	Upper Bound (ksf)	Mean (ksf)	Lower Bound (ksf)
Bass Islands Group		53	48	42	10.2	7.6	5.8
Salina Group	Unit F	44	38	28	3.1	2.3	1.6
	Unit E	61	58	53	41.9	34.6	30
	Unit C	55	52	47	70.5	58	50.1
	Unit B	59	56	52	69.4	57.3	49.7

ksf = kips per square foot

Table 2.5.4-209 Statistical Analysis of Measured Compression and Shear Wave Velocities using P-S Suspension Logger in the Bass Islands Group

[EF3 COL 2.0-29-A]

Statistical Description	Compression Wave Velocity, V_p (fps)				Shear Wave Velocity, V_s (fps)			
	TB-C5	RB-C8	CB-C3	RB-C4	TB-C5	RB-C8	CB-C3	RB-C4
Minimum	8,400	7,800	10,400	7,800	2,600	3,300	3,500	3,400
Maximum	19,600	16,700	19,000	20,800	9,000	9,700	10,500	10,800
Median	13,900	14,200	14,200	13,200	6,800	6,900	7,800	6,300
Mean	13,600	13,700	14,400	13,200	6,700	6,900	7,300	6,600
Standard Deviation	2,500	1,900	2,300	2,800	1,400	1,300	1,600	1,800
Count(1)	55	53	52	39	55	53	52	39

Notes: All velocity values listed above are rounded to the nearest 100 fps.

1. Count is the number samples obtained or tests performed and it is dimensionless.

fps = feet per second

Table 2.5.4-210 Statistical Analysis of Results from Field and Laboratory Test Performed for the Salina Group Unit F

[EF3 COL 2.0-29-A]

Statistical Description	Percent Recovery (%)	RQD (%)	Measured Moisture Content (%)	Total Unit Weight (pcf)	Unconfined Compression Strength of Intact Rock (ksf)	Modulus of Elasticity of Intact Rock (ksf)
Minimum	0.0	0.0	0.0	137	45	16,000
Maximum	100.0	100.0	2.4	196	3,070	1,080,000
Median	60.0	0.0	0.1	156	750	547,200
Mean	59.4	13.5	0.4	157	940	529,300
Standard Deviation	29.5	19.1	0.7	19	910	376,200
Count(1)	506	506	13	13	13	13

Notes:

1. Count is the number samples obtained or tests performed and it is dimensionless.

pcf = pounds per cubic foot
ksf = kips per square foot
% = percent
RQD = rock quality designation

Table 2.5.4-211 Statistical Analysis of Measured Compression and Shear Wave Velocities using P-S Suspension Logger in Salina Group Unit F

[EF3 COL 2.0-29-A]

Statistical Description	Compression Wave Velocity, V_p (fps)				Shear Wave Velocity, V_s (fps)			
	TB-C5	RB-C8	CB-C3	RB-C4	TB-C5	RB-C8	CB-C3	RB-C4
Minimum	5100	7200	7500	6900	1800	2900	2800	2600
Maximum	12300	12100	14200	12600	5200	6400	7500	6600
Median	7700	10000	9100	9000	3000	4400	3800	4500
Mean	8000	9700	9400	9300	3200	4600	4000	4200
Standard Deviation	1200	1600	1500	1600	700	1100	1000	1100
Count(1)	76	18	80	28	76	18	80	28

Notes: All velocity values listed above are rounded to the nearest 100 fps.

1. Count is the number samples obtained or tests performed and it is dimensionless.

fps = feet per second

Table 2.5.4-212 Statistical Analysis of Results from Field and Laboratory Test Performed for the Salina Group Unit E

[EF3 COL 2.0-29-A]

Statistical Description	Percent Recovery (%)	RQD (%)	Measured Moisture Content (%)	Total Unit Weight (pcf)	Unconfined Compression Strength of Intact Rock (ksf)	Modulus of Elasticity of Intact Rock (ksf)
Minimum	30.0	0.0	0.1	140	450	273,600
Maximum	100.0	100.0	16.8	166	2,760	1,339,200
Median	100.0	86.0	0.3	150	1,750	640,800
Mean	93.6	71.6	3.9	151	1,750	671,400
Standard Deviation	12.4	30.7	6.6	8	840	332,400
Count[(2)]	107	107	8	8	8	8

Notes:

1. Count is the number samples obtained or tests performed and it is dimensionless.

pcf = pounds per cubic foot
ksf = kips per square foot
% = percent
RQD = rock quality designation

Table 2.5.4-213 Statistical Analysis of Measured Compression and Shear Wave Velocities using P-S Suspension Logger in the Salina Group Unit E

[EF3 COL 2.0-29-A]

Statistical Description	Compression Wave Velocity, V_p (fps)				Shear Wave Velocity, V_s (fps)			
	TB-C5	RB-C8	CB-C3	RB-C4	TB-C5	RB-C8	CB-C3	RB-C4
Minimum	7500	9000	10400	10000	2800	5000	4900	4300
Maximum	21500	20200	13300	11300	10800	10900	8200	6800
Median	17300	17100	11100	11000	9100	9700	5600	5400
Mean	15300	16200	11500	10700	7900	9100	6100	5500
Standard Deviation	4300	2500	1000	600	2700	1500	1100	900
Count(1)	54	57	7(2)	8(2)	54	57	7(2)	8(2)

Notes: All velocity values listed above are rounded to the nearest 100 fps.

1. Count is the number samples obtained or tests performed and it is dimensionless.
2. Borings CB-C3 and RB-C4 only penetrated approximately 20 to 30 feet into the Salina Group Unit E; therefore, only a limited number of measurements were performed.

fps = feet per second

Table 2.5.4-214 Statistical Analysis of Results from Field and Laboratory Test Performed for the Salina Group Unit C

[EF3 COL 2.0-29-A]

Statistical Description	Percent Recovery (%)	RQD (%)	Measured Moisture Content (%)	Total Unit Weight (pcf)	Unconfined Compression Strength of Intact Rock (ksf)	Modulus of Elasticity of Intact Rock (ksf)
Minimum	94.0	80.0	0.9	167	1,390	676,800
Maximum	100.0	100.0	0.9	167	2,200	849,600
Median	100.0	100.0	0.9	167	1,790	763,200
Mean	99.4	97.2	0.9	167	1,790	763,200
Standard Deviation	1.7	5.1	0.0	0.4	570	122,200
Count(1)	37	37	2	2	2	2

Notes:

1. Count is the number samples obtained or tests performed and it is dimensionless.

pcf = pounds per cubic foot
ksf = kips per square foot
% = percent
RQD = rock quality designation

Table 2.5.4-215 Statistical Analysis of Measured Compression and Shear Wave Velocities using P-S Suspension Logger in the Salina Group Unit C

[EF3 COL 2.0-29-A]

Statistical Description	Compression Wave Velocity, V_p (fps)		Shear Wave Velocity, V_s (fps)	
	TB-C5	RB-C8	TB-C5	RB-C8
Minimum	14200	13600	8100	8200
Maximum	19000	18000	10500	10400
Median	16300	15900	8900	9000
Mean	16100	15900	8900	9000
Standard Deviation	900	1000	400	400
Count(1)	53	57	53	57

Notes: All velocity values listed above are rounded to the nearest 100 fps.

1. Count is the number samples obtained or tests performed and it is dimensionless.

fps = feet per second

Table 2.5.4-216 Statistical Analysis of Results from Field and Laboratory Test Performed for Salina Group Unit B [EF3 COL 2.0-29-A]

Statistical Description	Percent Recovery (%)	RQD (%)	Measured Moisture Content (%)	Total Unit Weight (pcf)	Unconfined Compression Strength of Intact Rock (ksf)	Modulus of Elasticity of Intact Rock (ksf)
Minimum	96.0	80.0	0.1	145	1,130	1,440,000
Maximum	100.0	100.0	0.3	170	1,940	1,569,600
Median	100.0	100.0	0.2	158	1,540	1,504,800
Mean	99.8	97.1	0.2	158	1,540	1,504,800
Standard Deviation	1.0	5.4	0.2	18	570	91,600
Count[1]	17	17	2	2	2	2

Notes:

1. Count is the number samples obtained or tests performed and it is dimensionless.

pcf = pounds per cubic foot
ksf = kips per square foot
% = percent
RQD = rock quality designation

Table 2.5.4-217 Statistical Analysis of Measured Compression and Shear Wave Velocities using P-S Suspension Logger in Salina Group - Unit B

[EF3 COL 2.0-29-A]

Statistical Description	Compression Wave Velocity, V_p (fps)		Shear Wave Velocity, V_s (fps)	
	TB-C5	RB-C8	TB-C5	RB-C8
Minimum	15,200	15,500	8,300	8,400
Maximum	20,800	20,200	11,400	11,900
Median	17,100	18,300	9,400	9,900
Mean	17,500	18,300	9,500	9,900
Standard Deviation	1,600	1,500	900	1,000
Count(1)	17(2)	18(2)	17(2)	18(2)

Notes: All velocity values listed above are rounded to the nearest 100 fps.

1. Count is the number samples obtained or tests performed and it is dimensionless.
2. Borings TB-C5 and RB-C8 penetrated approximately 40 to 50 feet into Salina Group Unit B

fps = feet per second

Table 2.5.4-218 Elevations, Boring Depths and Depths to Top of Each Soil/Rock Layer Observed from Each Boring (Sheet 1 of 3)

[EF3 COL 2.0-29-A]

	Coordinates				Depth to Top of Each Soil or Rock Layer						
			Ground El. (NAVD 88)	Boring Depth	Lacustrine	Glacial Till	Bass Islands Group	Salina Group Unit F	Salina Group Unit E	Salina Group Unit C	Salina Group Unit B
Boring No.	Plant East	Plant North	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
CB-C1	4451.90	6363.40	580.58	128.0	12.7	16.5	32.5	125.2	--	--	--
CB-C2	4517.10	6353.16	580.48	271.0	13.0	18.3	27.0	123.3	247.2	--	--
CB-C3	4673.77	6341.35	581.08	273.7	15.4	17.5	32.5	121.7	246.6	--	--
CB-C4	4663.36	6230.32	580.78	133.5	13.0	15.0	28.5	116.8	--	--	--
CB-C5	4663.37	6122.01	580.98	131.0	12.3	15.3	26.5	111.4	--	--	--
CST-AB1	4331.06	6577.60	580.18	30.8	13.8	18.1	30.9	--	--	--	--
CT-E1	4992.63	6061.79	579.88	26.5	15.0	18.3	26.5	--	--	--	--
EB/TSC-C2	4698.01	6579.19	581.37	50.5	12.1	15.5	29.0	--	--	--	--
EB/TSC-E3	4819.48	6567.30	581.78	130.5	11.5	15.5	28.7	122.6	--	--	--
FO-E1	4915.04	6490.21	581.18	130.3	12.8	18.2	28.5	117.9	--	--	--
FWS/ACB-C1	4776.34	6203.14	581.41	105.0	16.0	17.8	28.3	--	--	--	--
HM-E1	4530.21	5985.29	580.98	75.5	13.0	19.0	27.6	--	--	--	--
PS-E1	4247.18	6164.86	579.91	28.8	8.3	17.0	28.8	--	--	--	--
RB-C1	4445.27	6273.16	579.18	271.0	10.3	15.5	29.0	126.0	238.5	--	--
RB-C2	4525.65	6273.16	579.28	270.0	10.5	14.5	31.0	118.6	241.7	--	--
RB-C3	4606.02	6273.16	580.08	274.0	13.5	16.0	30.0	121.6	243.6	--	--
RB-C4	4448.80	6188.94	580.18	271.5	12.0	17.8	31.3	120.5	239.6	--	--

Table 2.5.4-218 Elevations, Boring Depths and Depths to Top of Each Soil/Rock Layer Observed from Each Boring (Sheet 2 of 3)

[EF3 COL 2.0-29-A]

	Coordinates		Ground El. (NAVD 88)	Boring Depth	Depth to Top of Each Soil or Rock Layer						
					Lacustrine	Glacial Till	Bass Islands Group	Salina Group Unit F	Salina Group Unit E	Salina Group Unit C	Salina Group Unit B
Boring No.	Plant East	Plant North	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
RB-C5	4525.65	6192.78	580.58	270.5	10.8	18.5	29.0	117.5	239.3	--	--
RB-C6	4606.02	6192.78	580.78	240.0	12.4	17.8	29.5	113.7	238.0	--	--
RB-C7	4450.00	6117.00	580.48	272.0	14.5	17.5	29.5	115.0	237.0		
RB-C8	4534.40	6110.90	580.38	471.5	13.5	16.5	26.0	116.7	236.7	329.5	422.8
RB-C9	4616.02	6112.40	580.88	270.5	14.7	19.0	28.0	113.7	237.7	--	--
RB-C10	4451.25	6050.66	581.06	271.3	13.8	21.0	27.4	114.5	237.7	--	--
RB-C11	4534.60	6051.82	580.02	135.5	13.0	18.5	27.0	113.5	--	--	--
RB-C12	4606.52	6043.51	581.08	178.1	13.5	20.5	27.0	110.9	--	--	--
RW-C1	4177.50	6384.11	580.68	270.9	12.5	14.5	27.5	125.9	251.1	--	--
RW-C2	4312.03	6384.11	580.88	75.5	12.5	18.0	29.7	--	--	--	--
RW-C3	4177.50	6291.89	580.38	105.5	10.0	17.2	29.5	--	--	--	--
RW-C4	4312.03	6291.89	580.18	146.5	13.0	16.0	32.5	129.0	--	--	--
TB-C1	4382.69	6671.84	580.68	30.9	11.0	17.3	30.9	--	--	--	--
TB-C2	4394.43	6520.09	579.68	30.0	14.3	17.5	30.0	--	--	--	--
TB-C3	4474.57	6673.83	580.58	51.0	14.0	17.5	28.0	--	--	--	--
TB-C4	4520.67	6510.21	580.68	54.0	16.4	19.0	28.5	--	--	--	--
TB-C5	4593.00	6677.70	580.78	471.1	10.0	12.0	24.6	121.3	245.5	339.7	426.1

Table 2.5.4-218 Elevations, Boring Depths and Depths to Top of Each Soil/Rock Layer Observed from Each Boring (Sheet 3 of 3)

[EF3 COL 2.0-29-A]

	Coordinates		Ground El. (NAVD 88)	Boring Depth	Depth to Top of Each Soil or Rock Layer						
					Lacustrine	Glacial Till	Bass Islands Group	Salina Group Unit F	Salina Group Unit E	Salina Group Unit C	Salina Group Unit B
Boring No.	Plant East	Plant North	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
TB-C6	4615.85	6510.21	580.58	51.3	15.0	17.0	32.0	--	--	--	--
WT-E1	4982.62	6223.84	580.48	27.1	15.5	17.5	27.1	--	--	--	--

NAVD 88 = North American Vertical Datum 1988
ft = feet

Table 2.5.4-219 Pressuremeter Testing Locations and Results in Boring RB-C6 (Sheet 1 of 2) [EF3 COL 2.0-29-A]

Date	File Name	Bottom of Test Pocket (ft)	Depth to Bottom of Pressure-meter (ft)	Core Run Recovery (%)	RQD (%)	Strain Before Testing (%)	Initial or Menard Modulus (E_o) (ksf)	Previous Unload-Reload Elastic Modulus (Eur,prev) (ksf)	Final Unload-Reload Elastic Modulus (Eur,last) (ksf)	Ratio of Eur,last / Eur,prev	Ratio of Eur,last / E_o	Note
9/14/2007	FMI-1Z	126.5	125.5	54	0	4	33,324	84,269	162,792	1.9	4.9	5
9/14/2007	FMI-3Z	142.0	140.2	34	0	6	1,992	7,661	9,193	1.2	4.6	5
9/14/2007	FMI-2Z	142.0	141.8	34	0	4	4,980	9,193	23,365	2.5	4.7	6
9/15/2007	FMI-5Z	163.5	161.9	33	0	--	--	--	--	--	--	7
9/15/2007	FMI-4Z	163.5	163.4	33	0	6	2,566	10,725	57,456	5.4	22.4	5
9/15/2007	FMI-7Z	173.5	171.8	15	0	--	--	--	--	--	--	8
9/15/2007	FMI-6Z	173.5	173.3	15	0	--	--	--	--	--	--	8
9/16/2007	FMI-9Z	185.0	183.4	34	0	--	--	--	--	--	--	8
9/16/2007	FMI-8Z	185.0	184.9	34	0	6	1,149	3,447	5,363	1.6	4.7	6
9/17/2007	FMI-10Z	209.0	208.0	28	0	5	613	7,278	8,427	1.2	13.8	5
9/17/2007	FMI-12Z	215.0	213.2	42	0	0	25,281	95,760	287,280	3.0	11.4	9
9/17/2007	FMI-11Z	215.0	214.7	42	0	5	6,129	45,965	95,760	2.1	15.6	5
9/18/2007	--	~229.0	--	--	--	--	--	--	--	--	--	10
9/18/2007	--	235.5	--	--	--	--	--	--	--	--	--	10
9/19/2007	--	~240.0	--	--	--	--	--	--	--	--	--	11

Table 2.5.4-219 Pressuremeter Testing Locations and Results in Boring RB-C6 (Sheet 2 of 2) [EF3 COL 2.0-29-A]

Date	File Name	Bottom of Test Pocket (ft)	Depth to Bottom of Pressure-meter (ft)	Core Run Recovery (%)	RQD (%)	Strain Before Testing (%)	Initial or Menard Modulus (E_o) (ksf)	Previous Unload-Reload Elastic Modulus (Eur,prev) (ksf)	Final Unload-Reload Elastic Modulus (Eur,last) (ksf)	Ratio of Eur,last / Eur,prev	Ratio of Eur,last / E_o	Note

Notes:

1. The center of the pressuremeter test section is 1.3 feet above the base of the pressuremeter.
2. FMI-2Z and FMI-3Z, and FMI-11Z and FMI-12Z were in which pressuremeter data were obtained in the same test pocket.
3. The previous unload-reload modulus is modulus from unload-reload curve prior to last unload-reload cycle.
4. RQD – Rock quality designation.
5. Test was successfully performed with three unload-reload cycles.
6. Test was successfully performed with two unload-reload cycles.
7. Borehole too big to test. No test performed.
8. Borehole too big to test. No test performed.
9. Test was successfully performed with four unload-reload cycles.
10. Borehole too large or partially filled. No test performed.
11. NQ bit broken off at 240 ft. Hole filled with sediment. No test performed.

ft = feet
% = percent
ksf = kips per square foot

Table 2.5.4-220 Results of Index, Gradation and Chemical Tests on Soil Samples (Sheet 1 of 2) [EF3 COL 2.0-29-A]

Boring No.	Sample No.	Depth (ft)	Gravel (%)	Sand (%)	Fines (%)	Silt Size Fraction (%)	Clay Size Fraction (%)	USCS Symbol	Natural Moisture Content (%)	LL (%)	PI (%)	G_s	pH	Chloride (mg/kg)	Sulfate (mg/kg)
CB-C1	TW-5	18.0-20.0	--	--	87.6	--	--	CL	--	41	22	--	--	--	--
CB-C2	TW-3	16.0-18.0	--	--	96.1	--	--	CH	--	51	31	--	--	--	--
CB-C3	TW-4	18.0-20.0	54.5	19.2	26.3	--	--	GC	18.9	41	22	--	--	--	--
CB-C3	SPT-6	23.0-24.5	4.1	36.4	59.5	--	--	CL-ML	8.5	18	7	--	--	--	--
CB-C4	S-2	13.0-14.0	--	--	--	--	--	CL	28.8	45	29	--	--	--	--
CB-C4	TW-3	15.0-17.0	2.6	9.8	87.6	30.8	56.8	CL	23	32	16	2.72	--	--	--
CB-C4	TW-4	17.5-19.5	0.9	7.4	91.7	42.9	48.9	CL	21.5	39	22	--	--	--	--
CB-C4	TW-6	22.5-24.5	--	--	67.4	--	--	CL	--	23	10	--	--	--	--
CB-C4	SPT-7	25.0-26.5	5.8	37.2	57	--	--	CL	9.2	18	7	--	--	--	--
CB-C5	TW-5	18.0-20.2	--	--	87.6	--	--	CL	--	29	13	--	--	--	--
CST-AB1	TW-3	15.5-17.5	--	--	90.8	--	--	CH	22.7	54	37	--	--	--	--
HM-E1	TW-4	16.0-18.0	--	--	90	--	--	CL	--	36	18	--	--	--	--
HM-E1	TW-6	21.5-23.5	--	--	--	--	--	--	--	--	--	--	7.76	<28.3	283
PS-E1	TW-4	13.5-15.5	--	--	94.1	--	--	CL	28.6	46	30	--	--	--	--
PS-E1	TW-6	18.5-20.5	--	--	--	--	--	--	--	--	--	--	7.52	<32.5	193
RB-C1	SPT-2	15.0-16.5	--	--	--	--	--	CL	19.5	37	19	--	--	--	--
RB-C1	TW-4	21.0-22.5	2.4	26.5	71.1	--	--	CL	12.2	23	10	--	--	--	--
RB-C1	SPT-5	25.0-26.5	12.7	35.2	52.1	--	--	CL-ML	10.7	18	7	--	--	--	--
RB-C2	TW-4	15.0-17.0	--	--	--	--	--	CL	22	38	20	--	--	--	--
RB-C2	SPT-5	20.0-21.5	0.7	18.6	80.7	--	--	CL	13.2	25	12	--	--	--	--

Table 2.5.4-220 Results of Index, Gradation and Chemical Tests on Soil Samples (Sheet 2 of 2) [EF3 COL 2.0-29-A]

Boring No.	Sample No.	Depth (ft)	Gravel (%)	Sand (%)	Fines (%)	Silt Size Fraction (%)	Clay Size Fraction (%)	USCS Symbol	Natural Moisture Content (%)	LL (%)	PI (%)	G_s	pH	Chloride (mg/kg)	Sulfate (mg/kg)
RB-C3	TW-5	20.0-22.0	--	--	--	--	--	--	--	--	--	--	7.44	<161	470
RB-C4	S-3	10.0-15.0	9.6	8.4	82	58.5	23.5	CL	24.7	34	17	--	--	--	--
RB-C4	TW-4	15.5-17.5	0	0.6	99.4	61.4	38	CL	22.7	37	20	--	--	--	--
RB-C4	SPT-5	18.0-19.5	0.6	9.7	89.7	32.3	57.4	CL	23.2	40	21	--	--	--	--
RB-C4	SPT-7	22.0-23.5	2.4	26.5	71.1	--	--	CL	9.2	24	9	--	--	--	--
RB-C4	P-9	24.0-26.0	10.4	73.1	16.5	--	--	SC	11.4	18	7	--	--	--	--
RB-C5	SPT-6	20.0-21.5	--	--	--	--	--	CL	13.5	24	10	--	--	--	--
RB-C7	SPT-2	14.5-16.0	--	--	--	--	--	CL	32.9	47	29	--	--	--	--
RB-C7	TW-3	19.5-21.5	0.9	18.3	80.8	--	--	CL	12.9	30	15	--	--	--	--
RB-C9	TW-4	18.0-20.0	0.2	3.3	96.5	--	--	CL	25.3	47	27	--	--	--	--
RB-C9	SPT-5	23.0-24.5	32.5	25.1	42.4	--	--	GC	9.6	23	11	--	--	--	--
RW-C1	SPT-2	13.0-14.5	--	--	--	--	--	CL	26.8	40	22	--	--	--	--
RW-C1	TW-3	14.5-16.5	2.5	20.8	76.7	--	--	CL	16	27	13	--	--	--	--
RW-C1	SPT-4	19.5-21.0	--	--	--	--	--	CL	12.9	24	11	2.71	--	--	--

ft = feet
% = percent
LL = liquid limit
PI = plasticity index
Gs = specific gravity
mg/kg = milligram per kilogram

Table 2.5.4-221 Results of Strength Tests on Soil Samples (Sheet 1 of 2) [EF3 COL 2.0-29-A]

Boring No.	Sample No.	Depth (ft)	Test Type	Soil Type	Confining Pressure (psi)	Maximum Principal Stress Difference Criterion $(\sigma_1' + \sigma_3')/2$ (psi)	Maximum Principal Stress Difference Criterion $(\sigma_1' - \sigma_3')/2$ (psi)	Peak Principal Stress Ratio Criterion $(\sigma_1' + \sigma_3')/2$ (psi)	Peak Principal Stress Ratio Criterion $(\sigma_1' - \sigma_3')/2$ (psi)	Undrained Shear Strength, s_u (psi)	Note
CB-C1	TW-5	18.0-20.0	CU-1	Glacial Till	8	17.65	8.3	9.73	5.18	--	
CB-C1	TW-5	18.0-20.0	CU-2	Glacial Till	15	31.65	13.39	20.76	10.88	13.39	
CB-C1	TW-5	18.0-20.0	CU-3	Glacial Till	30	54.63	26.73	38.13	20.46	--	
CB-C2	TW-3	16.0-18.0	CU-1	Lacustrine	10	11.58	5.65	11.06	5.57	5.65	1
CB-C2	TW-3	16.0-18.0	CU-2	Lacustrine	20	22.41	10.00	20.01	9.31	--	
CB-C3	TW-4	18.0-20.0	CU-1	Lacustrine	20	18.38	10.54	18.04	10.45	10.54	2
CB-C3	TW-4	18.0-20.0	CU-2	Glacial Till	40	52.86	26.47	45.35	24.86	--	
CB-C4	TW-6	22.5-24.5	CU-1	Glacial Till	20	62.72	33.22	30.86	18.97	33.22	3
CB-C4	TW-6	22.5-24.5	CU-2	Glacial Till	40	70.15	37.80	42.90	24.94	--	
CB-C5	TW-5	18.0-20.0	CU-1	Glacial Till	15	50.62	26.91	19.92	12.89	26.91	
CB-C5	TW-5	18.0-20.0	CU-2	Glacial Till	30	50.75	23.24	42.56	20.88	--	
CB-C5	TW-5	18.0-20.0	CU-3	Glacial Till	40	37.95	15.67	37.58	15.54	--	
HM-E1	TW-4	16.0-18.0	CU-1	Lacustrine	10	28.02	14.15	17.51	9.86	14.15	
HM-E1	TW-4	16.0-18.0	CU-2	Lacustrine	20	24.91	10.61	20.32	9.50	--	
HM-E1	TW-4	16.0-18.0	CU-3	Lacustrine	40	49.12	24.72	42.07	22.12	--	
RB-C1	TW-4	21.0-22.5	CU-1	Glacial Till	15	97.41	49.94	20.94	15.05	49.94	1
RB-C1	TW-4	21.0-22.5	CU-2	Glacial Till	30	103.29	51.52	55.44	31.85	--	
RB-C7	TW-3	19.5-21.5	CU-1	Glacial Till	24	54.12	29.14	26.31	17.59	--	4
CB-C4	TW-4	17.5-19.5	UU	Glacial Till	15	--	--	--	--	12.47	5

Boring No.	Sample No.	Depth (ft)	Test Type	Soil Type	Confining Pressure (psi)	Maximum Principal Stress Difference Criterion $(\sigma_1' + \sigma_3')/2$ (psi)	Maximum Principal Stress Difference Criterion $(\sigma_1' - \sigma_3')/2$ (psi)	Peak Principal Stress Ratio Criterion $(\sigma_1' + \sigma_3')/2$ (psi)	Peak Principal Stress Ratio Criterion $(\sigma_1' - \sigma_3')/2$ (psi)	Undrained Shear Strength, s_u (psi)	Note
CST-AB1	TW-3	13.5-15.5	UU	Lacustrine	15	--	--	--	--	9.31	5
PS-E1	TW-4	13.5-15.5	UU	Lacustrine	15	--	--	--	--	1.94	5
RB-C9	TW-5	18.0-20.0	UU	Glacial Till	15	--	--	--	--	9.17	5
CB-C4	TW-3	15.0-17.0	UC	Glacial Till	--	--	--	--	--	16.18	
RB-C2	TW-4	15.0-17.0	UC	Glacial Till	--	--	--	--	--	15.97	
RB-C4	TW-4	15.5-17.5	UC	Lacustrine	--	--	--	--	--	3.47	
RW-C1	TW-3	14.5-16.5	UC	Glacial Till	--	--	--	--	--	22.36	
RB-C4	P-9	24.0-26.0	DS-1	Glacial Till	8	--	--	--	--	--	6
RB-C4	P-9	24.0-26.0	DS-2	Glacial Till	15	--	--	--	--	--	6
RB-C4	P-9	24.0-26.0	DS-3	Glacial Till	30	--	--	--	--	--	6

Notes:

1 Only two tests performed due to limited samples.

2. Only two tests performed due to limited samples. Sample for CU-1 was identified as lacustrine clay based on visual description and measured moisture content.

3. Only two tests performed due to limited samples.

4. Only one test performed due to limited samples.

5. Confining Pressure is total confining pressure.

6. Confining Pressure is effective vertical confining pressure applied to sample. Cohesion intercept, c' = 84.9 psf (at peak shear stress) and angle of internal friction, ϕ' = 37 degree (at peak shear stress)

Table 2.5.4-222 Results of Unconfined Compression Tests on Rock Samples (Sheet 1 of 3) [EF3 COL 2.0-29-A]

Boring No.	Run No.	Sample Depth	Rock Unit	Sample Length (L)	Sample Diameter (D)	L/D Ratio	Total Unit Weight	Unconfined Compressive Strength		Elastic Modulus	
		(ft)		(in)	(in)		(pcf)	(psi)	(ksf)	(psi)	(ksf)
CB-C2	22	133.2 - 134.1	Salina F	6.45	3.23	2.0	192.9	5,040	730	3,800,000	547,200
CB-C4[1]	14	91.8 - 92.5	Bass Islands	4.73	2.35	2.0	156.0	13,690	1,970	11,400,000	1,641,600
RB-C1[1]	6	62.1 - 63.2	Bass Islands	4.74	2.40	2.0	151.5	10,830	1,560	7,300,000	1,051,200
RB-C1[1]	10	79.3 - 80.5	Bass Islands	4.56	2.37	1.9	147.7	15,640	2,250	5,000,000	720,000
RB-C1[1]	16	108.0 - 108.7	Bass Islands	4.39	2.30	1.9	150.0	21,450	3,090	7,300,000	1,051,200
RB-C1[1]	19	120.4 - 121.3	Bass Islands	4.56	2.32	2.0	169.5	14,550	2,100	7,800,000	1,123,200
RB-C1[1]	64	251.5 - 254.5	Salina E	4.60	2.34	2.0	149.4	19,140	2,760	5,300,000	763,200
RB-C2[1]	13	71.1 - 72	Bass Islands	4.70	2.37	2.0	162.5	11,050	1,590	5,700,000	820,800
RB-C2[1]	23	124.0 - 124.5	Salina F	3.84	2.38	1.6	196.2	5,960	860	3,000,000	432,000
RB-C2[1]	34	163.8 - 164.6	Salina F	4.48	2.39	1.9	138.4	310	40	1,100,000	158,400
RB-C2[1]	43	187.0 - 188.0	Salina F	4.37	2.40	1.8	137.3	400	60	111,000	16,000
RB-C2[1]	47	207.8 - 208.3	Salina F	4.65	2.40	1.9	137.5	2,980	430	4,300,000	619,200
RB-C2[1]	55	242.2 - 243.2	Salina F	4.63	2.37	2.0	162.4	21,310	3,070	7,500,000	1,080,000
RB-C3[1]	54	234.8 - 235.3	Salina F	4.46	2.30	1.9	156.1	1,660	240	670,000	96,500
RB-C3[1]	56	247.0 - 247.5	Salina E	4.72	2.37	2.0	149.4	8,230	1,190	1,900,000	273,600
RB-C4[1]	10	81.5 - 82.8	Bass Islands	4.59	2.40	1.9	150.7	6,680	960	4,800,000	691,200
RB-C4[1]	18	119.2 - 120.4	Bass Islands	4.73	2.39	2.0	167.7	11,750	1,690	8,100,000	1,166,400
RB-C4[1]	44	194.2 - 195.1	Salina F	4.80	2.39	2.0	158.1	5,180	750	6,400,000	921,600
RB-C4[1]	48	212.2	Salina F	4.71	2.40	2.0	162.2	16,760	2,410	6,700,000	964,800
RB-C5[1]	6	57.0 - 58.3	Bass Islands	4.59	2.41	1.9	147.8	14,360	2,070	7,400,000	1,065,600

Table 2.5.4-222 Results of Unconfined Compression Tests on Rock Samples (Sheet 2 of 3) [EF3 COL 2.0-29-A]

Boring No.	Run No.	Sample Depth (ft)	Rock Unit	Sample Length (L) (in)	Sample Diameter (D) (in)	L/D Ratio	Total Unit Weight (pcf)	Unconfined Compressive Strength (psi)	Unconfined Compressive Strength (ksf)	Elastic Modulus (psi)	Elastic Modulus (ksf)
RB-C5(1)	12	89.5 - 90.5	Bass Islands	4.69	2.40	2.0	154.8	14,680	2,110	9,300,000	1,339,200
RB-C5(1)	44	213.0 - 214.0	Salina F	4.52	2.40	1.9	142.2	10,940	1,580	1,000,000	144,000
RB-C5(1)	45	219.8 - 220.5	Salina F	4.85	2.39	2.0	145.2	5,840	840	6,700,000	964,800
RB-C7(1)	7	50.0 - 51.9	Bass Islands	4.60	2.39	1.9	124.9	8,630	1,240	2,300,000	331,200
RB-C7(1)	17	101.7 - 102.4	Bass Islands	4.59	2.39	1.9	152.8	19,170	2,760	5,300,000	763,200
RB-C8(1)	11	75.5 - 84.4	Bass Islands	6.34	3.25	2.0	152.5	11,130	1,600	7,000,000	1,008,000
RB-C8(1)	43	240.1 - 240.9	Salina E	6.30	3.24	1.9	140.4	17,750	2,560	5,400,000	777,600
RB-C8(1)	57	306.8 - 308.0	Salina E	5.45	3.28	1.7	142.6	3,110	450	3,500,000	504,000
RB-C8(1)	63	338.8 - 340.1	Salina C	6.35	3.26	1.9	166.6	9,670	1,390	5,900,000	849,600
RB-C8(1)	80	424.7 - 426.7	Salina B	6.00	3.27	1.8	145.4	7,840	1,130	10,000,000	1,440,000
RB-C8(1)	85	441.7 - 451.1	Salina B	6.37	3.28	1.9	170.5	13,480	1,940	10,900,000	1,569,600
RB-C9(1)	18	114.5 - 114.9	Salina F	4.80	2.40	2.0	159.0	6,390	920	1,700,000	244,800
RB-C11(1)	9	70.5 - 71.1	Bass Islands	4.84	2.40	2.0	140.9	8,690	1,250	2,500,000	360,000
RB-C11(1)	13	93.3 - 94.3	Bass Islands	4.76	2.40	2.0	133.8	15,100	2,170	4,710,000	678,200
RW-C1	6	55.5 - 56.2	Bass Islands	4.62	2.40	1.9	157.0	10,830	1,560	5,600,000	806,400
RW-C1	9	68.0 - 69.0	Bass Islands	4.73	2.40	2.0	144.3	10,370	1,490	5,600,000	806,400
RW-C1	11	80.6 - 81.2	Bass Islands	4.76	2.40	2.0	159.7	10,940	1,580	6,000,000	864,000
RW-C1	54	261.2 - 262.0	Salina E	5.03	2.40	2.1	153.4	12,310	1,770	5,600,000	806,400
TB-C5	6	49.3 - 50.3	Bass Islands	6.55	3.23	2.0	140.7	7,800	1,120	3,700,000	532,800
TB-C5	17	98.4 - 99.3	Bass Islands	6.62	3.27	2.0	164.4	22,320	3,210	8,000,000	1,152,000

Table 2.5.4-222 Results of Unconfined Compression Tests on Rock Samples (Sheet 3 of 3) [EF3 COL 2.0-29-A]

Boring No.	Run No.	Sample Depth (ft)	Rock Unit	Sample Length (L) (in)	Sample Diameter (D) (in)	L/D Ratio	Total Unit Weight (pcf)	Unconfined Compressive Strength (psi)	Unconfined Compressive Strength (ksf)	Elastic Modulus (psi)	Elastic Modulus (ksf)
TB-C5	41	219.1 - 220.0	Salina F	6.53	3.26	2.0	149.6	1,910	280	4,800,000	691,200
TB-C5	59	304.0 - 305.0	Salina E	6.52	3.25	2.0	166.2	12,020	1,730	9,300,000	1,339,200
TB-C5	62	319.0 - 320.0	Salina E	6.35	3.27	1.9	149.7	6,690	960	3,600,000	518,400
TB-C5	67	343.4 - 344.3	Salina C	6.33	3.25	1.9	167.1	15,250	2,200	4,700,000	676,800

ft = feet
in = inches
pcf = pounds per cubic foot
psi = pounds per square inch
ksf = kips per square foot
L/D = length to diameter ratio

Notes:
(1) Sample obtained close to or below the base of the safety-related structure

Table 2.5.4-223 Results of Direct Shear Tests on Rock Discontinuities [EF3 COL 2.0-29-A]

					Residual Shear Stress	
		Sample Depth		Normal Stress	Cohesion Intercept, c’	Frcition Angle, ϕ'
Boring No.	Run No.	(ft)	Rock Unit	(psf)	(psi)	(degree)
CB-C2	2	33.4	Bass Islands	2,880	0	47.7
CB-C2	9	69.0	Bass Islands	5,760	0	37.8
CB-C4	4	44.5	Bass Islands	3,600	0	53.7
CB-C4	6	57.0	Bass Islands	5,040	0	63.1
RB-C3	3	46.9	Bass Islands	4,320	0	65.9
RB-C4	2	43.0	Bass Islands	3,600	0	32.6
RB-C4	4	49.7	Bass Islands	4,320	0	47.7
RB-C4	6	60.1	Bass Islands	5,040	0	55.5
RB-C9	5	53.3	Bass Islands	4,320	0	54.5
RB-C9	6	59.3	Bass Islands	5,040	0	73.9
RB-C9	10	73.7	Bass Islands	6,480	0	48.6
RB-C11	2	36.6	Bass Islands	2,880	0	38.7

ft = feet
in = inches
psf = pounds per square foot
psi = pounds per square inch

Table 2.5.4-224 Foundation Elevations of Major Structures in the Power Block Area
[EF3 COL 2.0-29-A]

Building	Structure Category(1)	Final Surface Grade Elevation in NAVD 88(2) (feet)	Bottom of Foundation Elevation in NAVD 88 (feet)	Depth of Foundation(3) (feet)
Reactor Building/Fuel Building (RB/FB)	I	589.3	523.7	65.6(3)
Control Building (CB)	I	589.3	540.4	48.9(3)
Firewater Service Complex (FWSC)	I	589.3	581.6	7.7(3)
Radwaste Building (RW)	NS	589.3	537.3	52
Turbine Building (TB)	II	589.3	563.4	25.9
Service Building (SB)	II	589.3	573.9	15.4

Note:

1. Information from DCD Table 3.2-1.
2. Information from Subsection 2.4.1.
3. Information from DCD Table 3.8-13.

I - Seismic Category I
II - Seismic Category II
NS - Nonseismic

Table 2.5.4-225 Locations, Logging Methods, and Depth Ranges for Geophysical Surveys Performed to obtain the Dynamic Characteristics of Soils and Rocks (Sheet 1 of 2) [EF3 COL 2.0-29-A]

Boring No.	Geophysical Method	Depth Range where Measurements Were Obtained (ft)	Sample Interval (ft)	Depth to Bottom of Casing[1] (ft)	Remarks
CB-C3	P-S Suspension	36 – 203	1.6	36.0	P-S Suspension – entire borehole. Downhole Seismic – no measurements between 125 and 205 feet.
		198 – 256	1.6		
	Downhole Seismic	37.5 – 125	2.5 – 5.0		
		205 – 250	5.0		
RB-C4	P-S Suspension	34 – 100	1.6	34.7	P-S Suspension – no measurements between 100 and 194 feet. Downhole Seismic – no measurements between 113 and 195 feet for both V_p & V_s; no measurements between 35 and 105 for V_s.
		194 – 251	1.6		
	Downhole Seismic	(P-wave) 35 – 113 (S-wave) 105 – 113	5		
		195 – 260	5		
RB-C8	P-S Suspension	31 – 118	1.6	29.5	P-S Suspension – no measurements between 125 and 205 feet. Downhole Seismic – no measurements between 125 and 205 feet.
		210 – 276	1.6		
		269 – 450	1.6		
	Downhole Seismic	31 – 110	5.0		
		210 – 270	5.0		
		270 – 435	5.0		
		279 – 455.5	1.6		
TB-C5	P-S Suspension	27.9 – 285	1.6	29.0	P-S Suspension – entire borehole. Downhole Seismic – only P-wave measurements between 280 and 325 feet.
		279 – 455.5	1.6		
	Downhole Seismic	(P-wave) 280 – 325	5.0		

Table 2.5.4-225 Locations, Logging Methods, and Depth Ranges for Geophysical Surveys Performed to obtain the Dynamic Characteristics of Soils and Rocks (Sheet 2 of 2)

[EF3 COL 2.0-29-A]

Boring No.	Geophysical Method	Depth Range where Measurements Were Obtained (ft)	Sample Interval (ft)	Depth to Bottom of Casing(1) (ft)	Remarks
RB-C6	P-S Suspension	11.5 – 36	1.6	33.7	P-S Suspension – within overburden only.
RB-C4	SASW	0 – 30	--	--	SASW survey – within overburden only.
RW-C1	SASW	0 – 30	--	--	SASW survey – within overburden only.
MW-381S	SASW	0 – 30	--	--	SASW survey – within overburden only.
MW-393	SASW	0 – 30	--	--	SASW survey – within overburden only.

Notes:

1. Steel casing was installed to prevent soils from collapsing into borehole. No P-S Suspension and Downhole Seismic Loggings were performed in overburden except in Boring RB-C6.

ft = feet

Table 2.5.4-226 Summary of Building Dimensions, Depths of Foundation Level and Loadings in the Power Block Area [EF3 COL 2.0-29-A]

Structures	Approximate Dimension (ft)	Depth of Foundation (ft)	Loading (ksf)
Reactor Building/Fuel Building (RB/FB)	230 X 161	65.6	14.6
Control Building (CB)	99 X 78	48.9	6.1
FWS Complex (FWSC)	171 X 66	7.7	3.45
Turbine Building (TB)	380 X 200	25.9	6.0
Radwaste Building (RW)	217 X 111	52.0	6.0
Service Building (SB)	163 X 111	15.4	4.0
Electrical Building/ Technical Support Center (EB/TSC)	255 X 144	5.0[(1)]	1.0[(1)]
Hot Machine Shop (HMS)	137 X 97	5.0[(1)]	1.0[(1)]
Ancillary Diesel Building (ADB)	71 x 61	5.0[(1)]	4.0

Notes: The dimensions are rounded to the nearest 1.0 ft, referenced from Final Surface Grade Elevation.

1. Assumed values.

ft = feet
ksf = kips per square foot

Table 2.5.4-227 Results of Bearing Capacity Analysis [EF3 COL 2.0-29-A]

Structure	Terzaghi Approach: Bearing Capacity: Ultimate (ksf)	Terzaghi Approach: Bearing Capacity: Allowable Under Static Loading Condition(1) (ksf)	Terzaghi Approach: Bearing Capacity: Allowable Under Dynamic Loading Condition(2) (ksf)	Uniform Building Code: Allowable Loading Condition(3) (ksf)	Required Maximum Bearing Demand: From Referenced DCD: Static Loading Condition(4) (ksf)	Required Maximum Bearing Demand: From Referenced DCD: Dynamic Loading Condition(5) (ksf)	Required Maximum Bearing Demand: From Fermi 3 Site-Specific SSI: Dynamic Loading Condition(6) (ksf)
Reactor/Fuel Building	281	94	125	259	14.6	23.0	26.4
Control Building	879	293	391	374	6.1	8.8	11.3
Firewater Service Complex	96	32	43	43	3.45	25.1	N/A

Note:

1. Allowable static bearing capacity using factor of safety of 3.
2. Allowable dynamic bearing capacity using factor of safety of 2.25.
3. Method 2 only allowed determination of allowable bearing capacity under static loading condition.
4. Criterion from Referenced DCD; (1) and (3) were used to check against (4); (1) and (3) are greater than (4), therefore satisfy the Referenced DCD criterion.
5. Criterion from Referenced DCD; (2) was used to check against (5); (2) is greater than (5), therefore satisfies the Referenced DCD criterion.
6. Fermi 3 site-specific SSI; (2) was used to check against (6); (2) is greater than (6), therefore satisfies Fermi 3 site-specific SSI dynamic loading condition.

ksf = kips per square foot

N/A = Not Applicable (No Site-Specific SSI analysis performed for the FWSC)

Table 2.5.4-228 Summary of Modulus of Elasticity of Bedrock Units based on Test Results, and Hoek-Brown Criterion

[EF3 COL 2.0-29-A]

Rock Unit		Average Modulus of Elasticity based on Laboratory Test (ksf)	Elastic Modulus of Elasticity based on Average V_s(2) (ksf)	Elastic Modulus based on Hoek-Brown Criterion: Upper Bound (ksf)	Elastic Modulus based on Hoek-Brown Criterion: Mean (ksf)	Elastic Modulus based on Hoek-Brown Criterion: Lower Bound (ksf)	Average Modulus of Elasticity based on Pressuremeter Test (ksf)
Bass Island Group		898,600	556,200	109,500	80,700	59,900	Not Measured
Salina Group	Unit F	529,200	132,600	31,700	24,200	19,300	20,800(3)
	Unit E	671,500	755,800	492,100	424,200	349,000	Not Measured
	Unit C	763,200(1)	1,007,600	623,000	559,300	482,100	
	Unit B	1,504,800(1)	1,156,900	1,324,700	1,228,400	1,102,700	

Notes: All Modulus values listed above are rounded to the nearest 100 ksf.

1. The calculated elastic moduli are based on mean V_s in Boring TB-C5 measured using P-S Suspension Logger.
2. Based on two unconfined compression tests performed.
3. The elastic modulus is based on average of five pressuremeter tests performed within Salina Group Unit F in Boring RB-C6.

ksf = kips per square foot

Table 2.5.4-229 Selected Parameters for Linear Elastic Model used for Settlement Analysis [EF3 COL 2.0-29-A]

Material		Elevation to Top of Layer (NAVD 88)[2] (ft)	Elastic Modulus for Settlement Analysis: Upper Bound (ksf)	Elastic Modulus for Settlement Analysis: Lower Bound (ksf)	Poisson's Ratio	Saturated Unit Weight (pcf)	Unsaturated Unit Weight (pcf)
Lean Concrete[1]		--	142,200	142,200	0.20	145	145
Bass Island Group		550	556,200	59,900	0.33	150	150
Salina Group	Unit F	460	132,600	19,300	0.39	150	150
	Unit E	340	671,500	349,000	0.30	150	150
	Unit C	250	763,200	482,100	0.28	160	160
	Unit B	160	1,156,900	1,102,700	0.29	160	160

Notes:

1. The elastic modulus of concrete is calculated using $E_{concrete}$ (psi) = 57,000 $f'_c{}^{1/2}$ and by using f'_c = 300 psi (reduced compressive strength for lean concrete).
2. Finished grade is assumed at El. 589.3 feet (NAVD 88).

ft = feet
ksf = kips per square foot
Pcf = pounds per cubic foot

Table 2.5.4-230 Calculated Rebound at Seismic Category I Structures due to Excavation to Foundation Level [EF3 COL 2.0-29-A]

	Rebound Due to Excavation at Foundation Corners and Center (inch)				
Building	**Northwest Corner**	**Southwest Corner**	**Southeast Corner**	**Northeast Corner**	**Center or close to Center(2)**
Reactor Building/Fuel Building	0.31	0.25	0.31	0.32	0.43
Control Building	0.33	0.35	0.29	0.28	0.34
Firewater Service Complex	0.26(1)	0.26(1)	0.21(1)	0.21(1)	0.24(1)

Notes: All values listed above are rounded to the nearest 0.01 inch.

1. The foundation soil under the FWSC will be removed to top of bedrock; therefore, rebound was estimated at the top of bedrock (Bass Islands Group) during excavation stage.
2. Nodes generated in the mesh may not be exactly at the center of the foundation.

Table 2.5.4-231 Calculated Total Settlements due to Backfilling and Applied Loads for Seismic Category I Structures [EF3 COL 2.0-29-A]

Building	Total(2) Settlements due to backfilling and applied loads at corners and center (inch)					
	Northwest Corner	Southwest Left Corner	Southeast Corner	Northeast Corner	Average of Four Corners	Center or close to Center(1)
Reactor Building/Fuel Building	0.47	0.42	0.52	0.51	0.48	0.75
Control Building	0.51	0.56	0.41	0.39	0.47	0.47
Firewater Service Complex	0.16	0.18	0.12	0.11	0.14	0.15

Notes: All values listed above are rounded to the nearest 0.01 inch.

1. Nodes generated in the mesh may not be exactly at the center of the foundation.
2. Total settlement equals calculated settlement due to applied structure from the rebounded position

Table 2.5.4-232 Comparing Acceptance Criteria in Referenced DCD [EF3 COL 2.0-29-A]

	Finite Element Model (FEM)		Acceptance Settlement in Referenced DCD / Calculated Settlement from FEM			
	Maximum Settlement at any point in Basemat	Minimum Settlement at any point in Basemat	Maximum Settlement at any Corner of Basemat	Average Settlement at Four Corners of Basemat	Maximum Differential Settlement along Longest Mat Foundation Dimension	Maximum Differential Displacement between Reactor and Control Building
Building	**(inch)**	**(inch)**	**(inch)**	**(inch)**	**(inch)**	**(inch)**
Reactor Building/Fuel Building	0.76	0.42	4.0 / 0.52(1)	2.6 / 0.48(1)	3.0 / 0.34(2)	3.3 / 0.37(3)
Control Building	0.56	0.39	0.7 / 0.56(1)	0.5 / 0.47(1)	0.6 / 0.17(2)	
Firewater Service Complex	0.18	0.11	0.7 / 0.18(1)	0.4 / 0.14(1)	0.5 / 0.07(2)	NA

Notes: All values listed above are rounded to the nearest 0.01 inch.

1. The calculated FEM settlements are obtained from Table 2.5.4-231.
2. The FEM differential settlement is obtained from (column 2 – column 1) in this Table. This is conservative since it is the maximum differential settlement at the basemat.
3. The value is based on the column 1 in the Reactor Building/Fuel Building row – column 2 in the Control Building row. This is conservative since it is the maximum differential settlement between these buildings.

Figure 2.5.4-201 Excavation Site Plan [EF3 COL 2.0-29-A]

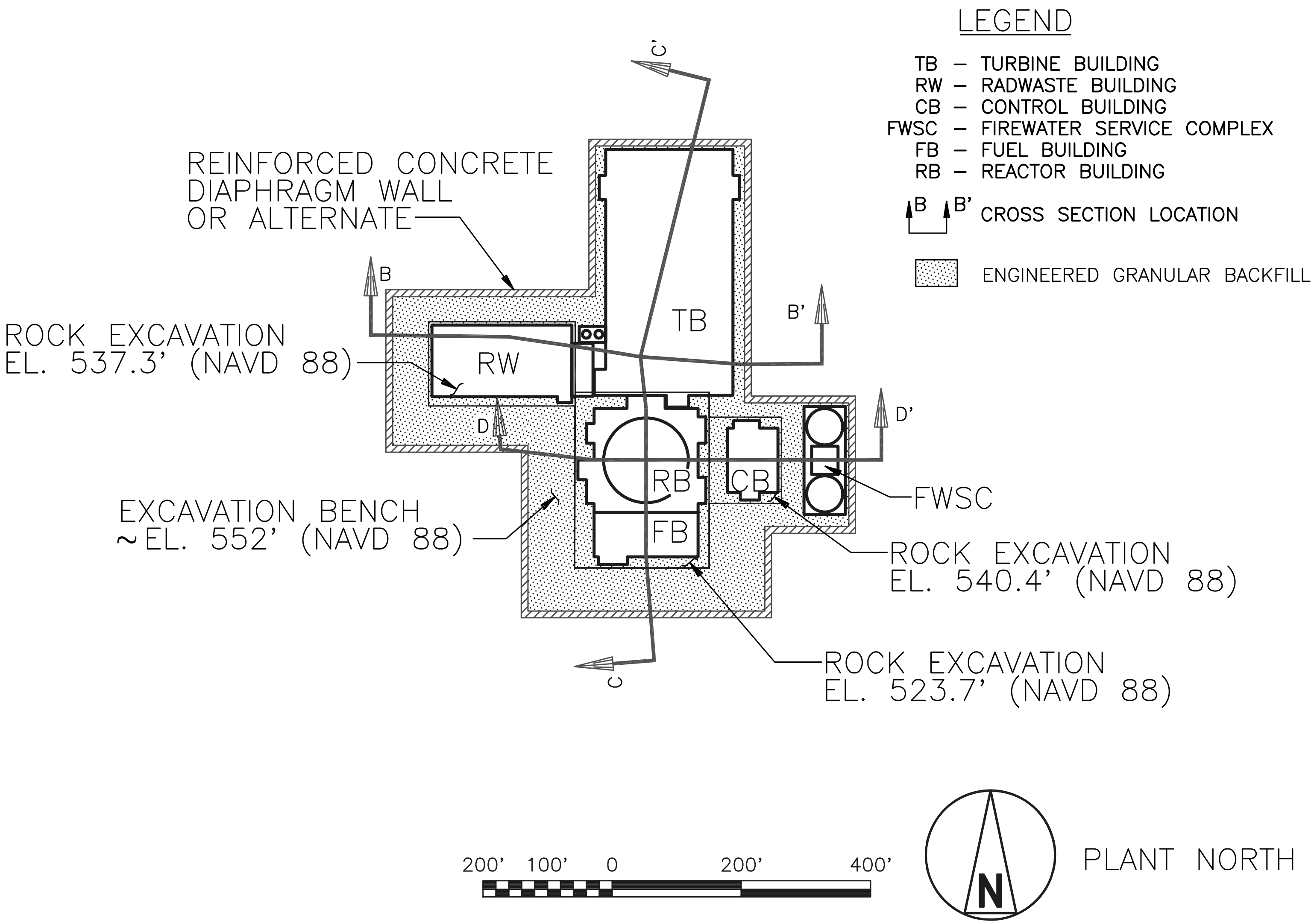


Figure 2.5.4-202 Excavation Cross Section D-D' [EF3 COL 2.0-29-A]

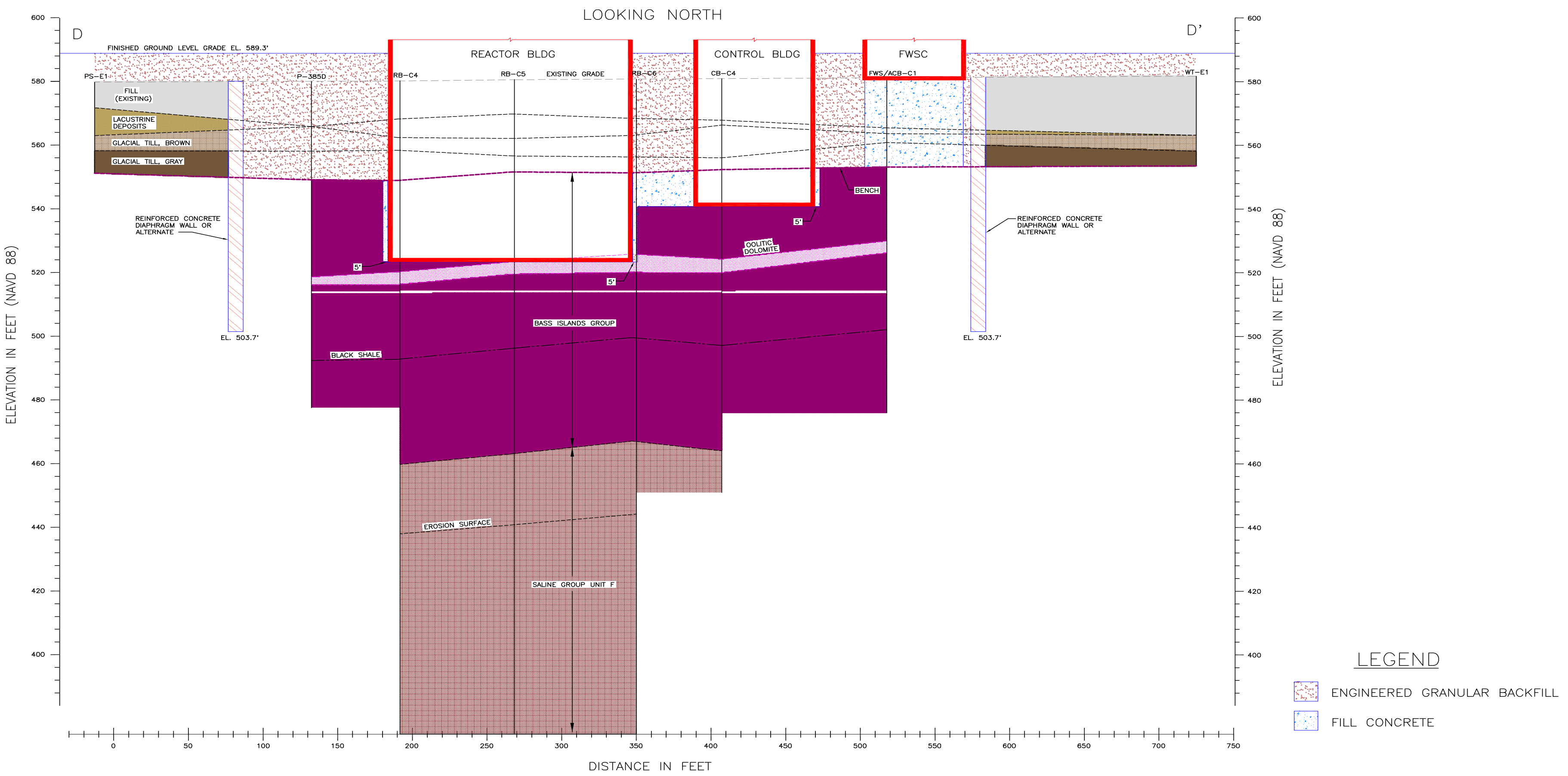
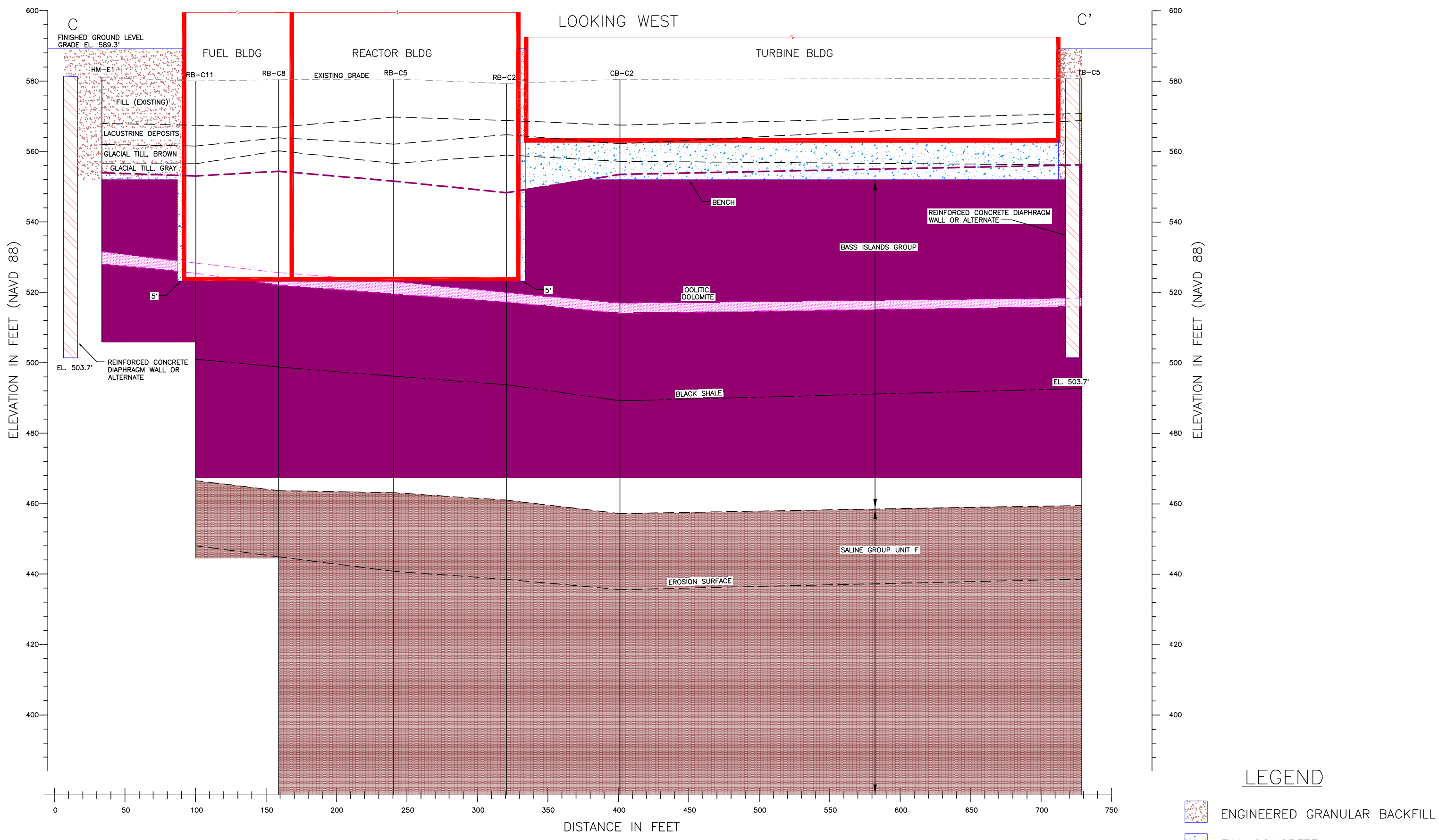


Figure 2.5.4-203 Excavation Cross Section C-C' [EF3 COL 2.0-29-A]



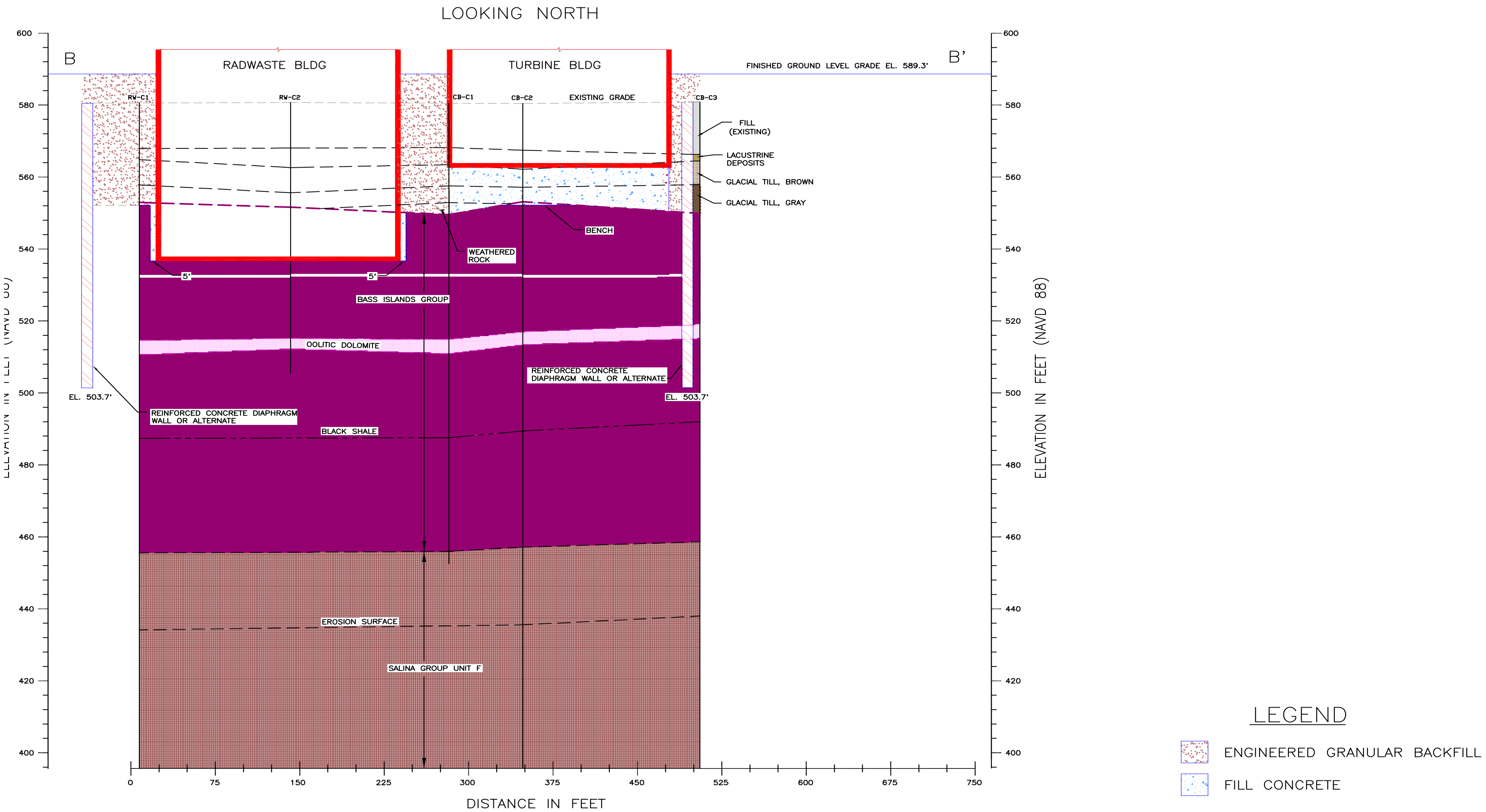

LOOKING NORTH
B
B'
RADWASTE BLDG
TURBINE BLDG
FINISHED GROUND LEVEL GRADE EL. 589.3'
EXISTING GRADE
RW-C1
RW-C2
CB-C1
CB-C2
CB-C3
FILL (EXISTING)
LACUSTRINE DEPOSITS
GLACIAL TILL, BROWN
GLACIAL TILL, GRAY
BENCH
WEATHERED ROCK
5'
5'
BASS ISLANDS GROUP
OOLITIC DOLOMITE
REINFORCED CONCRETE DIAPHRAGM WALL OR ALTERNATE
REINFORCED CONCRETE DIAPHRAGM WALL OR ALTERNATE
EL. 503.7'
EL. 503.7'
BLACK SHALE
EROSION SURFACE
SALINA GROUP UNIT F
ELEVATION IN FEET (NAVD 88)
ELEVATION IN FEET (NAVD 88)
DISTANCE IN FEET
LEGEND
ENGINEERED GRANULAR BACKFILL
FILL CONCRETE

Figure 2.5.4-205 Comparison of measured V_s and V_p with RQD for Boring TB-C5
[EF3 COL 2.0-29-A]

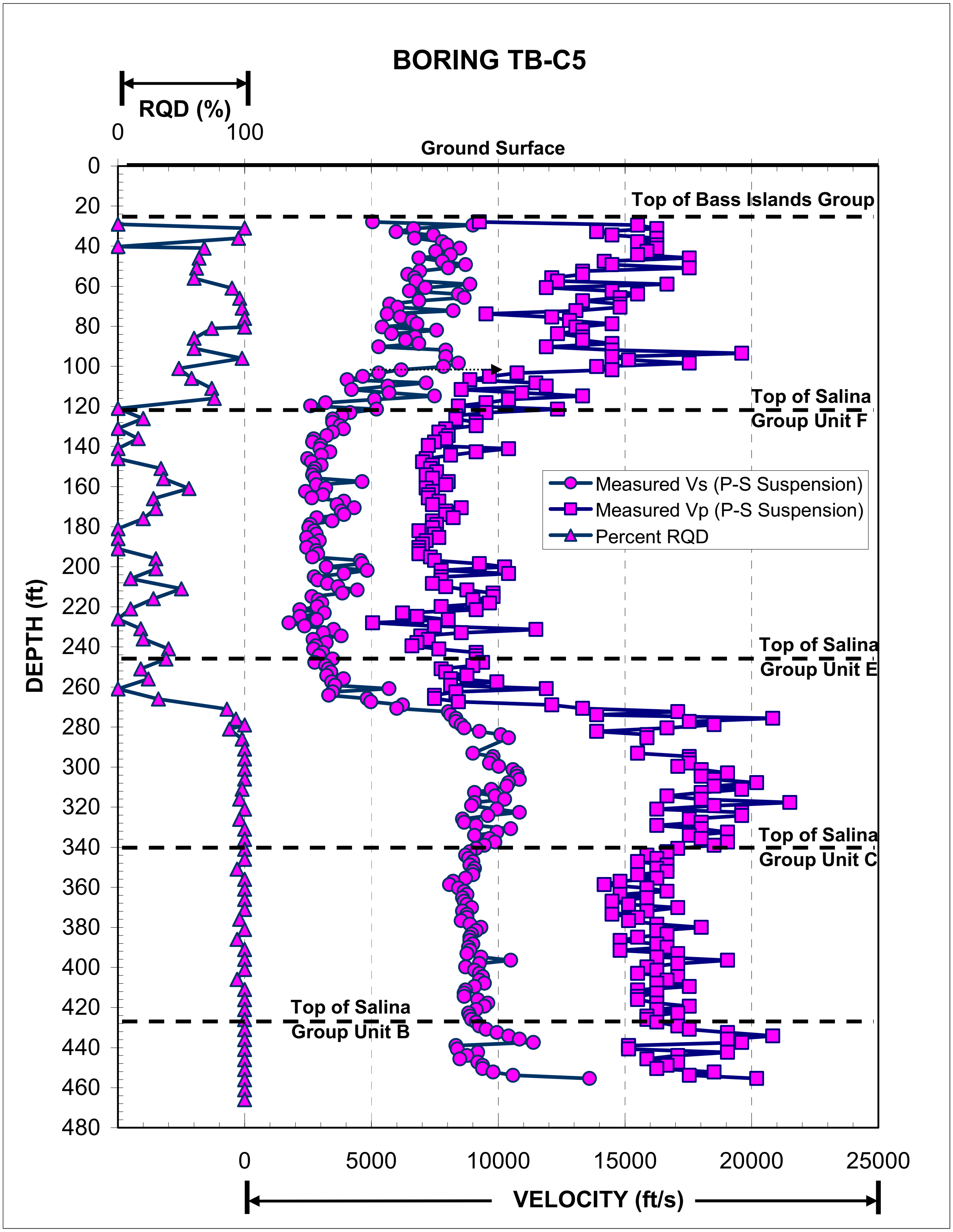


Figure 2.5.4-206 Comparison of measured V_s and V_p with RQD for Boring RB-C8
[EF3 COL 2.0-29-A]

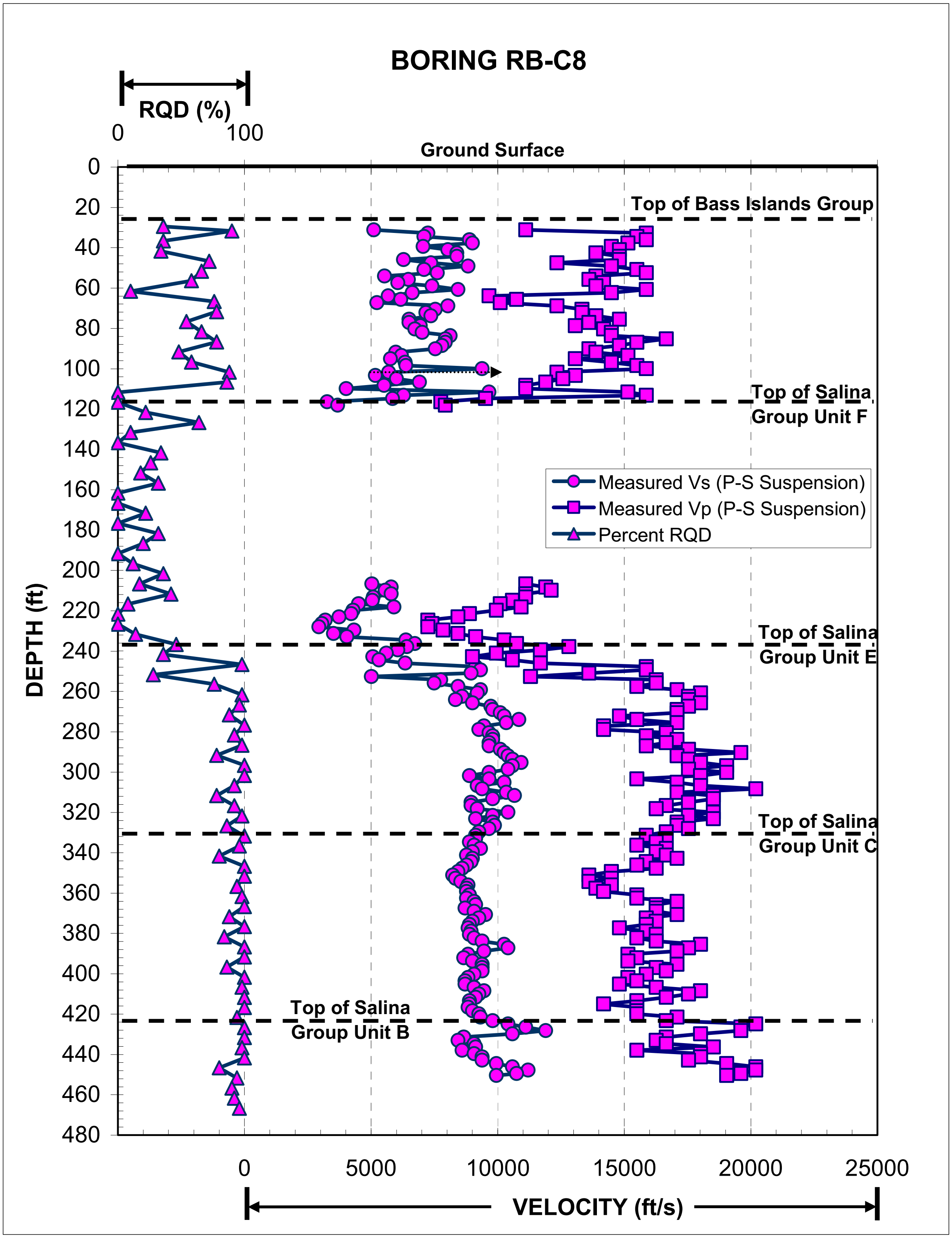


Figure 2.5.4-207 Comparison of measured V_s and V_p with RQD for Boring CB-C3
[EF3 COL 2.0-29-A]

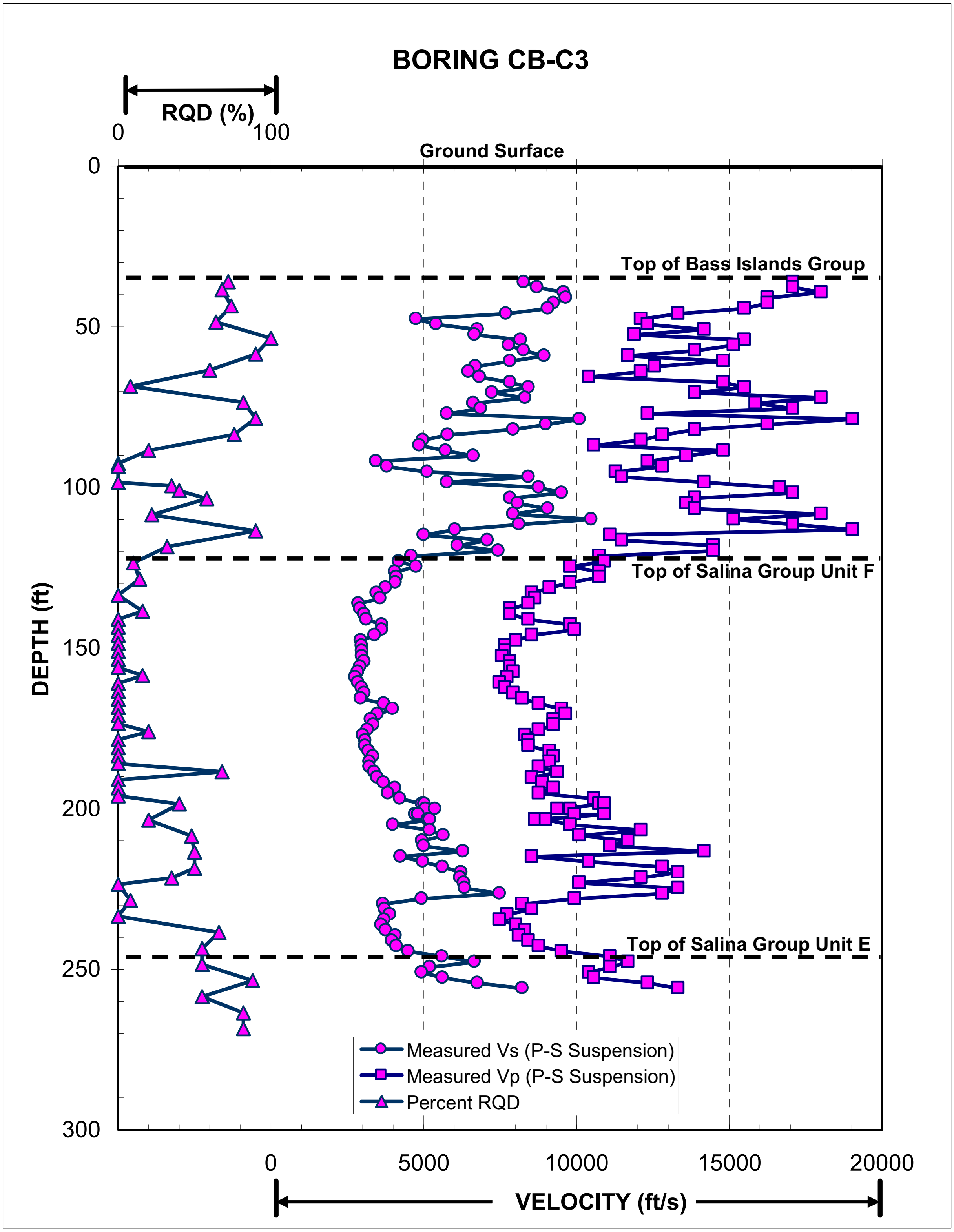


Figure 2.5.4-208 Comparison of measured V_s and V_p with RQD for Boring RB-C4
[EF3 COL 2.0-29-A]

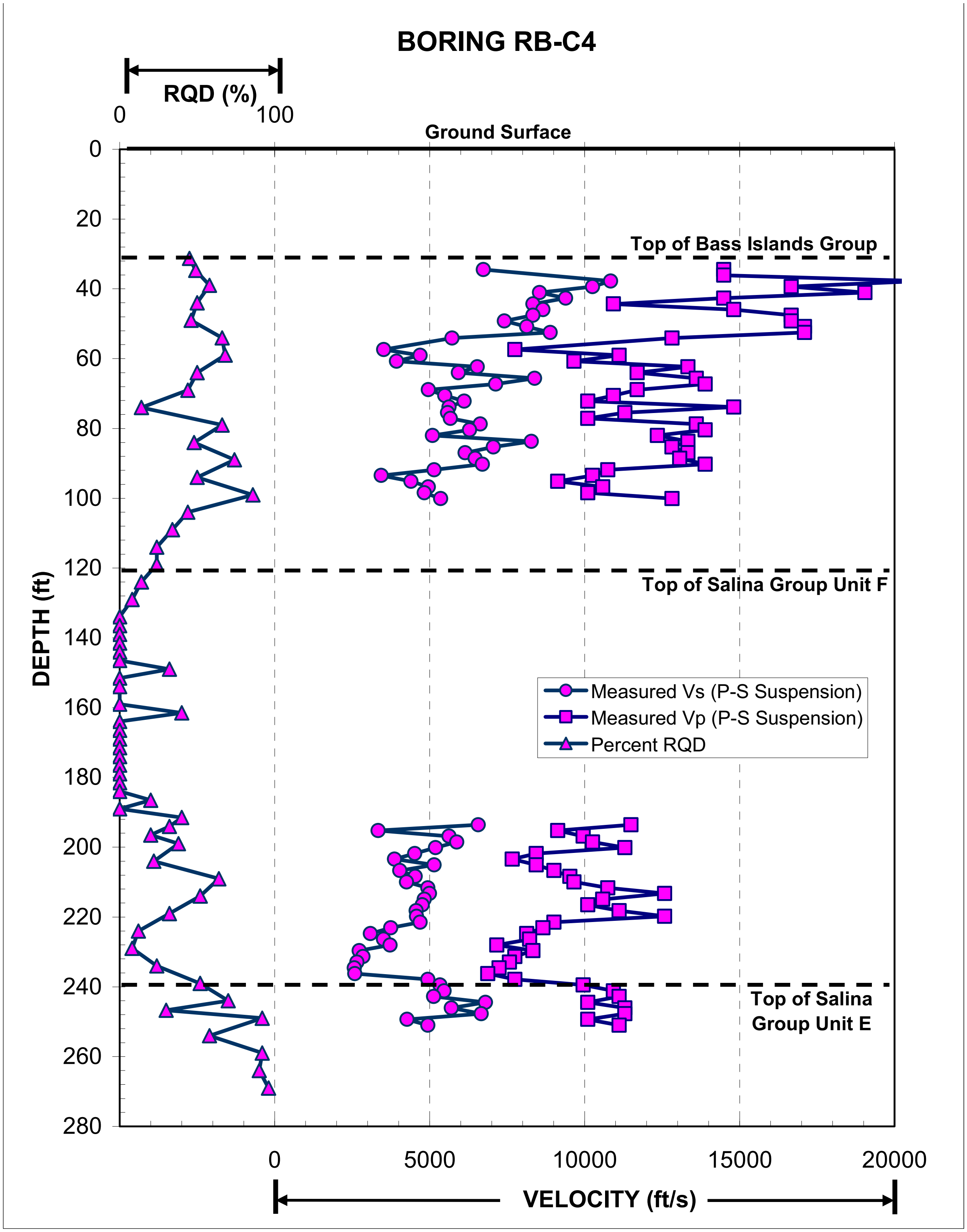


Figure 2.5.4-209 Influence of geologic features within Bass Islands Group on measured seismic wave velocities in Borehole TB-C5 [EF3 COL 2.0-29-A]

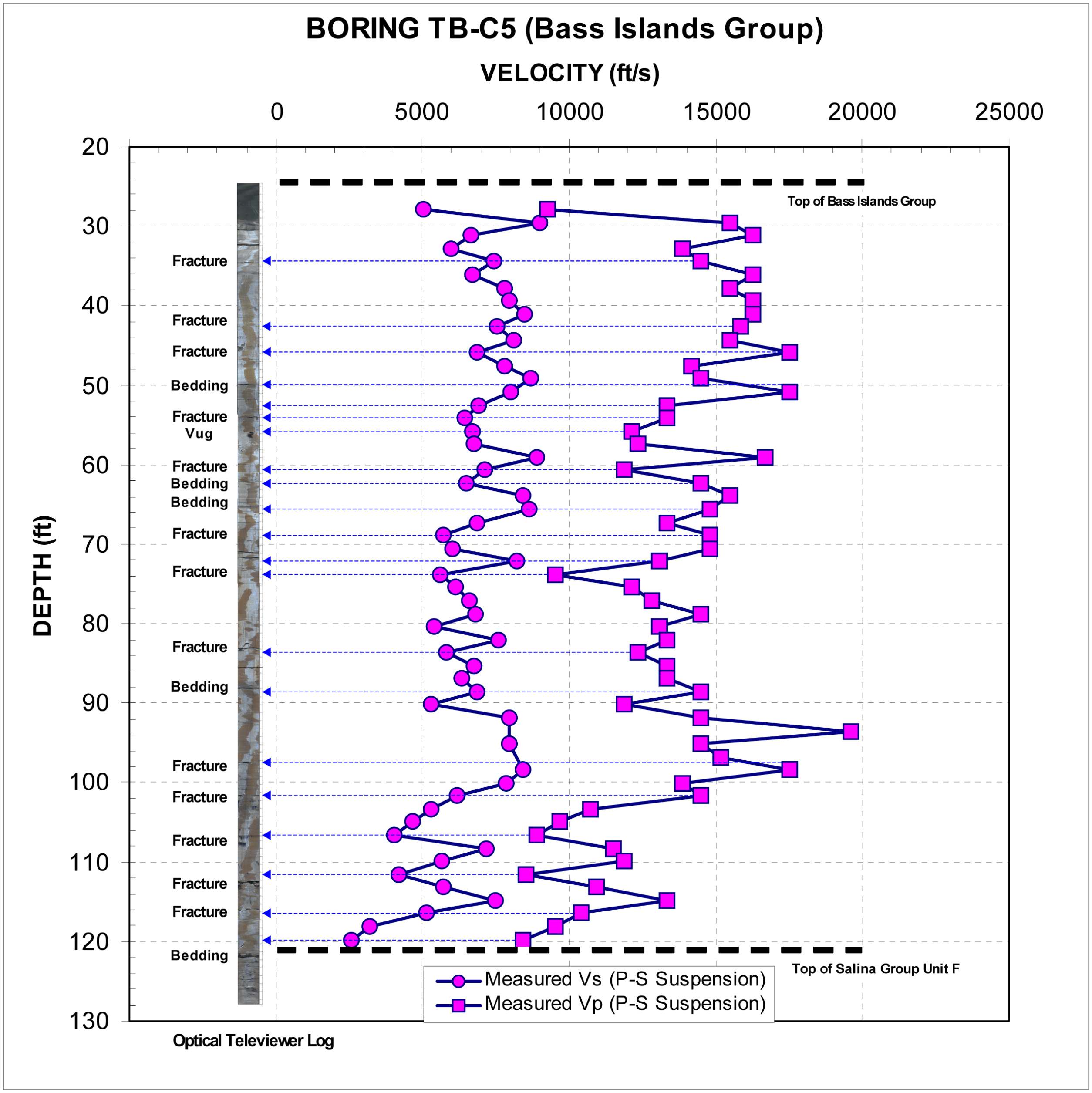


Figure 2.5.4-210 Influence of geologic features within Bass Islands Group on measured seismic wave velocities in Borehole RB-C8 [EF3 COL 2.0-29-A]

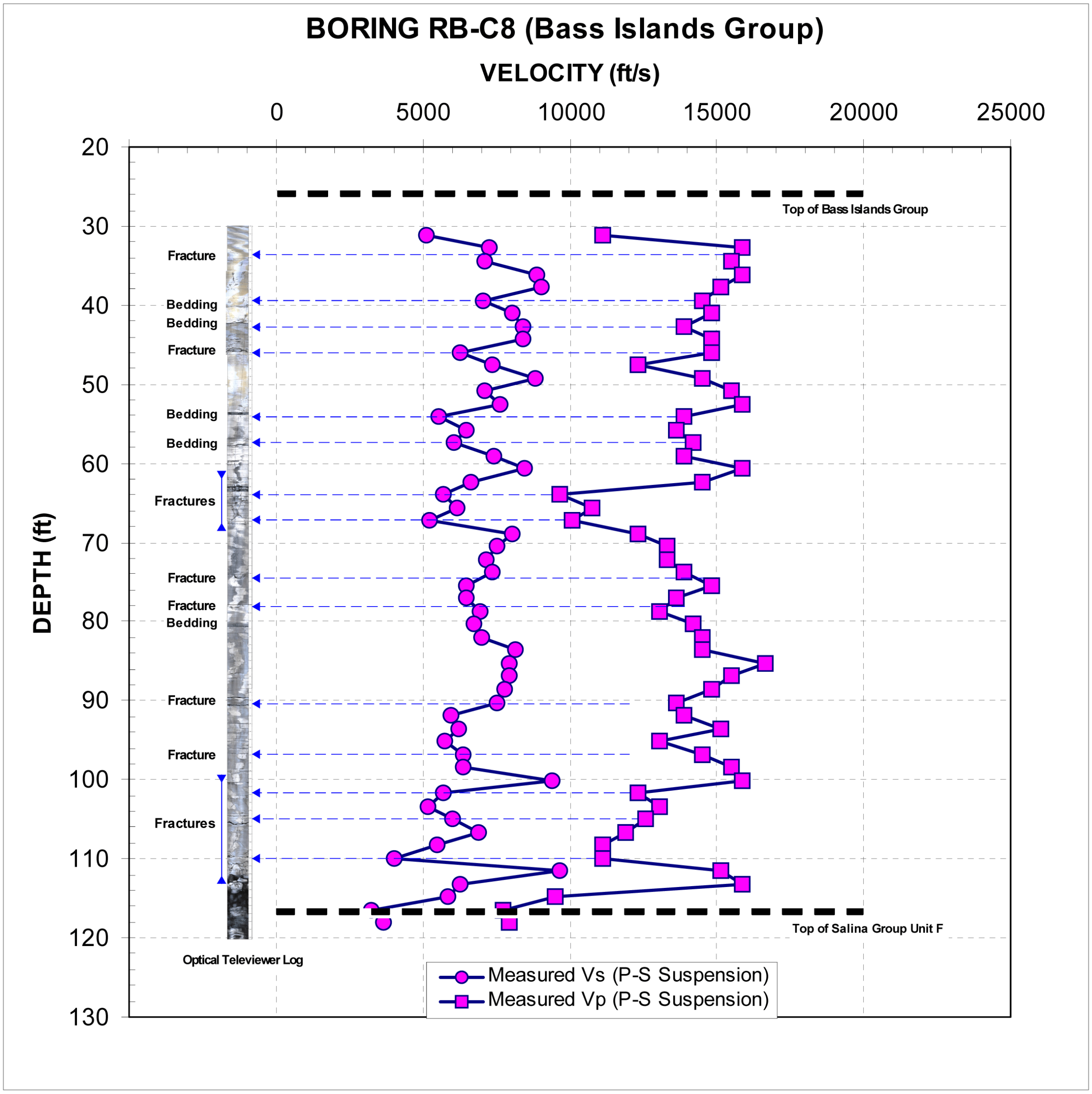


Figure 2.5.4-211 Influence of geologic features within Bass Islands Group on measured seismic wave velocities in Borehole CB-C3 [EF3 COL 2.0-29-A]

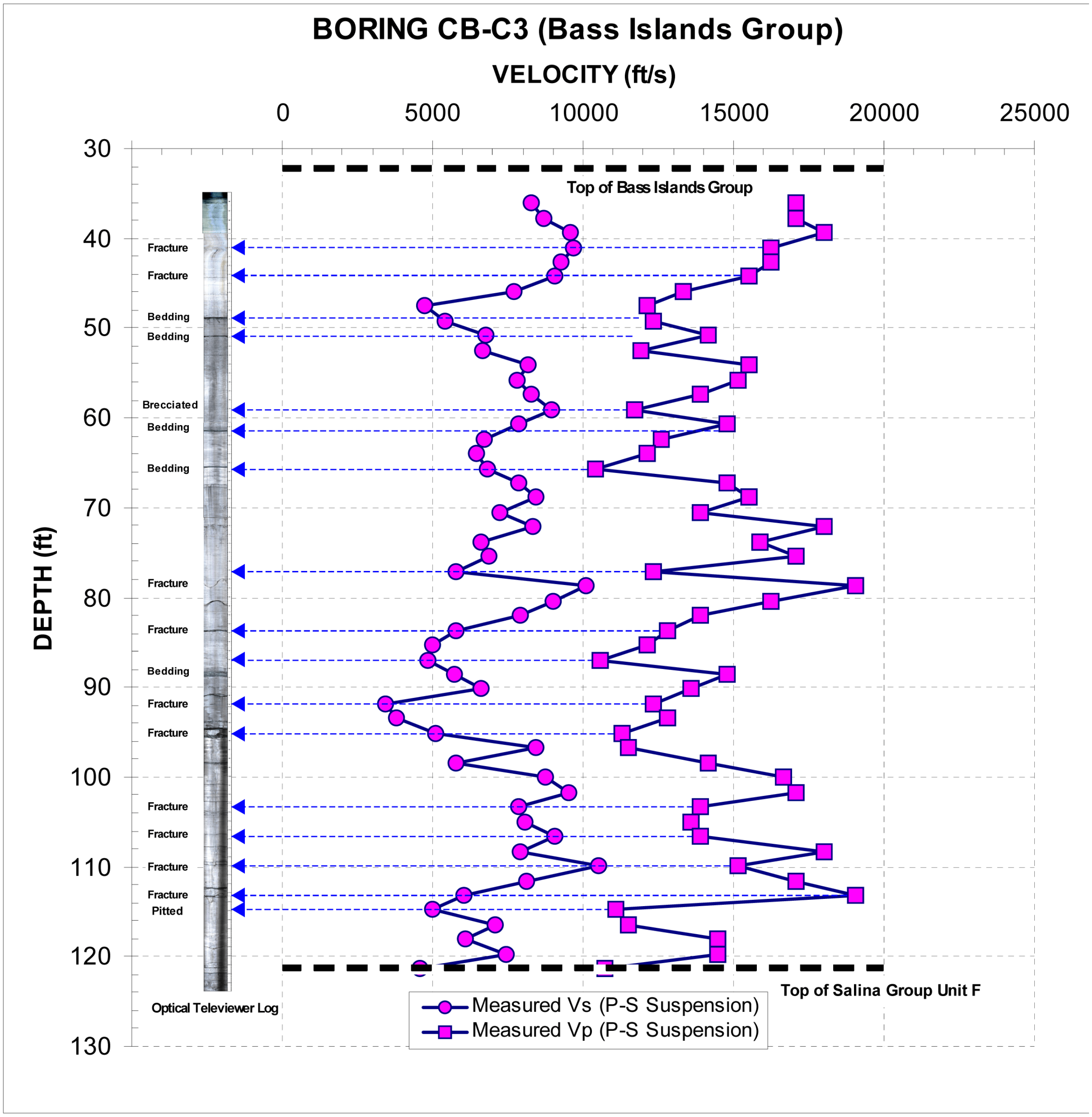


Figure 2.5.4-212 Influence of geologic features within Bass Islands Group on measured seismic wave velocities in Borehole RB-C4 [EF3 COL 2.0-29-A]

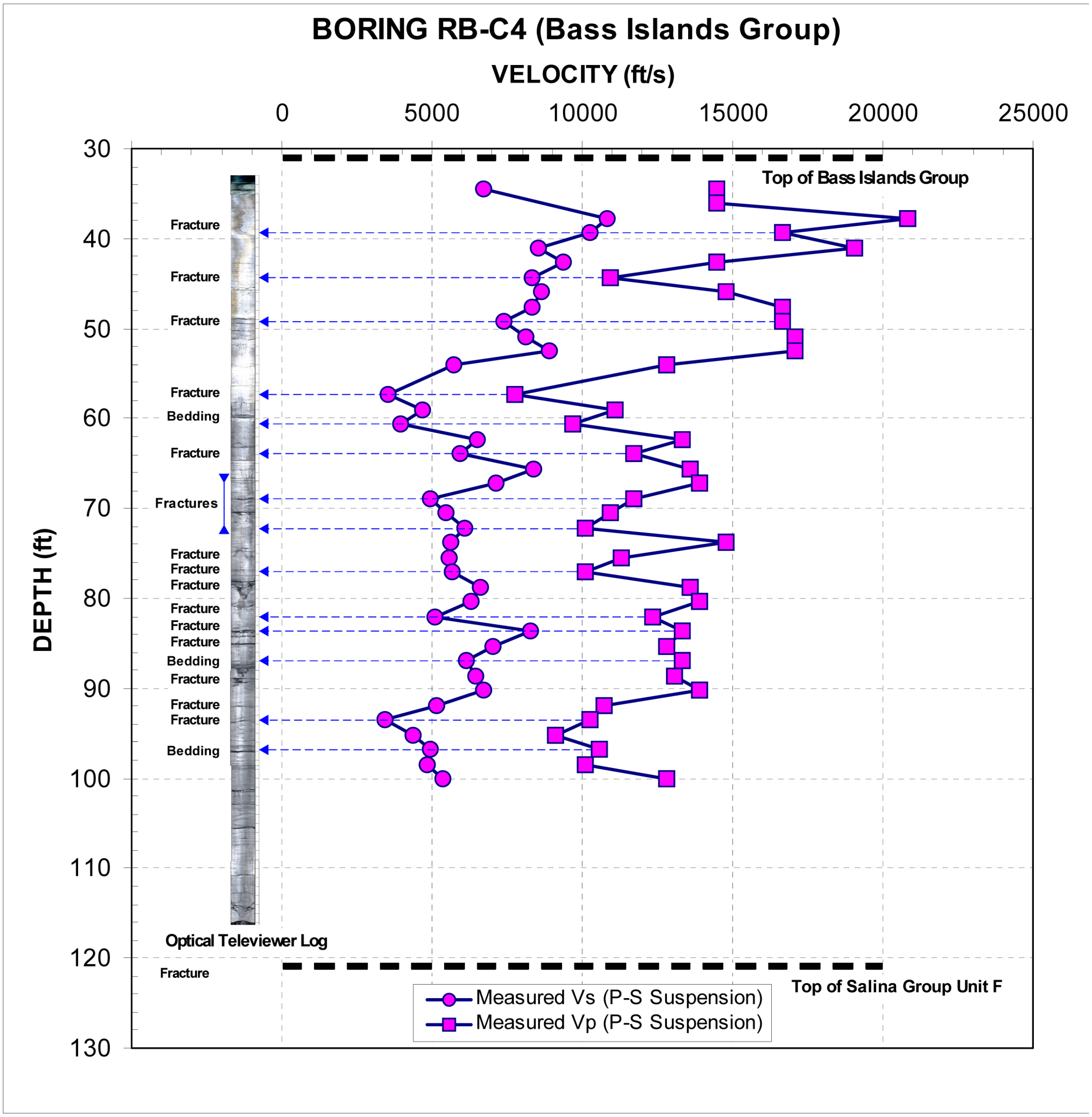


Figure 2.5.4-213 Influence of shale or claystone content within Salina Group Unit F on measured seismic wave velocities in Boring TB-C5 [EF3 COL 2.0-29-A]

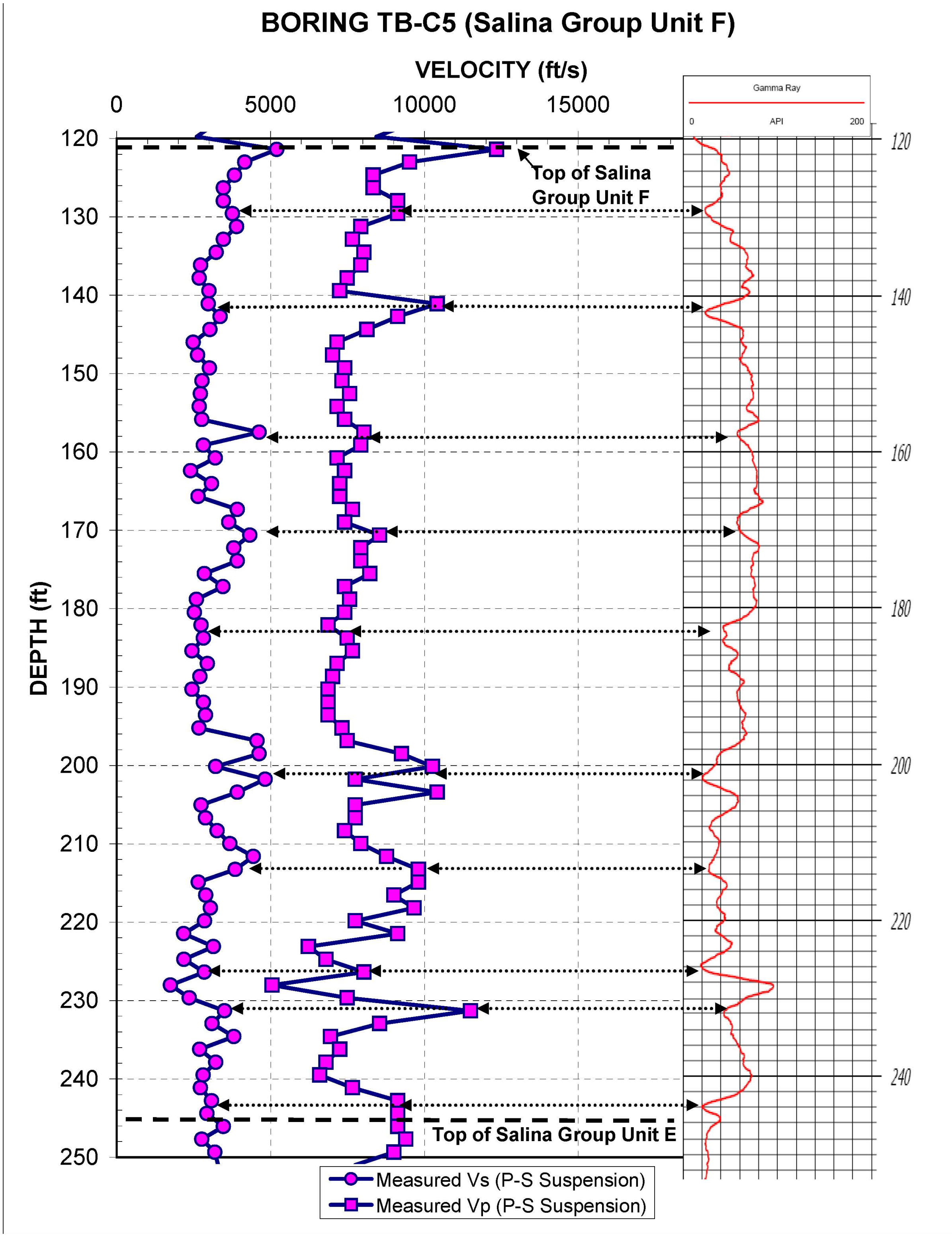


Figure 2.5.4-214 Influence of shale or claystone content within Salina Group Unit F on measured seismic wave velocities in Boring CB-C3[EF3 COL 2.0-29-A]

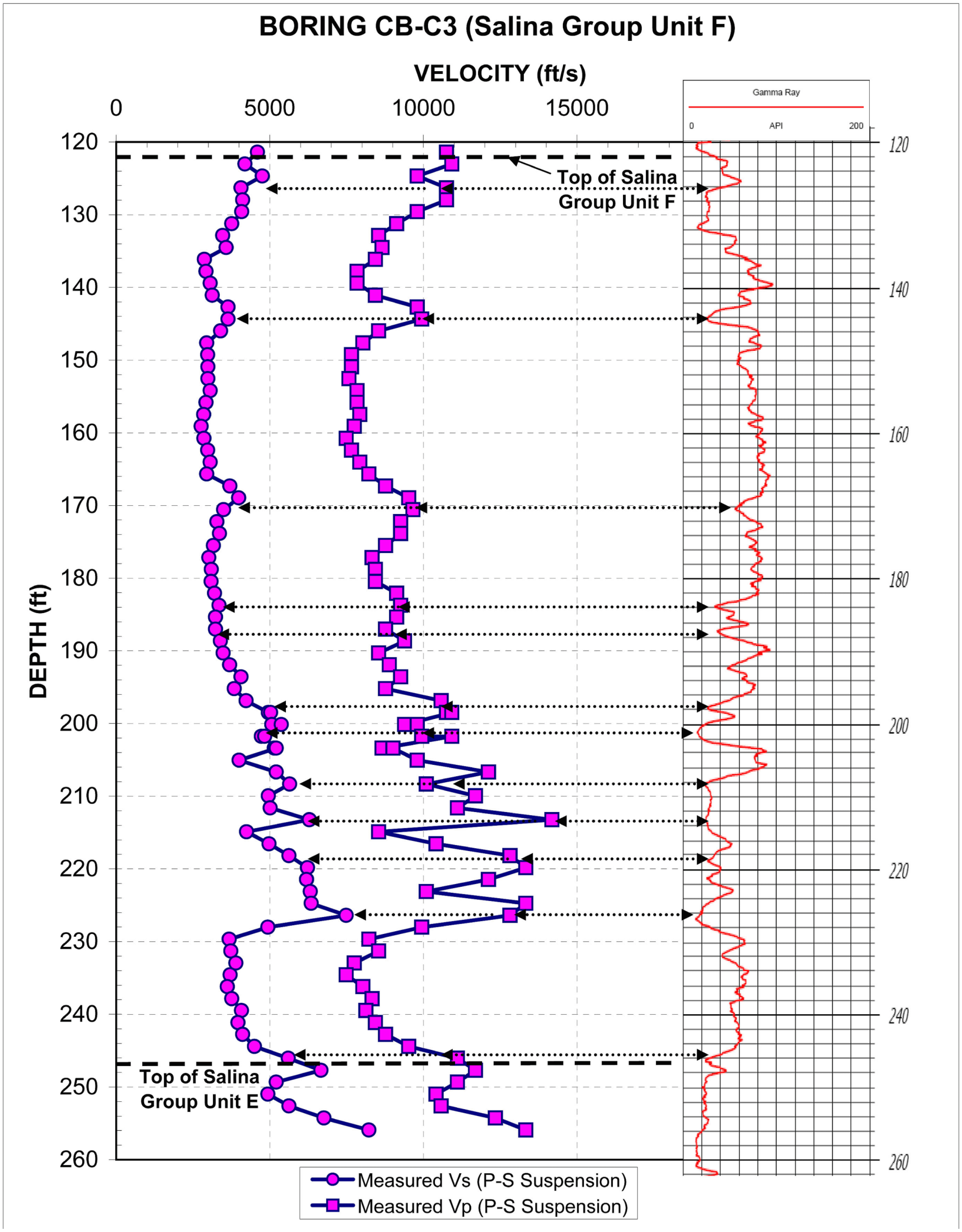


Figure 2.5.4-215 Compression wave velocity measurements using both P-S and Downhole methods in Borings TB-C5, RB-C8, CB-C3, and RB-C4
[EF3 COL 2.0-29-A]

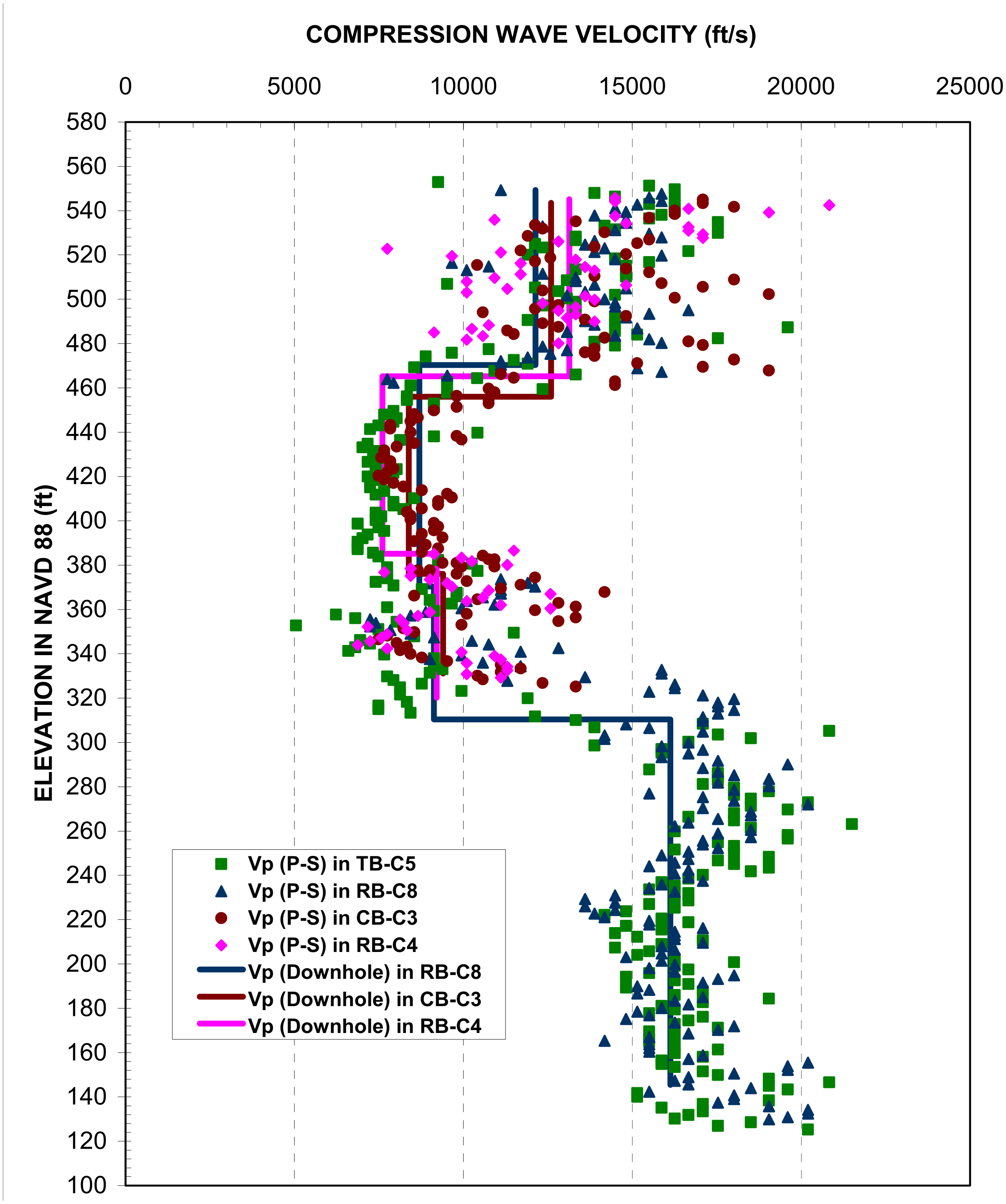


Figure 2.5.4-216 Shear wave velocity measurements using both P-S and Downhole Methods in Borings TB-C5, RB-C8, CB-C3, and RB-C4
[EF3 COL 2.0-29-A]

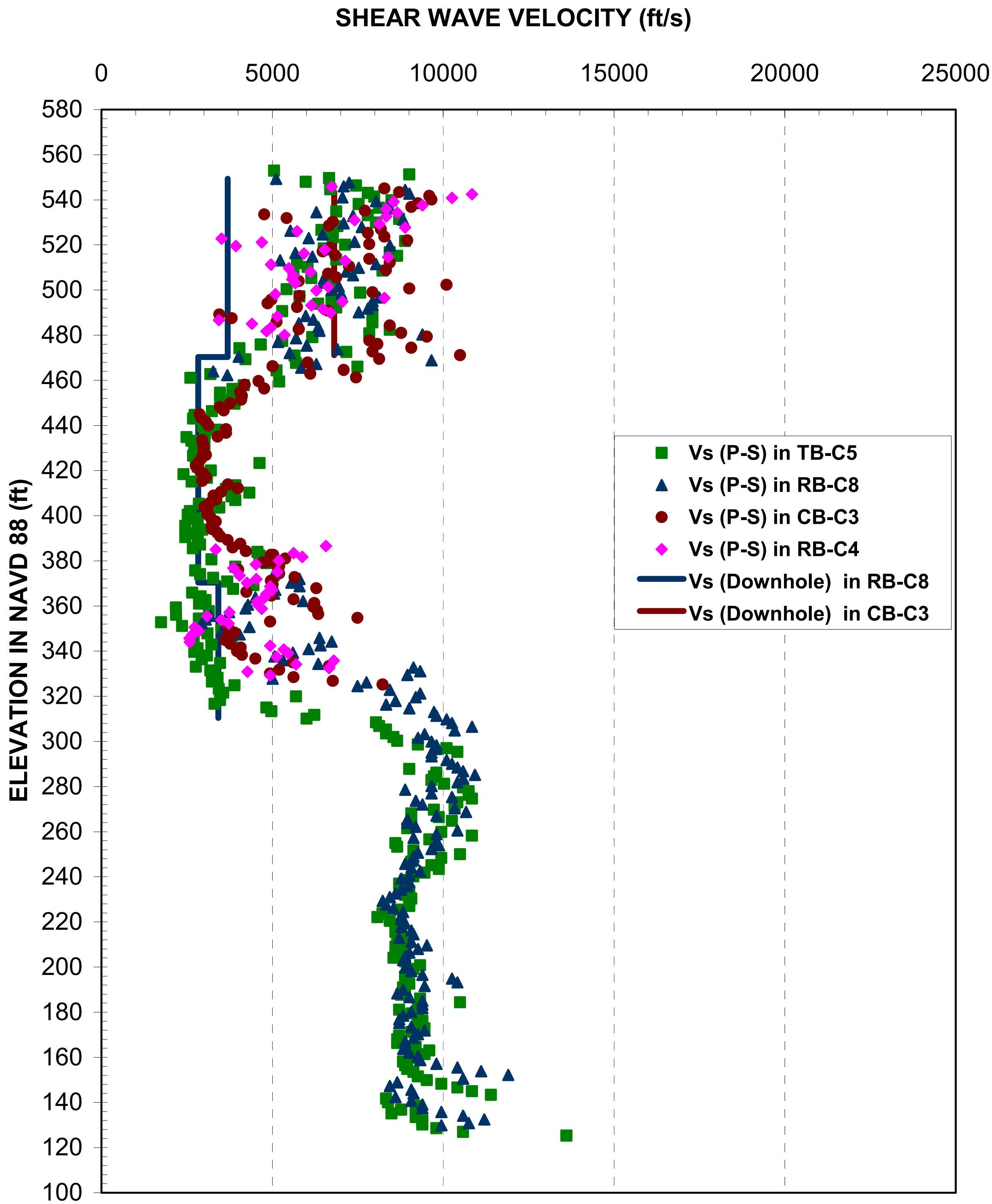


Figure 2.5.4-217 Comparison of measured shear and compression wave velocity profile using P-S suspension method in Boring RB-C6 with measured N-values within overburden [EF3 COL 2.0-29-A]

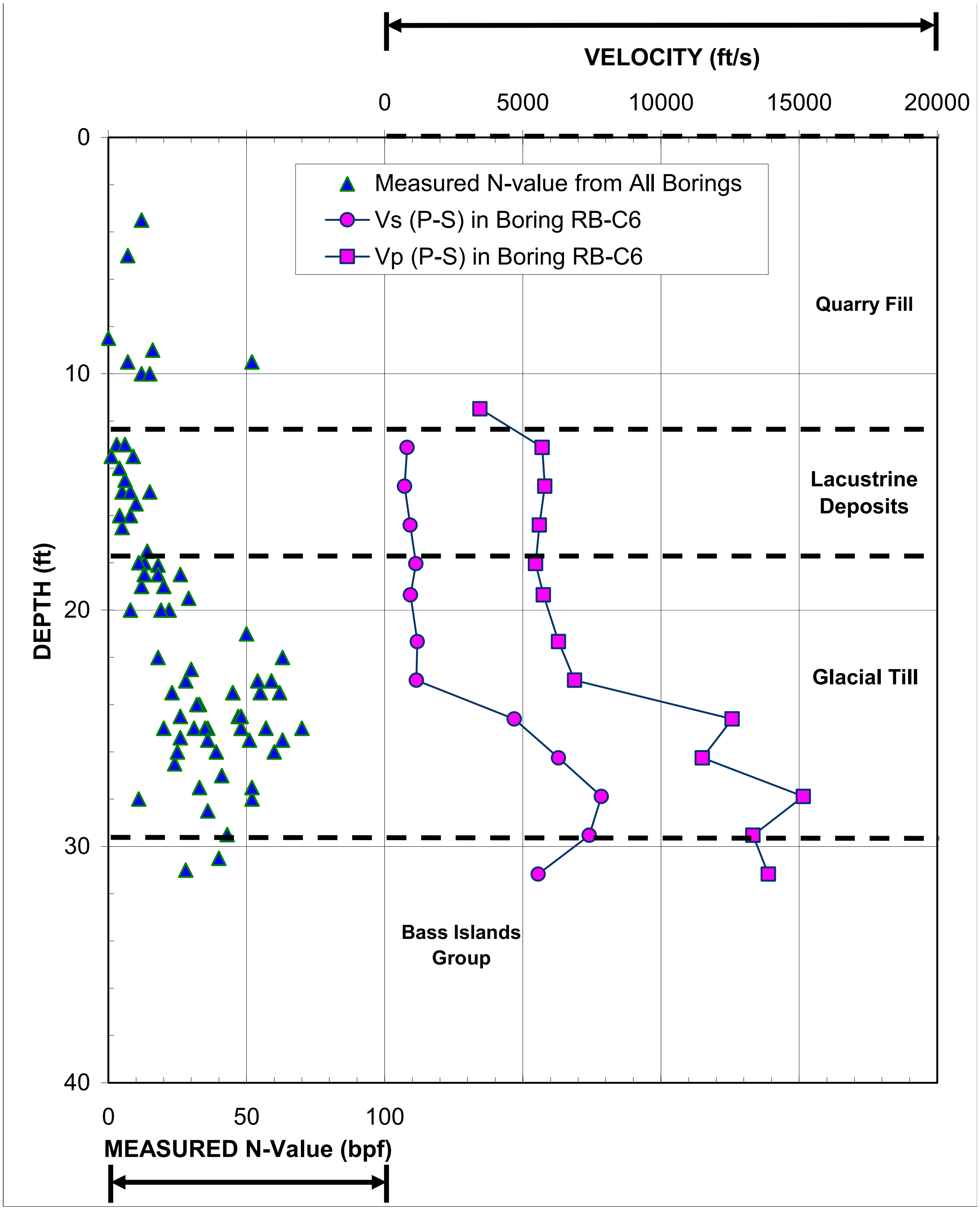


Figure 2.5.4-218 Comparison of measured shear and compression wave velocity profile using P-S suspension method in Boring RB-C6 with gravel content within overburden [EF3 COL 2.0-29-A]

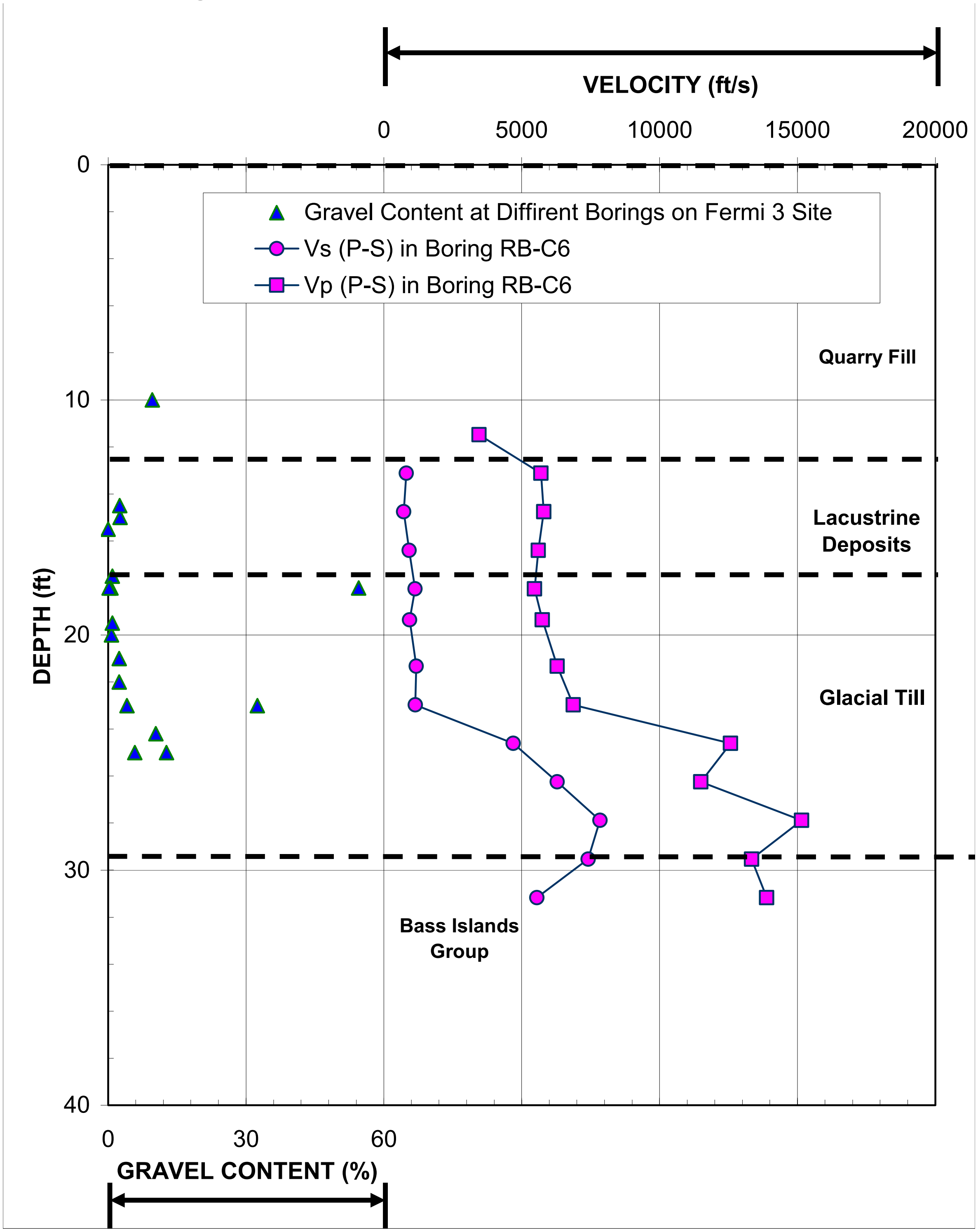
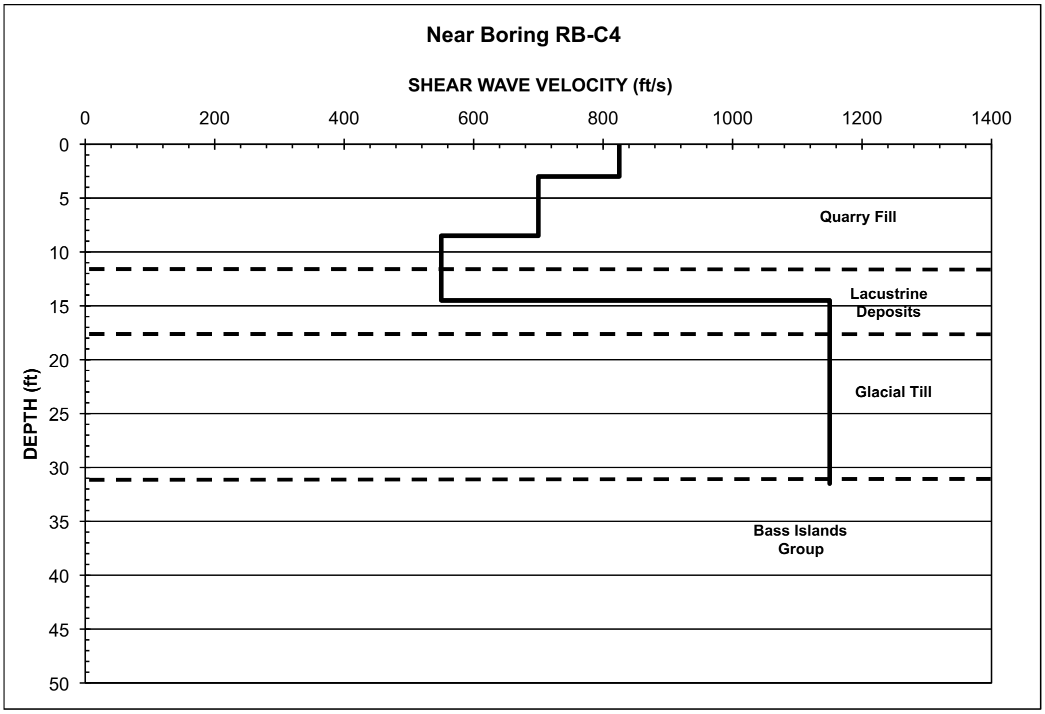


Figure 2.5.4-219 Measured shear wave velocity profile using SASW method in the overburden near Borings RB-C4, RW-C1, MW-381 and MW-393 (Sheet 1 of 2) [EF3 COL 2.0-29-A]



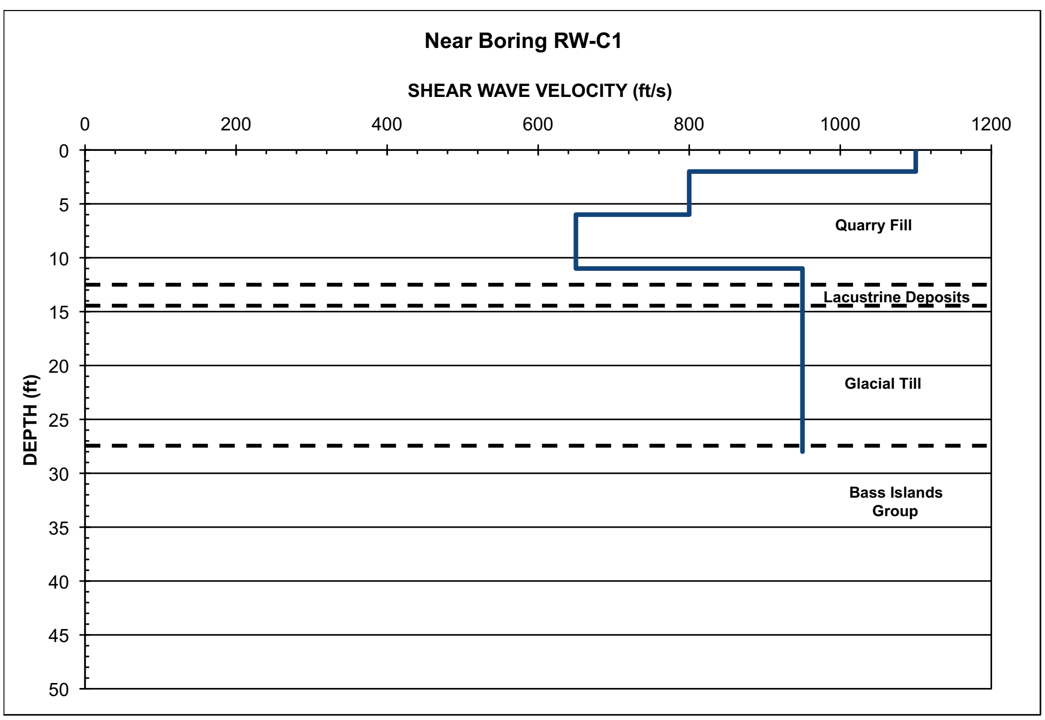
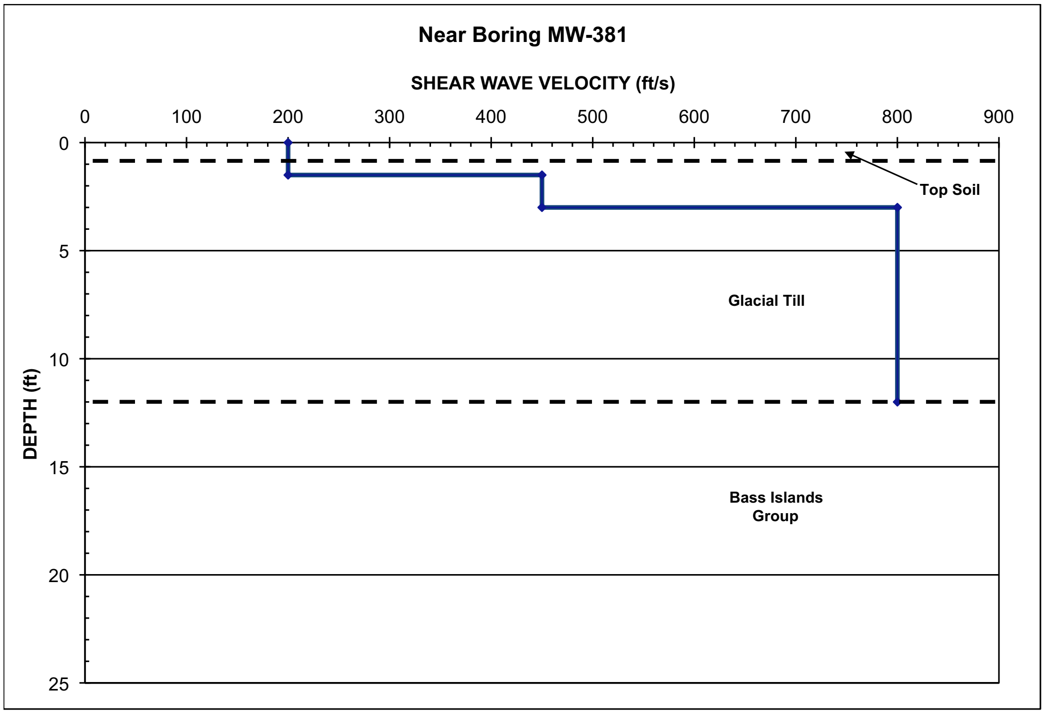


Figure 2.5.4-219 Measured shear wave velocity profile using SASW method in the overburden near Borings RB-C4, RW-C1, MW-381 and MW-393 (Sheet 2 of 2) [EF3 COL 2.0-29-A]



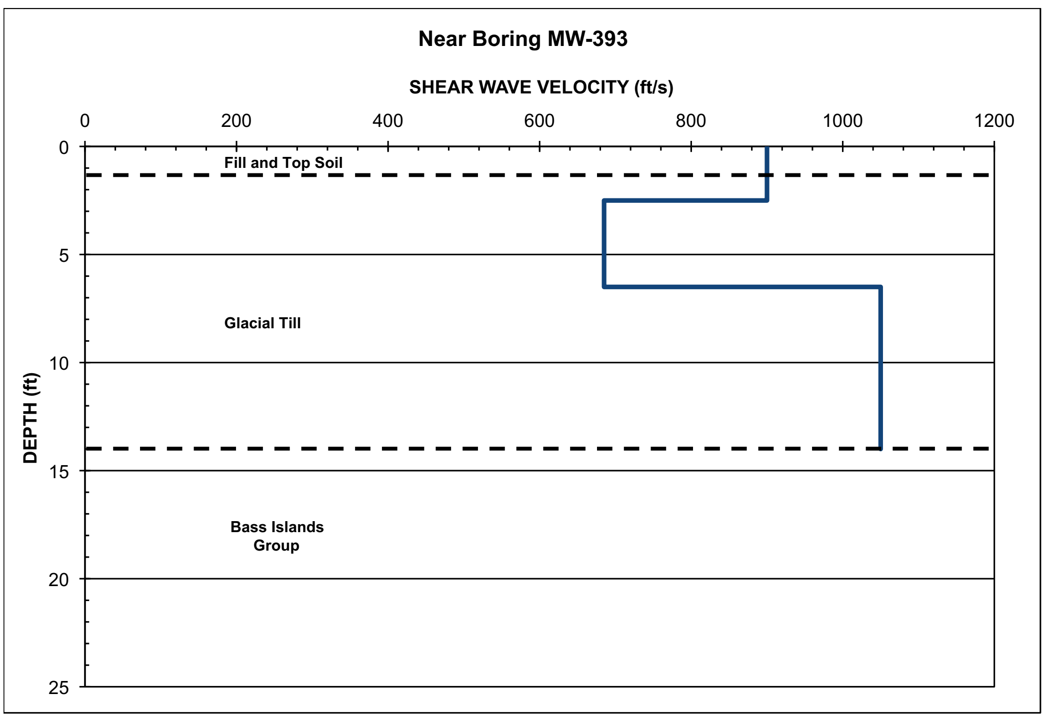


Figure 2.5.4-220 Measured shear and compression wave velocity profiles using P-S Suspension method in Boring TB-C5 [EF3 COL 2.0-29-A]

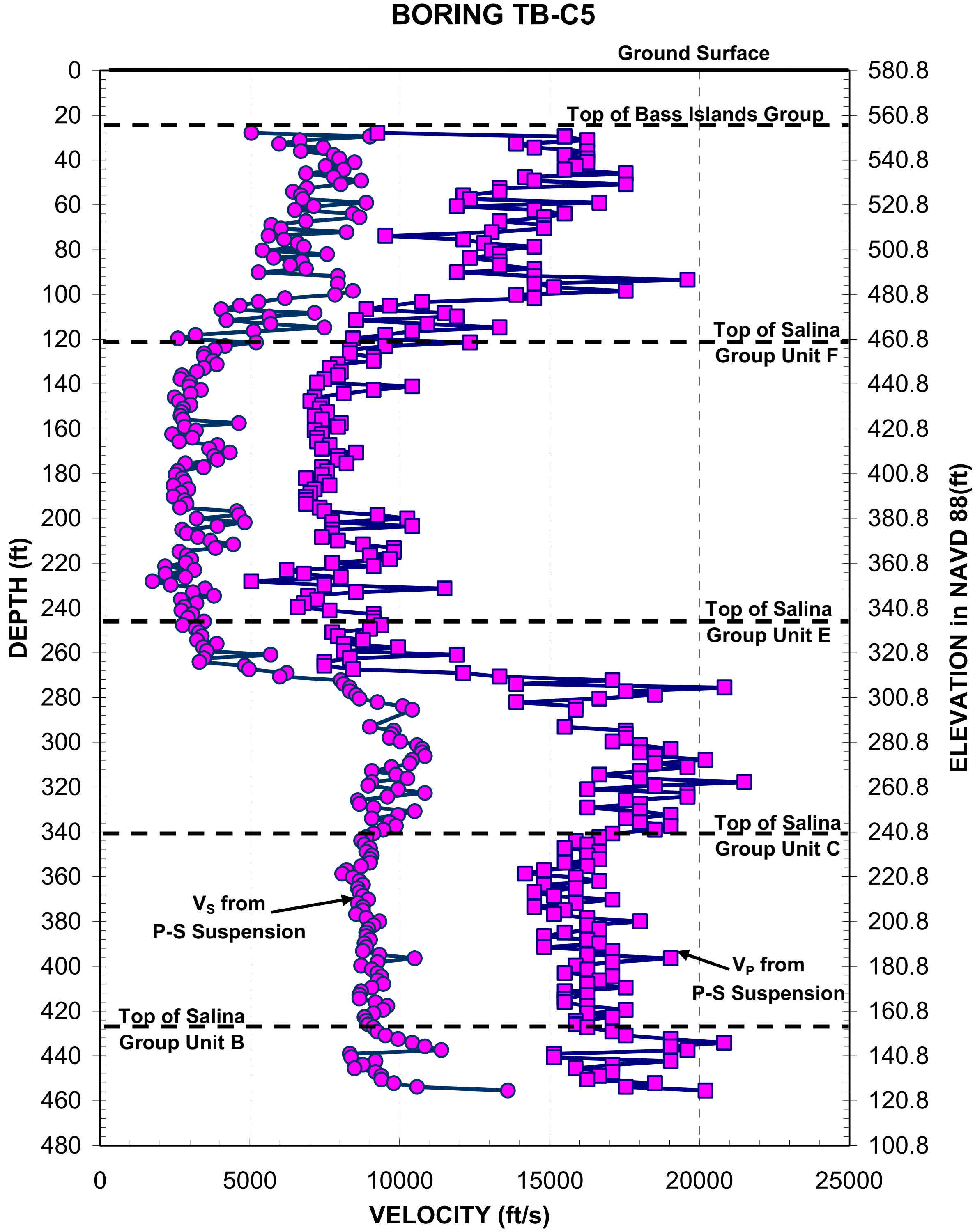


Figure 2.5.4-221 Measured shear and compression wave velocity profiles using P-S Suspension and Downhole Seismic methods in Boring RB-C8 [EF3 COL 2.0-29-A]

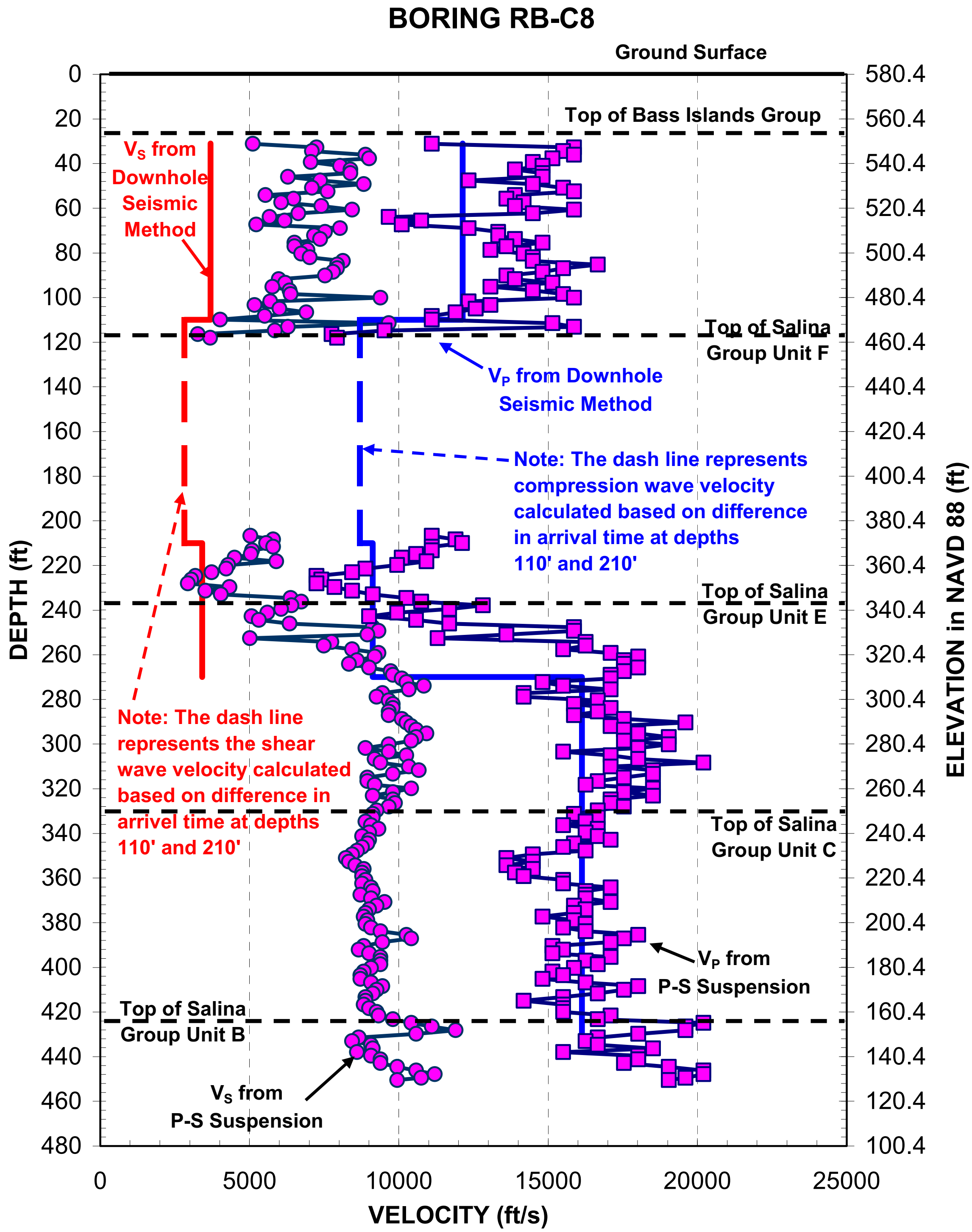


Figure 2.5.4-222 Measured shear and compression wave velocity profiles using P-S Suspension and Downhole Seismic methods in Boring CB-C3 [EF3 COL 2.0-29-A]

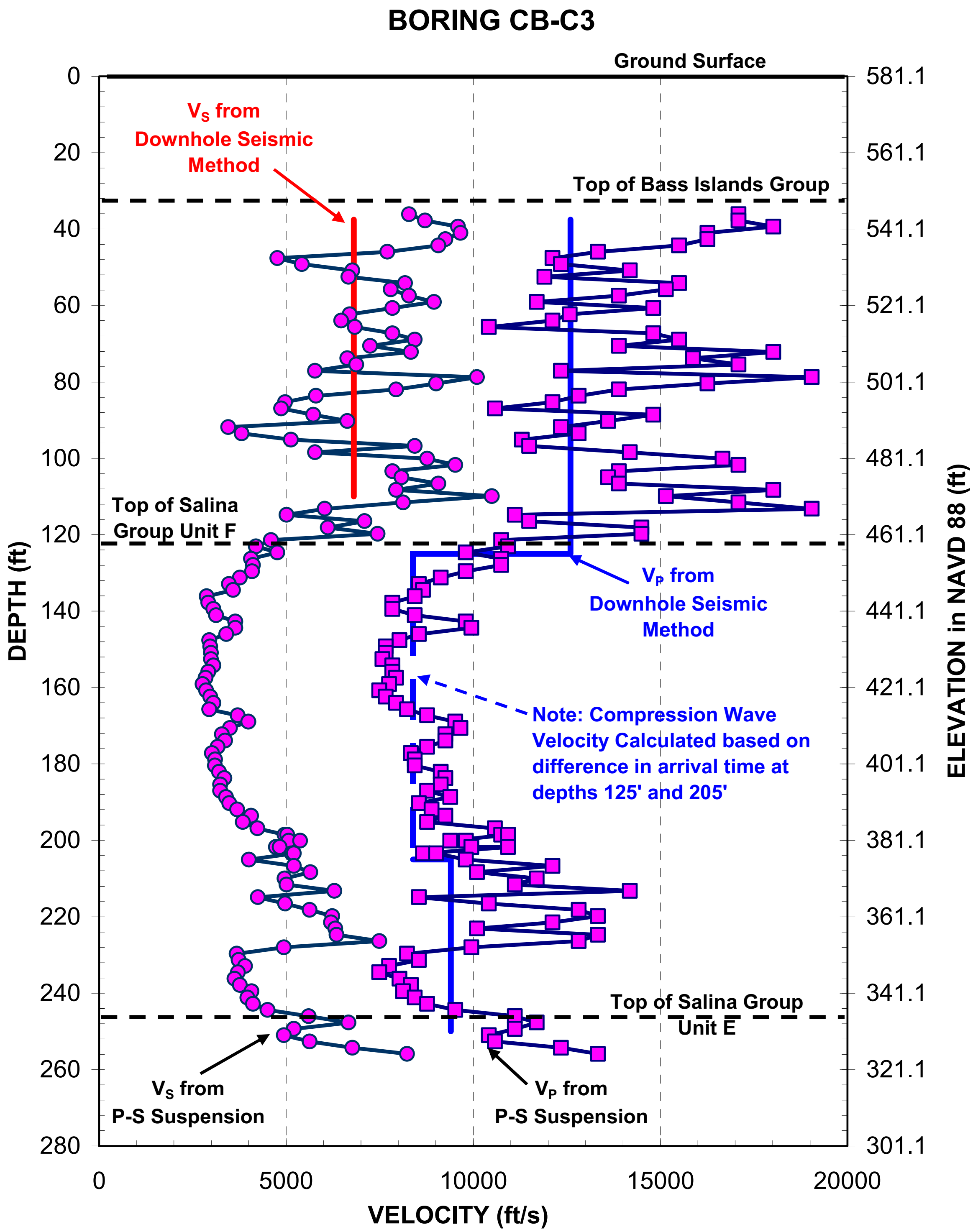


Figure 2.5.4-223 Measured shear and compression wave velocity profiles using P-S Suspension and Downhole Seismic methods in Boring RB-C4 [EF3 COL 2.0-29-A]

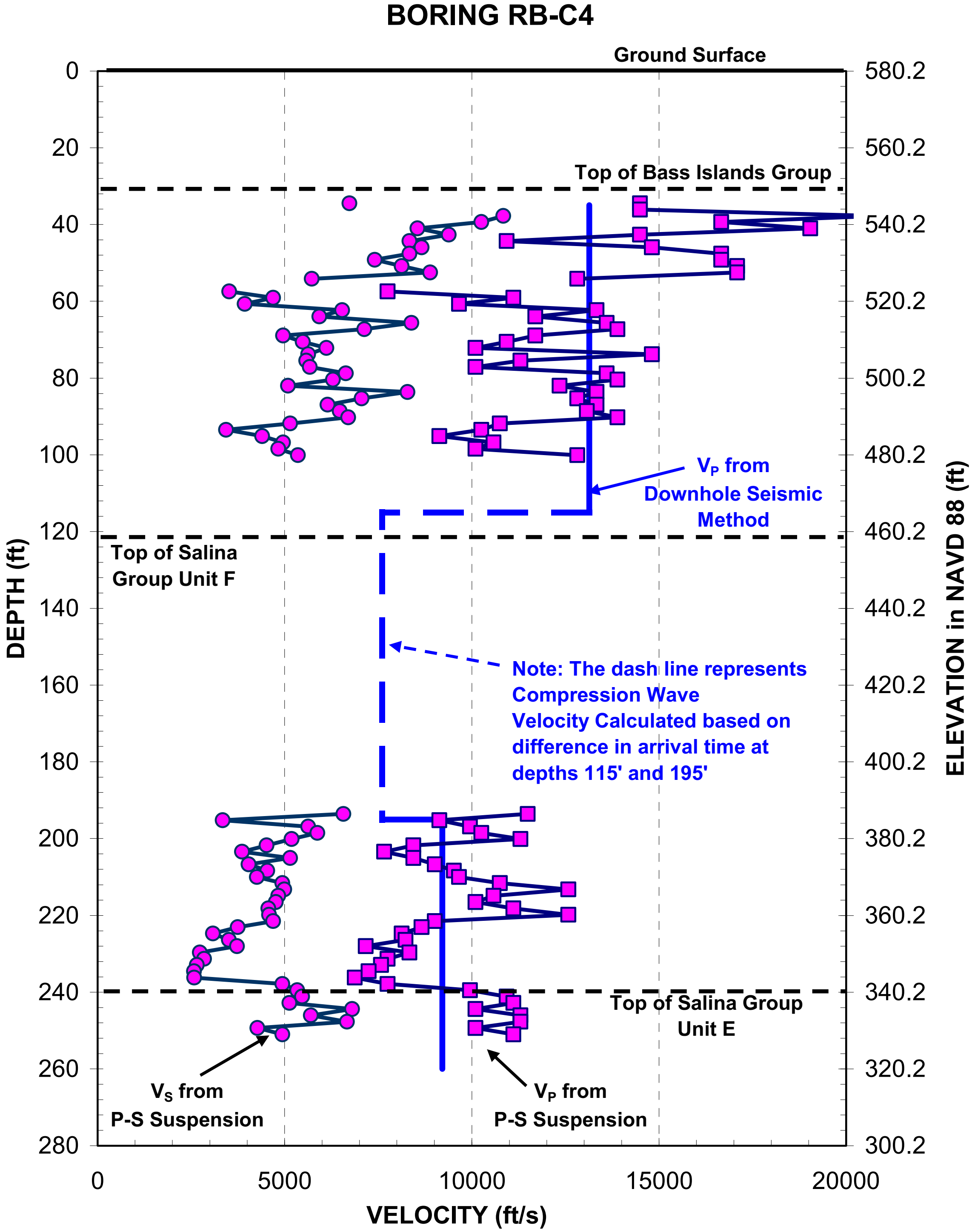


Figure 2.5.4-224 Measured shear and compression wave velocity profiles in the overburden using P-S Suspension Logger in Boring RB-C6 [EF3 COL 2.0-29-A]

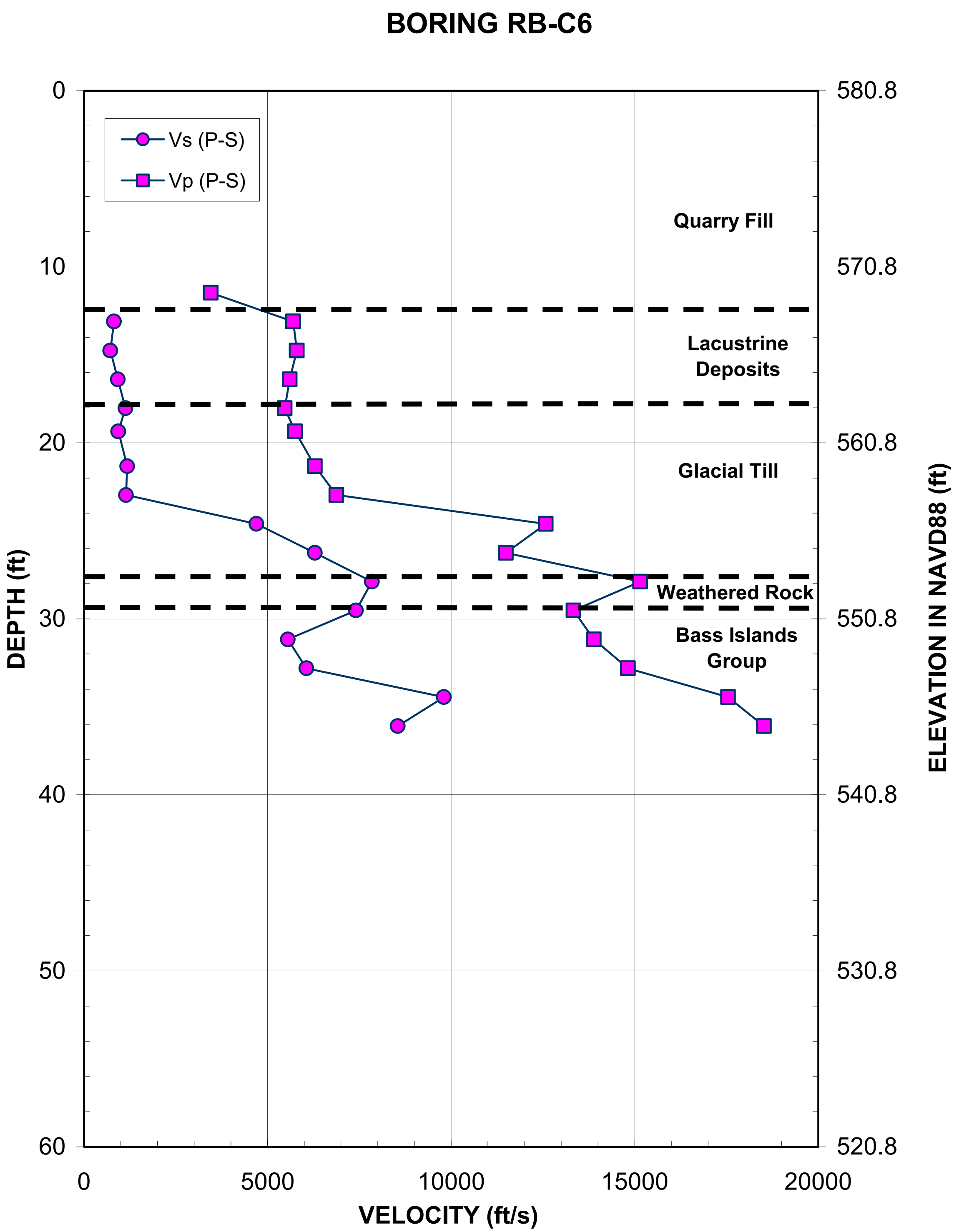
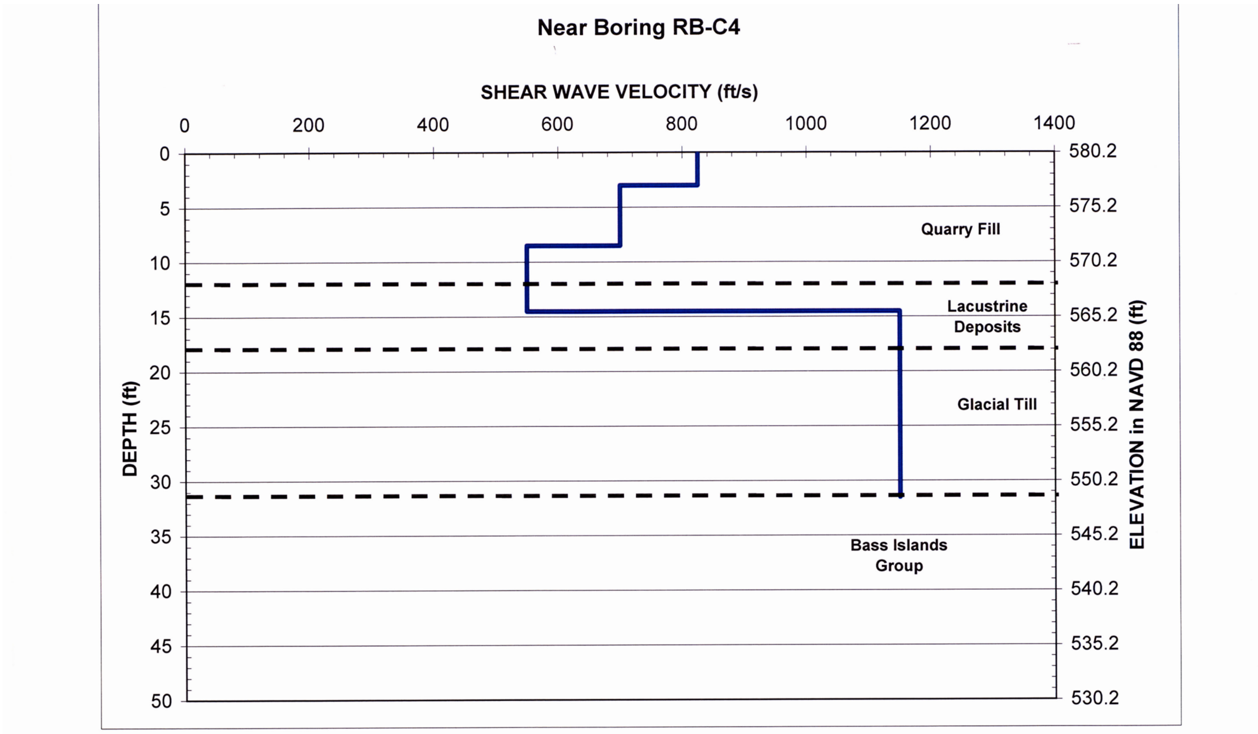


Figure 2.5.4-225 Measured shear and compression wave velocity profiles in the overburden using SASW method near Borings RB-C4 and RW-C1

[EF3 COL 2.0-29-A]



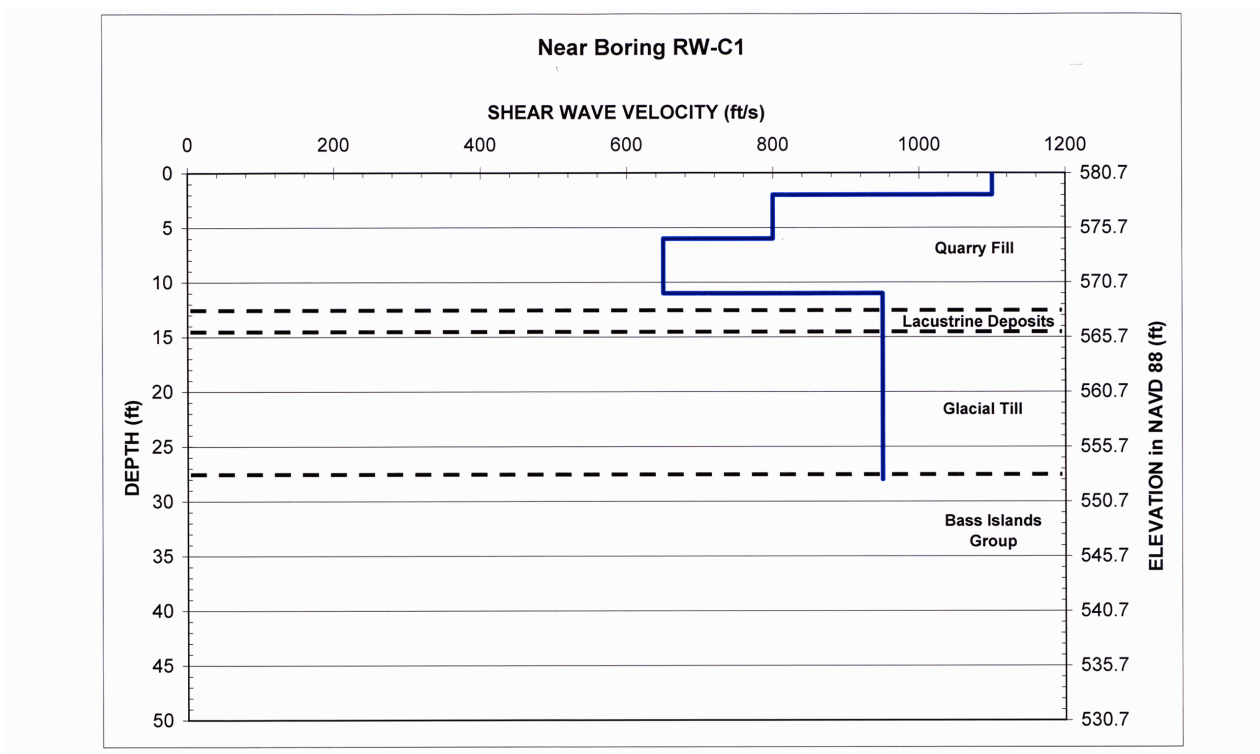
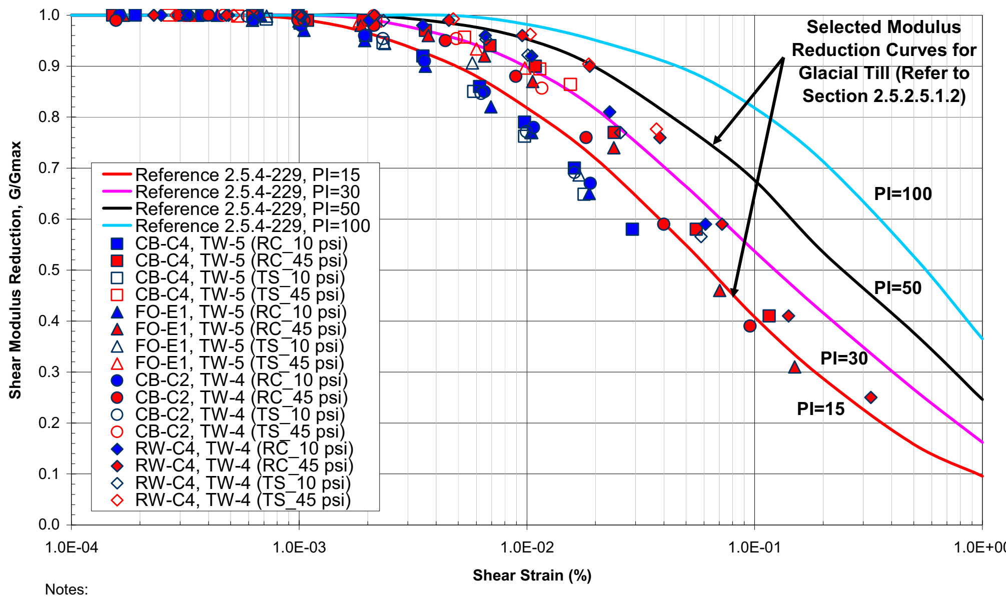
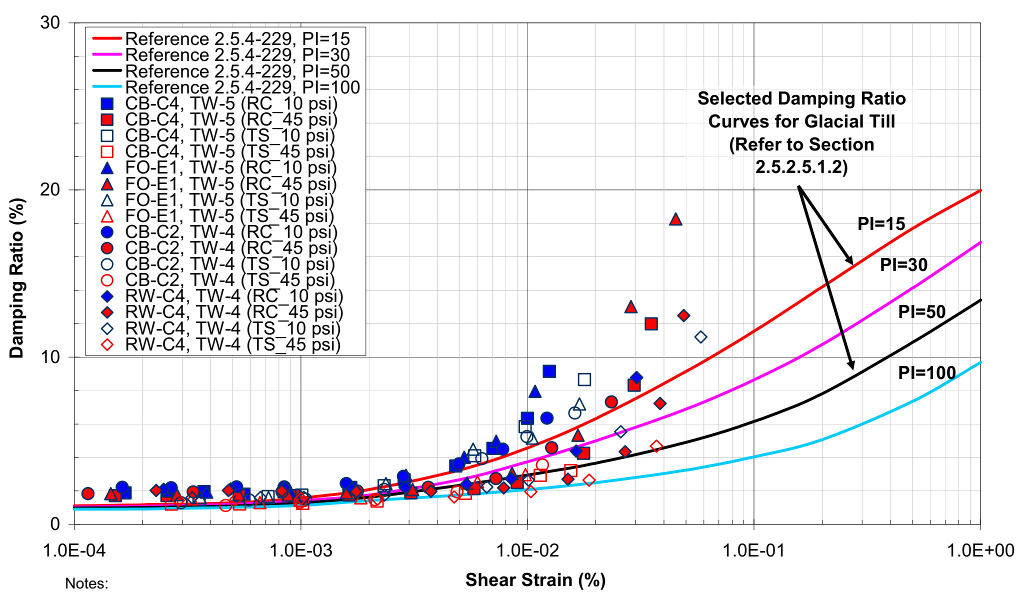


Figure 2.5.4-226 Selected Shear Modulus Reduction and Damping Curves for Glacial Till [EF3 COL 2.0-29-A]

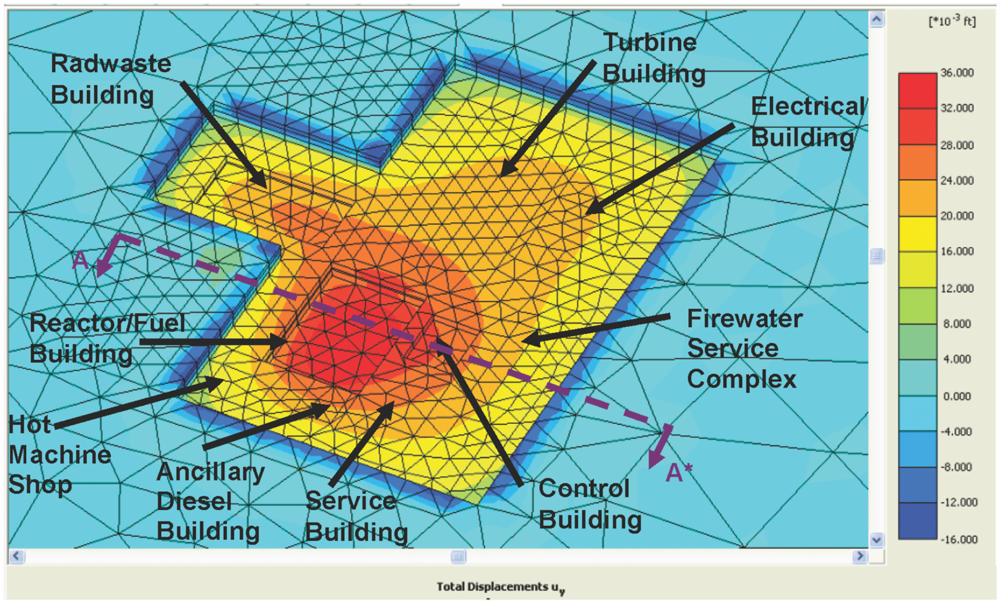


Notes:
1) RC_10psi = Resonant Column Test with Isotropic Confining pressure of 10 psi
2) RC_45psi = Resonant Column Test with Isotropic Confining pressure of 45 psi
3) TS_10psi = Torsional Shear Test with Isotropic Confining pressure of 10 psi
4) TS_45psi = Torsional Shear Test with Isotropic Confining pressure of 45 psi
5) Tests performed on thin-wall tube samples (designated with TW-4 and TW-5) from Borings CB-C4, FO-E1, CB-C2 and RW-C4

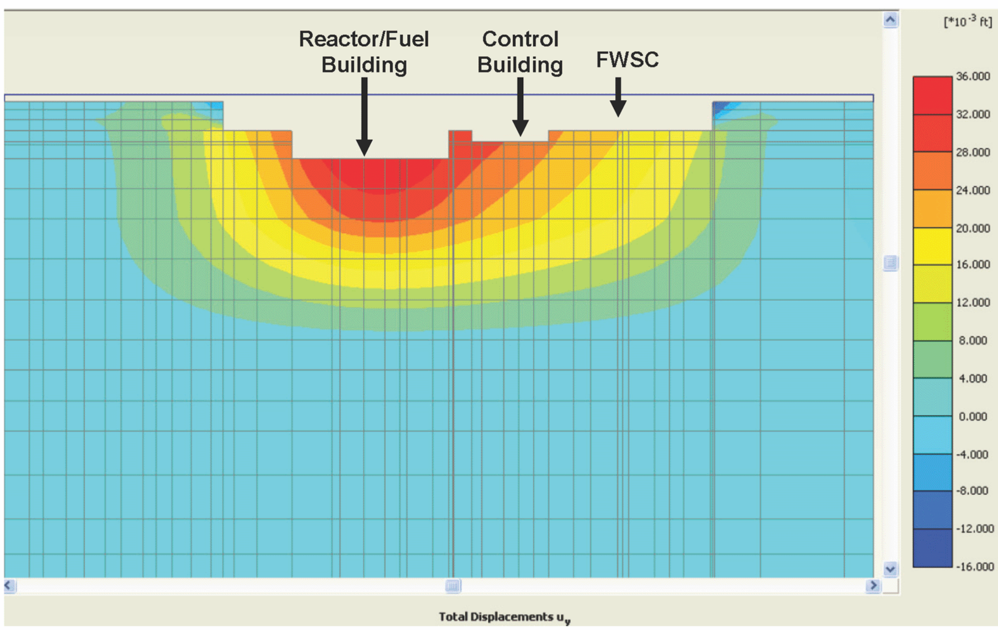


Notes:
1) RC_10psi = Resonant Column Test with Isotropic Confining pressure of 10 psi
2) RC_45psi = Resonant Column Test with Isotropic Confining pressure of 45 psi
3) TS_10psi = Torsional Shear Test with Isotropic Confining pressure of 10 psi
4) TS_45psi = Torsional Shear Test with Isotropic Confining pressure of 45 psi
5) Tests performed on thin-wall tube samples (designated with TW-4 and TW-5) from Borings CB-C4, FO-E1, CB-C2 and RW-C4

Figure 2.5.4-227 Total vertical displacement at end of excavation stage (Rebound due to Excavation) [EF3 COL 2.0-29-A]



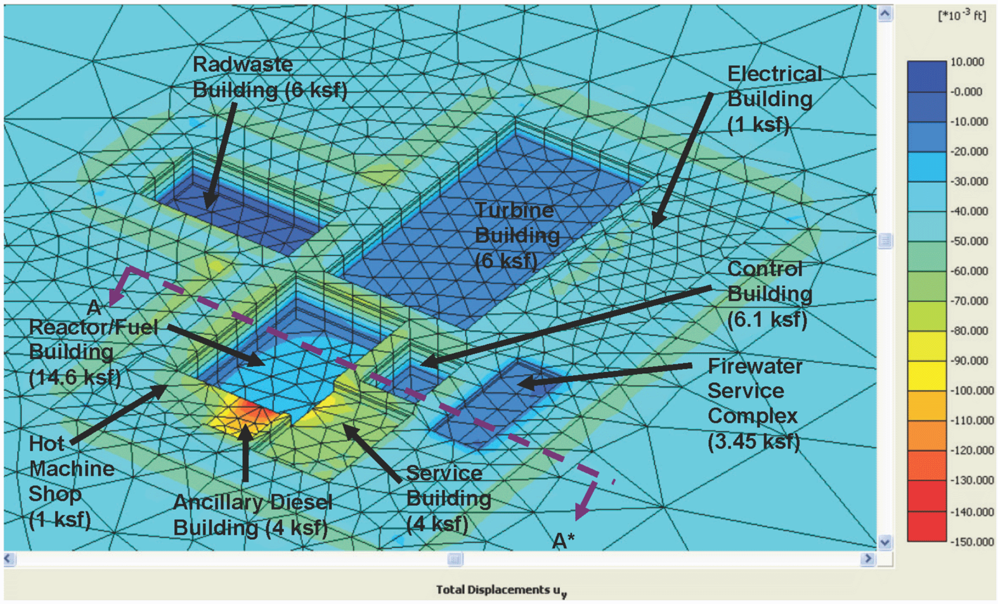
a) Plan View (Rebound)



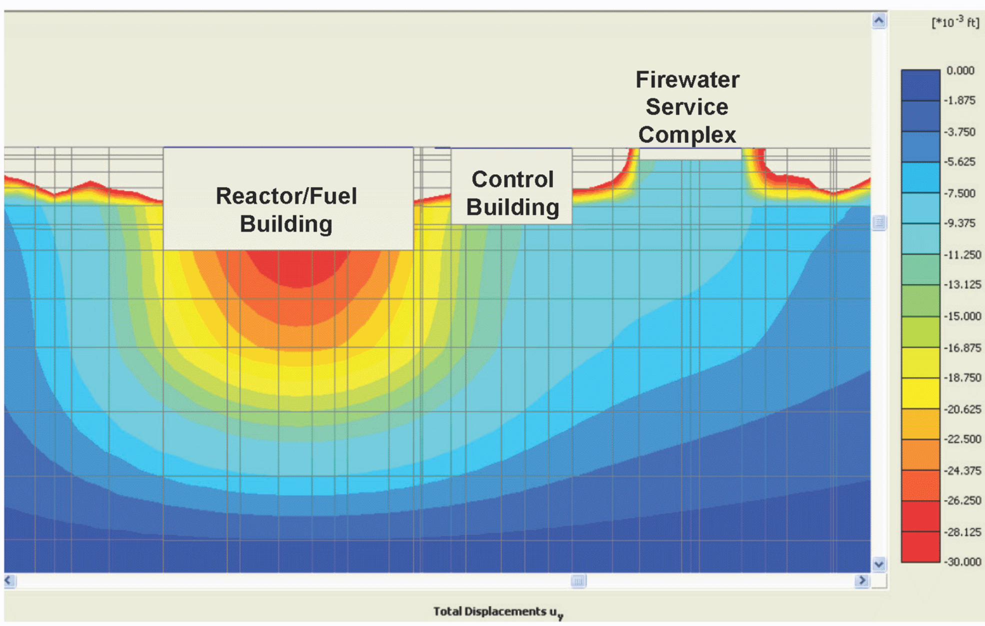
b) Profile View Through Cross-Section A-A* (Center Line of Reactor/Fuel Building)

Note: Positive displacements represent upward soil movements.

Figure 2.5.4-228 Net Settlement at Base of Seismic Category I at End of Loading Stage [EF3 COL 2.0-29-A]



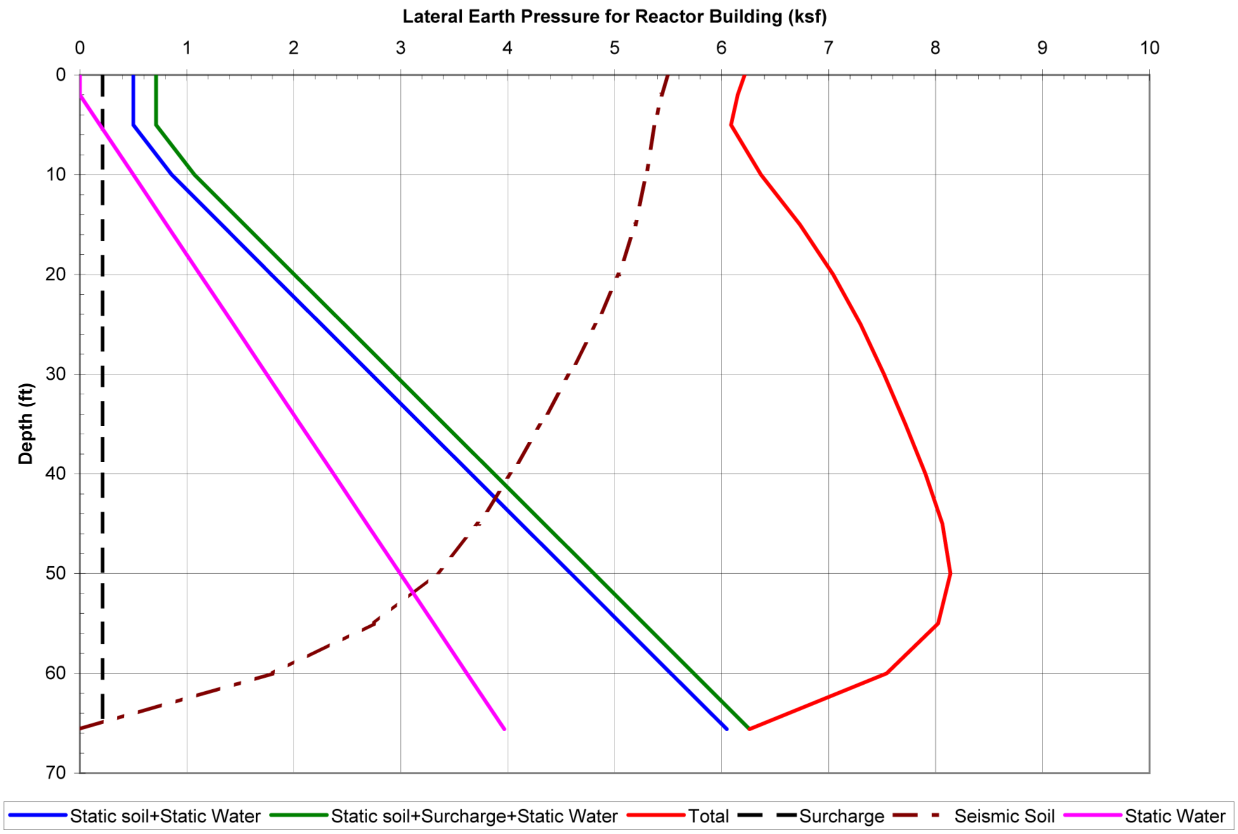
a) Plan View (Net Settlement)



b) Profile View Through Cross-Section A-A* (Center Line of Reactor/Fuel Building)

Note: Net settlement = the settlement from original position prior to excavation

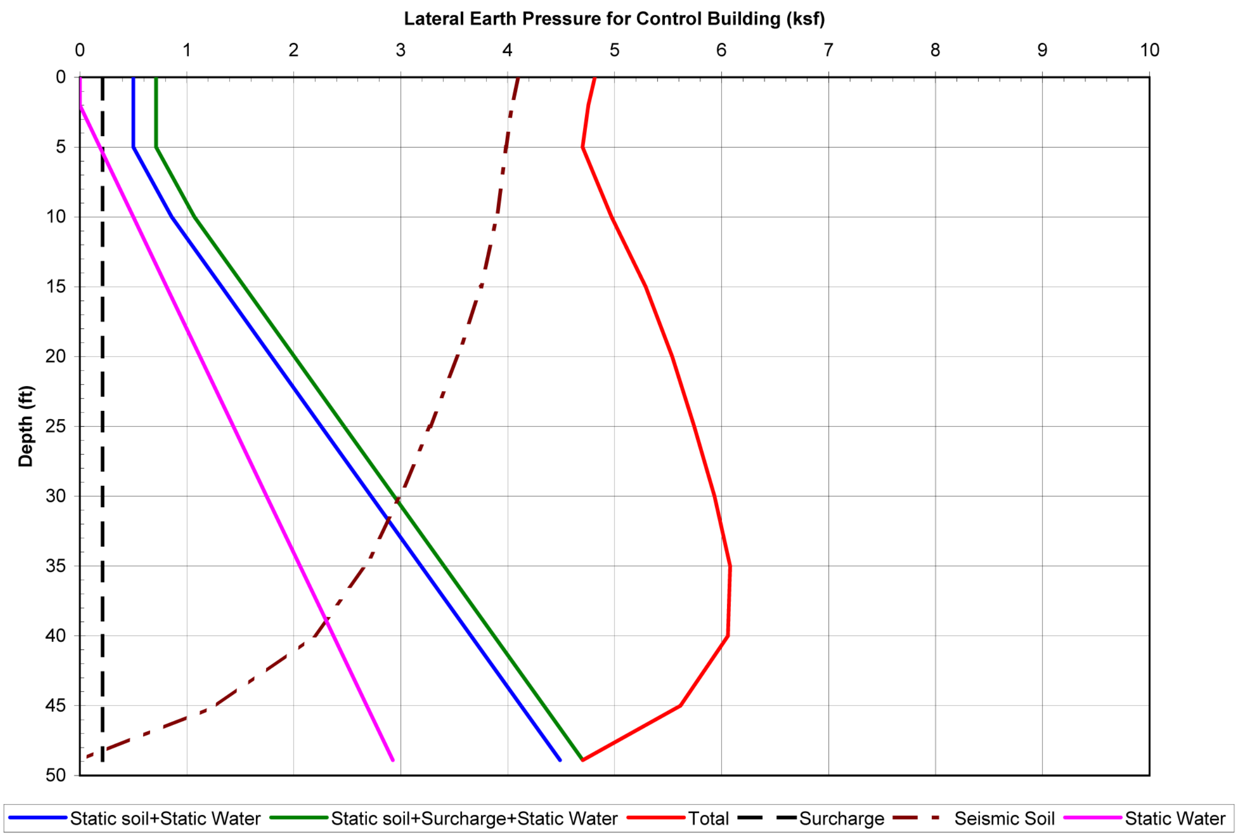
Figure 2.5.4-229 Lateral Earth Pressure on Reactor Building Walls [EF3 COL 2.0-29-A]



Notes:

1. Lateral load of 500 psf due to compaction is included in the static soil pressure.
2. Total = Static Soil + Static Water + Surcharge + Seismic Soil.

Figure 2.5.4-230 Lateral Earth Pressure on Control Building Walls [EF3 COL 2.0-29-A]



Notes:

1. Lateral load of 500 psf due to compaction is included in the static soil pressure.
2. Total = Static Soil + Static Water + Surcharge + Seismic Soil.