FPL.

L-2015-116
10 CFR 52.3

April 10, 2015

U.S. Nuclear Regulatory Commission
Attn: Document Control Desk
Washington, D.C. 20555-0001

Re: Florida Power & Light Company
Proposed Turkey Point Units 6 and 7
Docket Nos. 52-040 and 52-041
Response to NRC Request for Additional Information Letter No. 082 (eRAI 7811)
SRP Section 02.05.04 – Stability of Subsurface Materials and Foundations

References:

1. NRC Letter to FPL dated February 18, 2015, Request for Additional Information Letter No. 082 Related to SRP Section 02.05.04 – Stability of Subsurface Materials and Foundations for the Turkey Point Nuclear Plant Units 6 and 7 Combined License Application

Florida Power & Light Company (FPL) provides, as attachments to this letter, its responses to the Nuclear Regulatory Commission's (NRC) requests for additional information (RAIs) 02.05.04-28, 02.05.04-31 and 02.05.04-37 provided in Reference 1. The attachments identify changes that will be made in a future revision of the Turkey Point Units 6 and 7 Combined License Application (if applicable).

If you have any questions, or need additional information, please contact me at 561-691-7490.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on April 10, 2015.

Sincerely,

William Maher
Senior Licensing Director – New Nuclear Projects

WDM/RFB

Attachment 1: FPL Response to NRC RAI No. 02.05.04-28 (eRAI 7811)
Attachment 2: FPL Response to NRC RAI No. 02.05.04-31 (eRAI 7811)
Attachment 3: FPL Response to NRC RAI No. 02.05.04-37 (eRAI 7811)

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NRD

cc:
PTN 6 & 7 Project Manager, AP1000 Projects Branch 1, USNRC DNRL/NRO
Regional Administrator, Region II, USNRC
Senior Resident Inspector, USNRC, Turkey Point Plant 3 & 4

NRC RAI Letter No. PTN-RAI-LTR-082

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-28 (eRAI 7811)

In response to RAI 02.05.04-2 and in FSAR Subsection 2.5.4.2.1.3.2.1 the applicant attributed the lower N-values to overwashing. The applicant demonstrated the overwashing effect by using the 24-inch sampler in its supplemental field investigation to display that the summations of 3rd and 4th blow counts are consistently higher than the summations of the 2nd and 3rd blow counts (SPT N-value). However, it is not clear why the "SPT N" values obtained from the supplemental investigation are consistently higher than those obtained during the initial investigation, given the similar condition of overwashing ahead of casing for both 18-inch sampler and 24-inch sampler. In accordance with 10 CFR 100.2, please provide insight into the difference between the original SPT N-values and the supplemental SPT N-values shown in Figure 2 of the RAI response.

FPL RESPONSE:

This response is divided into two sections: (1) Discussion of disturbance effects and (2) Comparison of initial and supplemental investigation results focusing on the 2nd and 3rd SPT intervals.

The discussion of disturbance effects is included to provide:

- Insight into the range of expected blow counts from the investigation.
- Reasons for the relatively low blow counts from the investigation when compared to the expected values.
- Insight into the relationship between drilling/sampling requirements and the resulting blow counts based on 2nd and 3rd SPT intervals.

<u>Disturbance Effects:</u>

Prior to conducting the supplemental field investigation (Reference 1), it was considered that the SPT blow counts, or SPT N-values from the initial investigation (Reference 2) associated with the soil formations, were lower than the expected values based on their significant depth (>100 feet) and the high shear wave velocity (>1500 ft/s).

Equivalences between shear wave velocities and blow counts are given in the site classification procedure for seismic design in ASCE/SEI 7-10 (Reference 3). From ASCE/SEI 7-10, the equivalent blow count number (average SPT N value) is higher than 50 for the shear wave velocity (V_s) range of between 1200 and 2500 feet per second. Thus, the blow counts at the Site would be expected to be higher than the values determined in the initial field investigation. Shear wave velocity measurements are used as the reference engineering parameter as they are not impacted by the disturbance effects.

In advance of SPT sampling/testing, the borehole bottom is typically cleaned by means of mud circulation. To avoid disturbance of the underlying soils to be tested, ASTM D 1586-11 (Reference 4) includes a provision to enforce the requirement of side discharge at the drill bit. However, the erodible nature of the silty sands at the Site suggests that even with side discharge of circulating mud, exposure to the effects of overwashing is inevitable. In addition, the soil formations are deep, ranging from approximately 115 feet to 460 feet depth. For such depths, disturbance due to stress relief effects is compounding with the effects of erosion from overwashing.

Having recognized these challenges, a targeted approach was taken during the supplemental investigation where a 24-inch split barrel sampler was used with special attention paid to the overwashing effect (i.e., avoiding prolonged overwashing). It was expected that the soil for the 3rd and 4th intervals would be better protected against the negative impacts of mud circulation, i.e., overwashing, and would therefore be more representative of actual in situ conditions. In addition to using the longer split barrel, the bentonite-water mix was controlled to provide a heavy mud to reduce the effect of stress relief.

From the results, it is recognized that the blow counts obtained with the use of the longer split barrel (intervals 3rd and 4th as opposed to 2nd and 3rd) are less impacted by the effects of overwashing as shown on Figure 1. However, it is acknowledged that the degree of disturbance cannot be fully eliminated.

Disturbance of the soils to be tested can be mitigated to various degrees depending on drilling/sampling requirements and approach. While it is clear that the blow counts based on 3rd and 4th SPT intervals are higher due to the lessened impacts of overwashing at the deeper location of these intervals, specific drilling/sampling conditions are also found to have an influence on the blow counts as obtained from the 2nd and 3rd intervals. The continuous or discontinuous nature of soil sampling has a direct correlation with the degree of exposure of soil to the effects of borehole preparation. Figure 2 is included to graphically show that the volume of material to be washed out of a borehole is higher when a discontinuous sampling method is used as compared to a continuous sampling method. A larger volume of soil to be removed is directly correlated with a longer exposure to the effects of overwashing. In general, three conceptual degrees of exposure to the effects of overwashing are considered based on the experience gained during the field investigation at the Site. Table 1 lists the three main combinations that can be associated to expected degrees of disturbance ranging from the highest exposure to the least exposure.

Table 1
Expected Degree of Disturbance from Overwashing Effects[1]

Sampling	Extracted Volume[2]	Interval Combination for N-value[4]	Expected Degree of Disturbance[3]
Discontinuous	High	2+3	Higher
Continuous	Low	2+3	Intermediate
—	—	3+4	Lower

Notes:
(1) This classification is qualitative based on the experience gained at the Site during drilling and sampling operations.
(2) Figure 2 shows that volume extracted for discontinuous sampling is greater than continuous sampling.
(3) Figure 4, R-6-1b vicinity, shows higher results for R-6-1b which was performed using continuous sampling and therefore has a lower degree of disturbance compared with B-601 (DH) performed using discontinuous sampling.
(4) Figure 1 shows that less disturbance occur at 3+4 blow counts than 2+3 blow counts intervals.

Comparison of Initial and Supplemental Investigation Results:

Figure 3 shows the uncorrected "SPT N" values for the vicinity of the R-6-1b and R-7-1 boreholes. To improve the accuracy of comparison between the initial and supplemental field investigation results, corrected blow counts are given on Figure 4.

On Figure 4, N_{60} includes energy correction (C_{ε}), borehole diameter (C_B), sampler type (C_S), and rod length (C_R) for the SPT N as shown in Equation 1 (FSAR References 2.5.4-219 and 225).

$N_{60} = N\, C_{\varepsilon}\, C_B\, C_S\, C_R$ Equation 1

When based on corrected blow counts, the difference between the results of the initial and supplemental field investigation is reduced for the center of Unit 7. However, values from the supplemental investigation are still slightly higher.

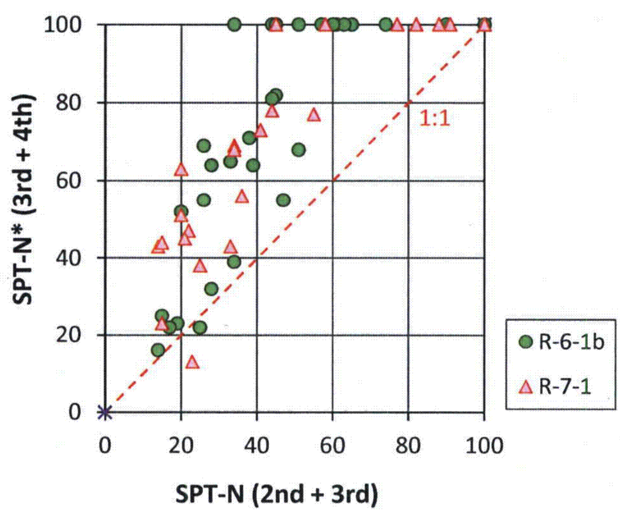
For the center of the reactor at Unit 6, corrected blow counts are consistently higher for the supplemental investigation.

The slight difference between the corrected blow counts from the initial and supplemental investigation found within the reactor of Unit 7 boreholes may be explained by the fact that sampling intervals were similar, i.e., discontinuous sampling. Alternatively the more pronounced difference within Unit 6 boreholes is attributed to the continuous sampling approach taken at borehole R-6-1b while at B-601 a discontinuous sampling was used.

In summary, the difference between the initial and supplemental field investigation results can be attributed to:

- Drilling/sampling requirements and approach, specifically the time exposure of soils to the effects of borehole cleaning prior to testing, i.e., continuous vs. discontinuous sampling, and
- The fact that the supplemental investigation's target was to reduce disturbance effects by avoiding unnecessary delays between sample locations, and to adopt a tight control on the consistency of circulating mud.

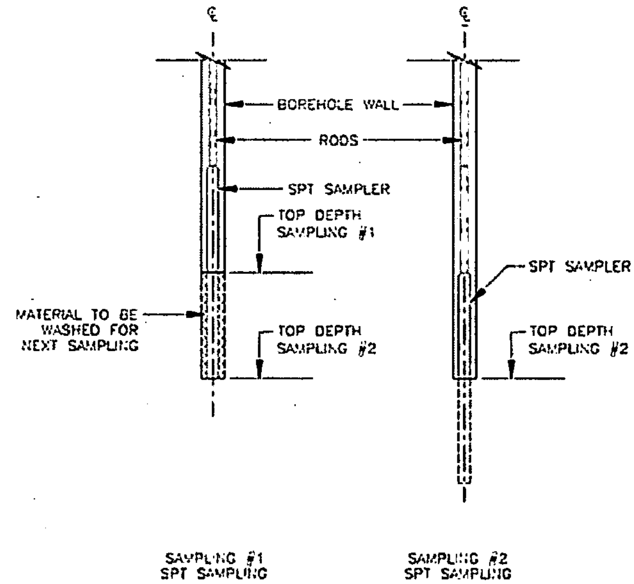
Figure 1 Plot of Uncorrected SPT N Values versus the Summation of 3rd and 4th Blow Counts from the Supplemental Investigation



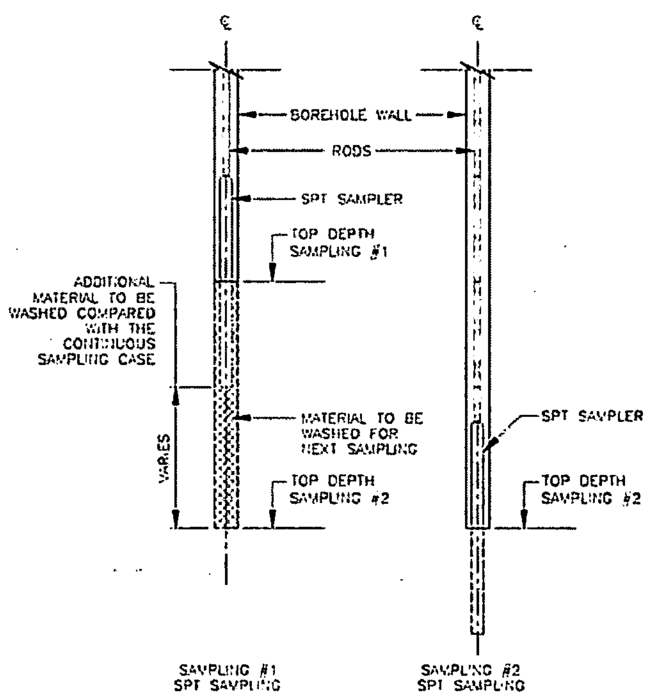
Note:
* Summation of 3rd and 4th blow count intervals

Data from Reference 1

Figure 2 Continuous and Discontinuous SPT Sampling



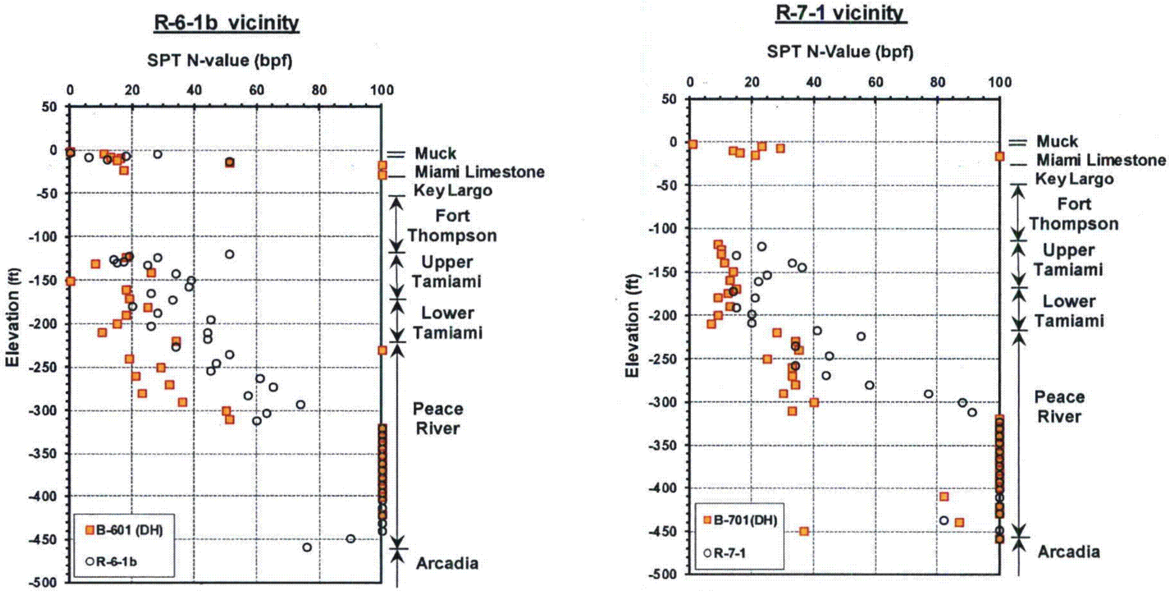
CONTINUOUS SAMPLING CASE
(NOT TO SCALE)



DISCONTINUOUS SAMPLING CASE
(NOT TO SCALE)

Note: General scheme developed as sampling was performed according to References 1 and 2

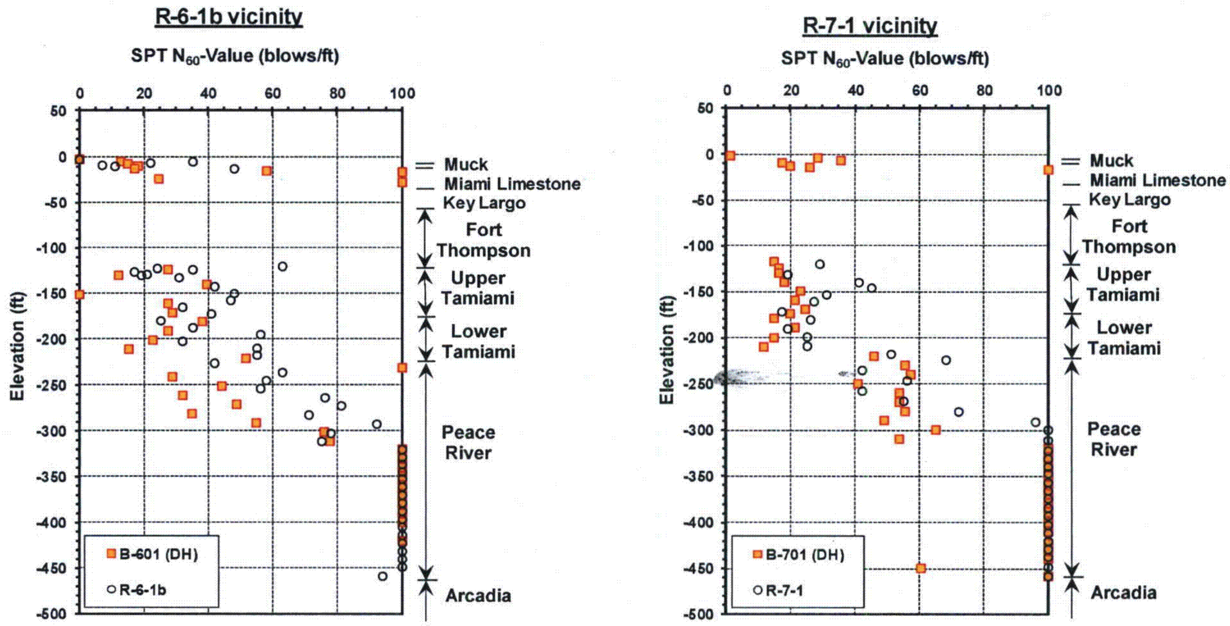
Figure 3 Plot of Uncorrected SPT N Values for R-6-1b and R-7-1 Vicinity



Data from References 1 and 2

Notes: Elevation is NAVD 88

Figure 4 Plot of Corrected SPT N_{60} Values for R-6-1b and R-7-1 Vicinity



Data from References 1 and 2

Notes: Elevation is NAVD 88

This response is PLANT SPECIFIC.

References:

1. Paul C. Rizzo Associates, Inc., *Supplemental Field Investigation Data Report, Turkey Point Nuclear Power Plant Units 6 & 7*, Rev. 2, Pittsburgh, Pennsylvania, included In COL Application Part 11, April 15, 2014.

2. MACTEC Engineering and Consulting, Inc., *Final Data Report – Geotechnical Exploration and Testing: Turkey Point COL Project Florida City, Florida*, Revision 2, included in COL Application Part 11, October 6, 2008.

3. American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI 7-10), *Minimum Design Loads for Buildings and Other Structures*, 2010.

4. ASTM D 1586-11, *Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*, ASTM International, West Conshohocken, PA, 2011. www.astm.org.

ASSOCIATED COLA REVISIONS:

FSAR Subsection 2.5.4.2.1.3.2.1 will be revised in a future COLA revision as follows:

2.5.4.2.1.3.2.1 Uncorrected N-Values

A summary of all N-values (inside and outside of the power block) measured in the field (uncorrected) during the initial and supplemental investigations is presented on Figure 2.5.4-212 and Table 2.5.4-202. Some very low N-values in the upper and lower Tamiami and Peace River Formations are questionable as described in Subsection 2.5.4.8.2. The "SPT N" values obtained from the supplemental investigation at R-6-1b and R-7-1 are consistently higher than those obtained during the initial investigation at B-601 (DH) and B-701 (DH) for both testing/sampling **(Figure 2.5.4-270)**. Furthermore, the summations of 3rd and 4th blow counts obtained at R-6-1b and R-7-1 are consistently higher than the summation of 2nd and 3rd blow counts ("SPT N"). This is a consistent trend throughout the SPT tests, and is attributed to overwashing as defined in Table 13 of NAVFAC DM 7.1 (Reference 301). Overwashing is considered to be the cause of the lower N-values**, and is described in more detailed in the subsection on disturbance effects below**. Engineering analyses or parameter developments are based only on the summation of the 2nd and 3rd blow counts per ASTM D 1586 (Reference 302). The 3rd and 4th blow counts are included for the purpose of investigating potential overwashing. **In a conservative approach,** ~~T~~the overall pool of "SPT N" values from the initial and supplemental investigations are used in development of ~~other~~ engineering parameters (friction angle and modulus of deformation) ~~or engineering analyses (liquefaction) with an understanding that these values are lower and conservative~~.

2.5.4.2.1.3.2.1.1 Disturbance Effects

Prior to conducting the supplemental field investigation, it was considered that the SPT blow counts, or SPT N-values (Reference 257) associated with the soil formations, were lower than the expected values based on their significant depth (>100 feet) and a high shear wave velocity above 1500 ft/s (Table 2.5.4-209).

Equivalences between shear wave velocities and blow counts are given in the site classification procedure for seismic design in ASCE/SEI 7-10 (Reference 322). From ASCE/SEI 7-10, the equivalent blow count number (average SPT N value) is higher than 50 for the shear wave velocity (V_s) range of between 1200 and 2500 ft/s. Thus, the blow counts at the site would be expected to be higher than the values determined in the initial field investigation (Reference 257). Shear wave velocity measurements are used as the reference engineering parameter as they are not impacted by the disturbance effects.

In advance of SPT sampling/testing, the borehole bottom is typically cleaned by means of mud circulation. To avoid disturbance of the underlying soils to be tested, ASTM D 1586-11 (Reference 302) includes a provision to enforce the requirement of side discharge at the drill bit. However, the erodible nature of the silty sands at the site suggests that even with side discharge of circulating mud, exposure to the effects of overwashing is inevitable. In addition, the soil

formations are deep, ranging from approximately 115 feet to 460 feet depth. For such depths, disturbance due to stress relief effects is compounding with the effects of erosion from overwashing.

Having recognized these challenges, a targeted approach was taken during the supplemental investigation where a 24-inch-long split barrel sampler was used with special attention paid to the overwashing effect (i.e., avoiding prolonged overwashing). It was expected that the soil for the 3rd and 4th intervals would be better protected against the negative impacts of mud circulation, i.e. overwashing, and would therefore be more representative of actual in situ conditions. In addition to using the longer split barrel, the bentonite-water mix was controlled to provide a heavy mud to reduce the effect of stress relief.

From the results, it is recognized that the blow counts obtained with the use of the longer split barrel (intervals 3rd and 4th as opposed to 2nd and 3rd) are less impacted by the effects of overwashing as shown on Figure 2.5.4-269.

Disturbance of the soils to be tested can be mitigated to various degrees depending on drilling/sampling requirements and approach. While it is clear that the blow counts based on 3rd and 4th SPT intervals are higher due to the lessened impacts of overwashing at the deeper location of these intervals, specific drilling/sampling conditions are also found to have an influence on the blow counts as obtained from the 2nd and 3rd intervals. The continuous or discontinuous nature of soil sampling has a direct correlation with the degree of exposure of soil to the effects of borehole preparation. Figure 2.5.4-271 is included to graphically show that the volume of material to be washed out of a borehole is higher when a discontinuous sampling method is used as compared to a continuous sampling method. A larger volume of soil to be removed is directly correlated with a longer exposure to the effects of overwashing.

FSAR Subsection 2.5.4.2.1.3.2.1 will be revised in a future COLA revision as follows:

2.5.4.2.1.3.2.2 N-Value Correction

Field SPT N-values are adjusted for SPT hammer energy, borehole diameter (CB), sampler (CS) and rod length (CR). This adjusted N-value, N_{60}, is determined using the following equation (References 219 and 225):

$$N_{60} = N\, C_{\varepsilon}\, C_B\, C_S\, C_R \qquad \text{Equation 2.5.4-2}$$

Where,

N = field measured SPT blow count
C_{ε} = hammer energy correction factor
C_B = borehole diameter correction factor
C_S = sampler correction factor
C_R = rod length correction factor

The SPT N-value used in correlations with engineering properties is a value traditionally based on 60 percent hammer efficiency. SPT hammer energy measurements are made for each drilling rig/hammer employed, in accordance with ASTM D 6066 (Reference 220), and the hammer energy measurements (expressed as energy transfer ratios, or ETRs) are obtained. As shown in Table 2.5.4-203, average ETRs range from 62.1 percent to 88.0 percent. The resulting energy correction factor, C_ε (expressed as ETR/60%), ranges from 1.04 to 1.47, also as shown in Table 2.5.4-203. N_{60}-values (from Equation 2.5.4-2) from each boring are corrected using the appropriate C_ε value. The resulting SPT N-values are termed N_{60}. For the liquefaction analysis, additional correction factors for overburden pressure are applied.

A summary of all N_{60} values with depth is shown on Figure 2.5.4-213 and in Table 2.5.4-204.

A comparison of corrected N-values, N_{60}, at Unit 6 [B-601(DH) vs. R-6-1b] and Unit 7 [B-701(DH) vs. R-7-1] boreholes shows the same trend discussed in Subsection 2.5.4.2.1.3.2.1, i.e. values from the supplemental investigation are higher than those from the initial investigation (Figure 2.5.4-272). The differences range from slight to more pronounced. A description of the source of these differences is given below in relation to the sampling approach taken during the supplemental investigation at Units 6 and 7.

For R-7-1 and B-701 (DH), where discontinuous sampling method was performed, results are only slightly different. The slight improvement in the results of the supplemental investigation over the initial investigation is likely due to a pre-established target during the supplemental investigation to reduce disturbance effects by avoiding unnecessary delays between sample locations, as well as a tight control on the consistency of circulating mud, with a resulting mitigation in the exposure of soils to the effects of overwashing.

Alternatively, there is a more pronounced difference within Unit 6 boreholes and this is attributed to the continuous sampling approach taken at borehole R-6-1b, as opposed to the discontinuous sampling at B-601(DH). This pronounced difference is attributed to the reduced time exposure of soils in borehole R-6-1b to the effects of borehole cleaning prior to testing.

In summary, the difference between the initial and supplemental field investigation results can be attributed to:

- **Drilling/sampling requirements and approach, specifically the time exposure of soils to the effects of borehole cleaning prior to testing, i.e. continuous vs. discontinuous sampling, and**

- **The fact that the supplemental investigation's target was to reduce disturbance effects by avoiding unnecessary delays between sample locations, and to adopt a tight control on the consistency of circulating mud.**

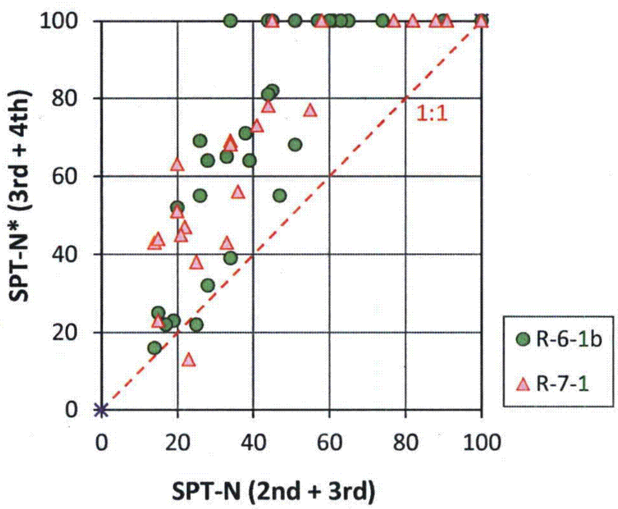
FSAR Subsection 2.5.4.13 will be revised in a future COLA revision as follows:

2.5.4.13 References

322. American Society of Civil Engineers/Structural Engineering Institute, ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures*, 2010.

The following figures will be added in a future revision of the COLA:

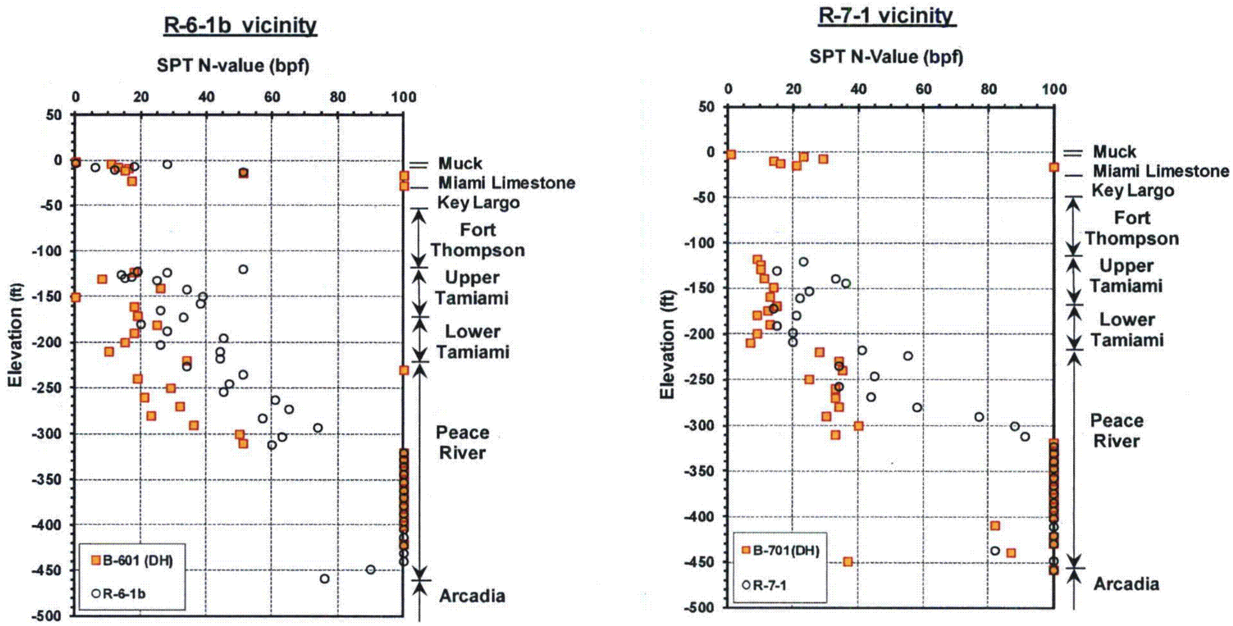
Figure 2.5.4-269 Plot of Uncorrected SPT N Values versus the Summation of 3rd and 4th Blow Counts from the Supplemental Investigation



Note:
* Summation of 3rd and 4th blow count intervals

Note: Data from Reference 290

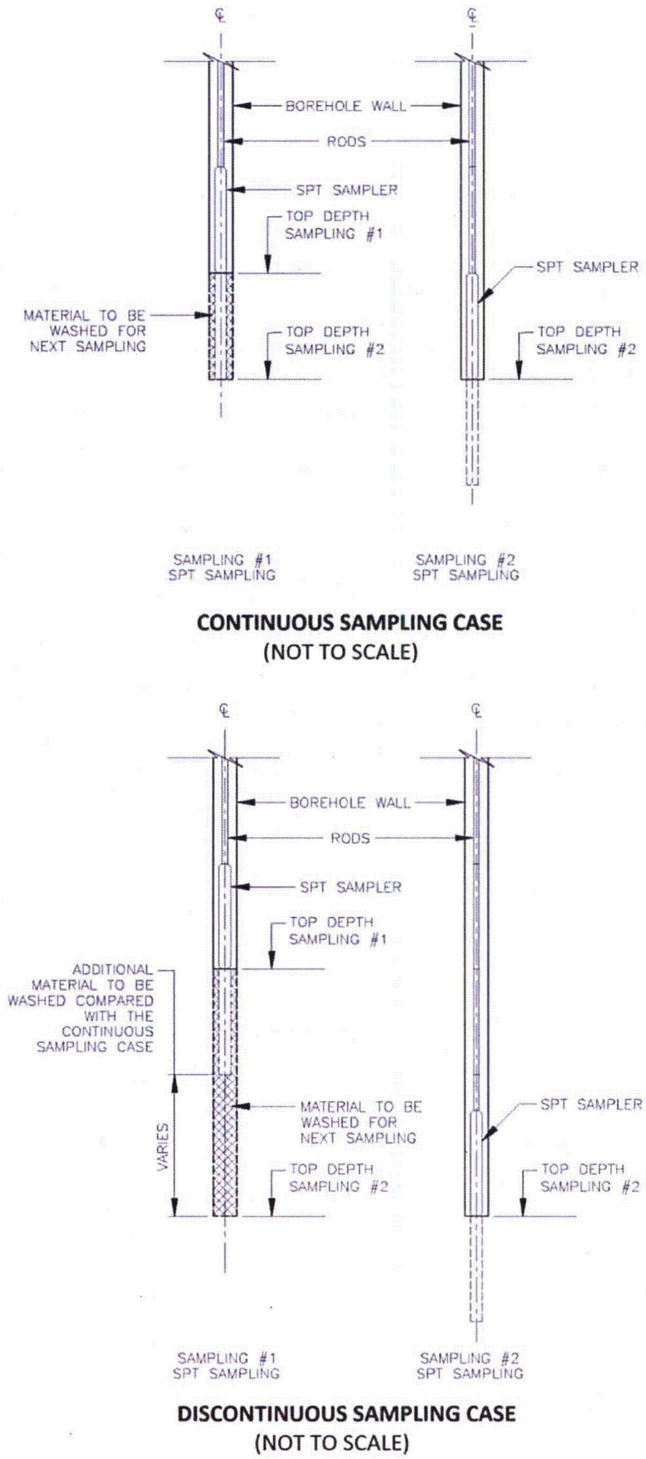
Figure 2.5.4-270 Plot of Uncorrected SPT N Values for R-6-1b and R-7-1 Vicinity



Data from References 257 and 290

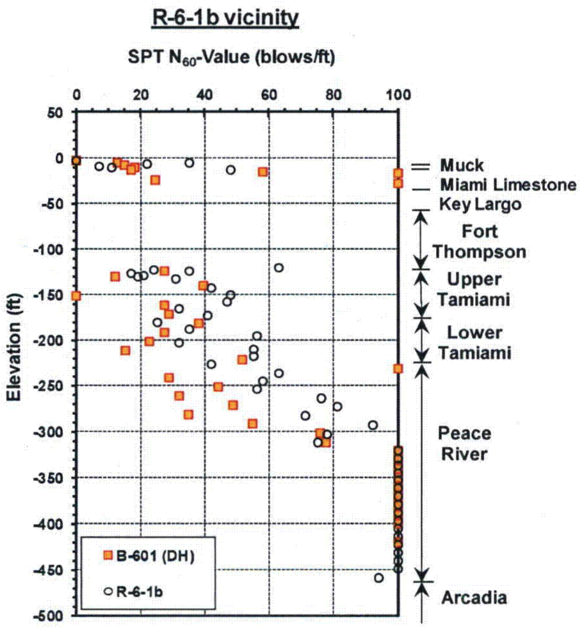
Notes: Elevation is NAVD 88

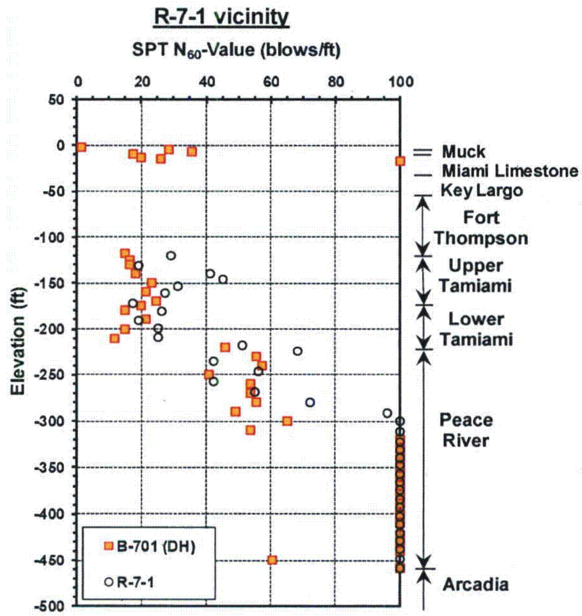
Figure 2.5.4-271 Continuous and Discontinuous SPT Sampling



Note: General scheme developed as sampling was performed according to References 257 and 290

Figure 2.4.5-272 Plot of Corrected SPT N_{60} Values for R-6-1b and R-7-1 Vicinity





Data from References 257 and 290

Notes: Elevation is NAVD 88

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-082

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-31 (eRAI 7811)

In response to RAI 02.05.04-12(d), the applicant indicated that the sulfate measured from groundwater samples classify the concrete exposure to sulfate attack as severe based on ACI, and proposed to make the first lift of concrete fill (bottom lift) from sulfate resisting Type V cement with maximum C3A content of 5 percent as specified in ACI 318-05/318R-05 to address the issue of concrete fill exposure to sulfate attack from groundwater. However, in additional to cement type, ACI 318-05/318R-05 also specify the maximum water-cementitious material ratio and minimum concrete strength for concrete to resist sulfate attacking. In accordance with 10 CFR 100.23, please specify what water-cementitious material ratio and concrete strength are to be used for the first lift of concrete fill, and provide corresponding updates into the FSAR to reflect the evaluation and prevention of groundwater chemicals attacking concrete fill.

FPL RESPONSE:

This response provides an update to the Response to RAI 02.05.04-12(d). The only update to the RAI 02.05.04-12(d) response is the commitment to the ACI 201.2R-08 instead of ACI 318-05/318R-05 for durability requirements, since the latter is more of a design code for reinforced concrete, and the former is actually guidance on the durability of concrete in general, which is the intended use.

The sulfate values measured from 24-water samples range from 2280 ppm to 4400 ppm, resulting in a median value of about 3800 ppm, or close to 0.4 percent by weight. This classifies the concrete exposure to sulfate attack as Class 2 exposure according to the ACI Guide to Durable Concrete, ACI 201.2R-08 (Reference 1).

Recommendations for improving sulfate resistance are provided in the ACI Guide to Durable Concrete, ACI 201.2R-08 (Reference 1). For the first lift of concrete (bottom lift), the requirements in Table 6.3 of Reference 1 for water-cementitious material ratio and type of cementitious materials will be followed in order to provide resistance to sulfate attack. The concrete mix for the first lift will contain a maximum water-cementitious material ratio by mass of 0.45 and a sulfate resisting Type V or equivalent as defined in Sections 6.2.5, 6.2.7, and 6.2.9 of the ACI Guide to Durable Concrete, ACI 201.2R-08 (Reference 1). In addition, Type V cement or equivalent according to ACI 201.2R-08 (Reference 1) will be used for all the lifts for additional protection. Additional details regarding to the design and construction approach for the lean concrete fill will be provided in the Response to RAI 03.08.05-03.

The Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC) that will be used to ensure that the first lift of concrete fill meets the ACI 201.2R-08 durability requirements are provided in Table 1. The ITAAC that will be used to ensure that the lean concrete fill meets specifications in ACI 207 are provided in the Response to RAI 02.05.04-33.

Table 1
ITAAC for Fill Concrete under Seismic Category I Structures

Design Commitment	Inspections, Tests, and Analyses	Acceptance Criteria
First lift of fill concrete placed under Seismic Category I structures meets durability requirements of ACI 201.2R-08 for Class 2 sulfate exposure.	Test, inspections, or combination of tests and inspection will be performed to determine the cement type and water-cementitious material ratio of the concrete mix for the first lift of fill concrete.	For the first lift of concrete fill, the cement is a sulfate resisting Type V cement (or equivalent as defined in ACI 201.2R-08) and that the maximum water-cementitious material ratio is 0.45.

This response is PLANT SPECIFIC.

References:

1. American Concrete Institute, *Guide to Durable Concrete* (ACI 201.2R-08), 2008.

ASSOCIATED COLA REVISIONS:

A new paragraph after the third paragraph in the FSAR Subsection 2.5.4.5.1.2 will be added in a future COLA revision as follows:

2.5.4.5.1.2 Power Block and Site Grade Raising

Structural fill consisting of excavated fill material is placed around but not below any nuclear island structure. Replacement material below the nuclear islands consists of lean concrete fill. The selection of lean concrete mix design is made at project detailed design. The compressive strength of 1.5 ksi is estimated for lean concrete fill.

The lean concrete fill exposure to sulfate attack from groundwater is classified as Class 2 exposure according to Reference 322. Recommendations for improving sulfate resistance are provided in the ACI Guide to Durable Concrete, ACI 201.2R-08 (Reference 322). For the first lift of concrete (bottom lift), the requirements in Table 6.3 of Reference 322 for water-cementitious material ratio and type of cementitious materials are followed in order to provide resistance to sulfate attack. The concrete mix for the first lift contains a maximum water-cementitious material ratio by mass of 0.45 and a sulfate resisting Type V cement or equivalent as defined in Sections 6.2.5, 6.2.7, and 6.2.9 of the ACI Guide to Durable Concrete

ACI 201.2R-08 (Reference 322). In addition, Type V cement or equivalent according to ACI 201.2R-08 (Reference 322) is used for all the lifts for additional protection.

FSAR Subsection 2.5.4.13 will be revised in a future COLA revision as follows:

2.5.4.13 References

323. American Concrete Institute, *Guide to Durable Concrete* (ACI 201.2R-08), 2008.

FSAR Subsection 14.3.3.5 will be added in a future COLA revision as follows:

14.3.3.5 Fill Concrete ITAAC

Subsection 2.5.4.5 discusses, in part, the excavations, backfill (including cementitious construction material) and earthwork analyses for Seismic Category I structures. The objective of this fill concrete ITAAC is to ensure reliable performance of the foundation bearing material over the life of the plant. Specifically, proper ITAAC are specified to ensure the first lift of concrete fill material is resistant to sulfate attack. By verifying water-cementitious material ratio and cement type, this ITAAC provides a method to confirm that sulfate-resistant properties of the fill material are achieved.

The following ITAAC will be added to the COLA, Part 10, Appendix B:

Table 3.8-5
Fill Concrete

Design Commitment	Inspections, Tests, and Analyses	Acceptance Criteria
First lift of fill concrete placed under Seismic Category I structures meets durability requirements of ACI 201.2R-08 for Class 2 sulfate exposure.	Test, inspections, or combination of tests and inspection will be performed to determine the cement type and water-cementitious material ratio of the concrete mix for the first lift of fill concrete.	For the first lift of concrete fill, the cement is a sulfate resisting Type V cement (or equivalent as defined in ACI 201.2R-08) and that the maximum water-cementitious material ratio is 0.45.

ASSOCIATED ENCLOSURES:

None

NRC RAI Letter No. PTN-RAI-LTR-082

SRP Section: 02.05.04 - Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-37 (eRAI 7811)

FSAR Subsection 2.5.4.5.4 discussed dewatering and excavation methods. However, for excavation methods, the applicant stated that "Excavation is performed with standard excavation equipment, but may be supplemented with other methods". In accordance with 10 CFR 100 and Regulatory Guide 1.206 Section C.I.2.5.4.5, please provide a detailed discussion on excavation methods, especially the methods for limestone excavation, with consideration of the installed reinforced diaphragm wall and grouting plug. Also provide more information of quality control and QA programs that will apply to foundation excavation and backfilling, and provide corresponding updates into the FSAR.

FPL RESPONSE:

The excavation limits for the nuclear island will be defined laterally by the boundary created by the reinforced concrete diaphragm walls (as shown on FSAR Figure 2.5.4-201) and vertically by the target elevation for the bottom of excavation (El. –35 feet North American Vertical Datum of 1988 [NAVD 88]), which has been selected by careful evaluation of geologic data and consideration of design details of the plant.

The excavation limits will be maintained through visual inspection and survey measurements as excavation proceeds. As the excavation proceeds, geologic mapping will be conducted on the exposed rock surfaces.

Excavation of materials above El. -35 feet (the bottom of excavation) is not classified as safety-related. Geologic mapping, inspection, and survey measurements will be conducted to meet 10 CFR 50 Appendix B and ASME NQA-1 quality requirements.

The Responses to RAIs 02.05.04-32 and 02.05.04-33 will discuss the Category I and lean concrete backfill material properties including proposed Inspection, Tests, Analyses, and Acceptance Criteria (ITAAC).

Excavation for the nuclear island will consist of excavation of the top layer of surficial sediments or "muck" and the underlying Miami Limestone down to approximately El. –35 feet to expose the Key Largo Limestone. Excavation of the muck will be accomplished with common excavation equipment such as bulldozers, loaders, track mounted excavators, etc. Excavation of the limestone rock will be accomplished using similar equipment for more weathered or poorly cemented rock (generally the Miami Limestone) and a variety of commercially available rock excavation tools such as ripping teeth, rock excavation buckets, and/or road header attachments for more intact

rock (generally the Key Largo Limestone). Drilling and blasting has been eliminated as a possible construction method.

The location of the diaphragm wall, and the corresponding horizontal extent of the excavation, will have established coordinates based on survey measurements. The bottom of the excavation will be monitored by geologic mapping and survey. In addition, excavator mounted tools such as rippers and road headers allow for precise control of the extents of the excavation because the operator, with the assistance of spotters as necessary, is able to maintain visual contact with the area being excavated. The average compressive strength of the Key Largo Limestone is 2689 psi; this is significantly lower than the anticipated compressive strength of the reinforced concrete diaphragm wall. This difference in material strength and continuity between the concrete and relatively weak, partly fractured rock will also create a different "feel" to the operator when encountered by the tool. These methods will ensure that the integrity of the diaphragm wall and grout plug is maintained throughout the excavation process. Measurement of the elevation of the excavation floor will be the primary means of preventing over excavation into the grout plug zone.

As discussed in the Response to RAI 02.05.04-14, inspection, geologic mapping, and surveying will be performed during excavation to ensure the complete removal of the Miami Limestone and that the required depth is reached.

A detailed excavation and foundation preparation plan will be developed prior to construction. While the excavation activities are not classified as safety-related, these plans will be treated as NQA-1 deliverables. The program is anticipated to include similar items to the following list:

Specification of excavation methods. As discussed above, it is anticipated that excavation methods will include mass excavation of soils (muck) and highly weathered or soft rock with conventional excavation equipment and ripping of moderately weathered rock with rock excavation tools.

- Quality control and quality assurance programs.
- Methods for dewatering and protection from degradation at the bottom of excavation during excavation and dewatering. Anticipated construction dewatering requirements are discussed in FSAR Subsection 2.5.4.6.2. It is not expected that the sound rock at the bottom of excavation will significantly degrade due to excavation, dewatering, or exposure to the elements during construction. Observable degraded or raveling rock at the bottom of excavation will be removed prior to placement of dental concrete or the mudmat.
- Specification of methods for construction dewatering, disposal of water, and management of seepage and piping.

- As the excavation proceeds, geologic mapping of the exposed rock will be performed by appropriately qualified geologist. This activity will be in accordance with NQA-1 requirements. Geologic mapping of the intermediate stages of the excavation will focus on major features such as prevailing fracture sets and bedding features in addition to general description of the exposed rock.
- The inspection and mapping of the completed excavations will be performed by appropriately qualified and trained project inspection personnel. Soundings, test holes, and similar measures will be used to augment visual identification of areas needing repairs and to document acceptance of corrective measures, as appropriate.
- The rock surface at the target excavation elevation (El. –35 feet) is expected to be undulatory and to require a variable amount of foundation preparation across the footprint, which may include additional excavation, supplemental grouting, placement of dental concrete or slush grout backfill, or other methods.

This response is PLANT SPECIFIC.

REFERENCES:

None.

ASSOCIATED COLA REVISIONS:

Text will be added after the second paragraph of FSAR Subsection 2.5.4.5.4 in a future COLA revision as follows:

The excavation limits for the nuclear island are defined laterally by the boundary created by the reinforced concrete diaphragm walls (as shown on Figure 2.5.4-201) and vertically by the target elevation for the bottom of excavation (El. –35 feet NAVD 88), which is selected by careful evaluation of geologic data and consideration of design details of the plant.

The excavation limits are maintained through visual inspection and survey measurements as excavation proceeds. As the excavation proceeds, geologic mapping is conducted on the exposed rock surfaces.

Excavation of materials above El. –35 feet (the bottom of excavation) is not classified as safety-related. Geologic mapping, inspection, and survey measurements are conducted to meet 10 CFR 50 Appendix B and ASME NQA-1 quality requirements.

Excavation for the nuclear island consists of excavation of the top layer of surficial sediments or "muck" and the underlying Miami Limestone down to approximately El. –35 feet to expose the Key Largo Limestone. Excavation of the muck is accomplished with common excavation equipment such as bulldozers, loaders, track mounted excavators, etc. Excavation of the limestone rock is

accomplished using similar equipment for more weathered or poorly cemented rock (generally the Miami Limestone) and a variety of commercially available rock excavation tools such as ripping teeth, rock excavation buckets, and/or road header attachments for more intact rock (generally the Key Largo Limestone). Drilling and blasting has been eliminated as a possible construction method.

The location of the diaphragm wall and the corresponding horizontal extent of the excavation have established coordinates based on survey measurements. The bottom of the excavation is monitored by geologic mapping and survey. In addition, excavator mounted tools such as rippers and road headers allow for precise control of the extents of the excavation because the operator, with the assistance of spotters as necessary, is able to maintain visual contact with the area being excavated. These methods ensure that the integrity of the diaphragm wall and grout plug is maintained throughout the excavation process. Measurement of the elevation of the excavation floor is the primary means of preventing over excavation into the grout plug zone.

Inspection, geologic mapping, and surveying are performed during excavation to ensure the complete removal of the Miami Limestone and that the required depth is reached.

A detailed excavation and foundation preparation plan is developed prior to construction. While the excavation activities are not classified as safety-related, these plans are treated as NQA-1 deliverables. The program includes similar items to the following list:

- **Specification of excavation methods. As discussed above, excavation methods include mass excavation of soils (muck) and highly weathered or soft rock with conventional excavation equipment and ripping of moderately weathered rock with rock excavation tools.**
- **Quality control and quality assurance programs.**
- **Methods for dewatering and protection from degradation at the bottom of excavation during excavation and dewatering. Construction dewatering requirements are discussed in Subsection 2.5.4.6.2. It is not expected that the sound rock at the bottom of excavation will significantly degrade due to excavation, dewatering, or exposure to the elements during construction. Observable degraded or raveling rock at the bottom of excavation is removed prior to placement of dental concrete or the mudmat.**
- **Specification of methods for construction dewatering, disposal of water, and management of seepage and piping.**
- **As the excavation proceeds, geologic mapping of the exposed rock is performed by appropriately qualified geologist. This activity is in accordance with NQA-1 requirements. Geologic mapping of the intermediate stages of the excavation is focused on major features such as**

prevailing fracture sets and bedding features in addition to general description of the exposed rock.

- **The inspection and mapping of the completed excavations is performed by appropriately qualified and trained project inspection personnel. Soundings, test holes, and similar measures are used to augment visual identification of areas needing repairs and to document acceptance of corrective measures, as appropriate.**
- **The rock surface at the target excavation elevation (El. –35 feet) is expected to be undulatory and to require a variable amount of foundation preparation across the footprint, which may include additional excavation, supplemental grouting, placement of dental concrete or slush grout backfill, or other methods.**