3.5 GEOLOGY, SOILS, AND GROUNDWATER

The analyses conducted in this section of the Tier II Draft EIS focus on the potential impacts on adjacent resources. Potential impacts of natural resources on-site were addressed. The Tier II analyses determined that there would be no adverse impact on on-site soil stability or soils adjacent to the Washington Monument during construction and operation of the museum. Impacts on geology would be less than significant because no significant geologic features were found on-site. Groundwater within the project site would be isolated from groundwater surrounding the site from the use of diaphragm slurry walls or another form of Support of Excavation (SOE) System. Groundwater would be captured and channeled to storm drains or other sewer disposal systems at a rate that would not induce settlement within the project site or at nearby structures. The overall quality of groundwater would not be degraded beyond its current condition (Smithsonian Institution 2008a).

The Tier I Final EIS determined that the soil at the site is predominantly fill installed in the mid-19th century when sewers were networked with Tiber Creek. The fill at the site ranges from a depth of 5.5 feet to 17 feet below the surface. Below the fill are layers of clayey sands or stiff plastic clays mixed with sand and gravel. No significant geologic features were found on the site (Smithsonian Institution, 2008a).

Groundwater at the site is recharged through precipitation, percolation and from infiltration of the Potomac River. The on-site soils are compacted due to heavy visitor use therefore limited percolation is expected. The site's proximity to the Potomac River provides the potential for abundant groundwater recharge (D.D. WRRC 1995). No site specific groundwater study was conducted for this project . Instead, the impact analysis assumed conditions similar to those observed at adjacent sites. Groundwater at the site was expected to be 15 to 25 feet below the surface. Due to the underlying clays, the Tier I analysis anticipated groundwater movement to be slow.

As evaluated in the Tier I Final EIS, impacts on soils would occur from the action alternatives as a result of construction activities and site preparation, resulting in soil disturbance, compaction, soil excavation, and the loss of soil productivity. Impacts on soils would be minimized through the implementation of an approved erosion and sediment control plan, pursuant to the District of Columbia's Soil Erosion and Sediment Control Program (Erosion and Sediment Control Act of 1977). The Tier I Final EIS called for additional analyses to be undertaken in order to better understand existing conditions and expected impacts. Sixteen test borings were collected during the period from February 2 to March 10, 2010, under the full observation of Froehling & Robertson, Inc, and were compared to previous borings conducted in the immediate area. Froehling & Robertson completed their analytical studies in April 2010, and the results are incorporated in the analysis in this Tier II document.

3.5.1 What are the existing sub-grade conditions of the site and surrounding area?

The general subsurface profile encountered at the site consists of organic soils, overlying fill, coastal plain soils, and residual soils. Table 3.5.1 summarizes the on-site soil conditionson-site.

Organic Soils

The soil surface layer consists of approximately two (2) to seven (7) inches of surficial organic soils. Surficial organic soil is typically a dark-colored soil material containing roots, fibrous matter, and/or organic components. It is generally unsuitable for engineering purposes (Froehling & Robertson, 2010).

Fill Materials

Fill materials were found below the surficial organic soils to depths ranging from 5.5 to 17 feet below the surface (elevation 5.4 to -7 feet AMSL). The fill materials consisted of sandy gravel, silty sand, clayey sand, sandy silt, and lean clay soils.

The Standard Penetration Test (SPT)[7] conducted in general accordance with American Society of Testing and Materials D 1586 indicated that the soils in this stratum are a very soft to hard consistency, or a very loose to very dense state. The SPT N-values recorded ranged from the Weight of Hammer to 50 blows per one inch of sampler penetration. Elevated SPT N-values in these soils can be attributed to varying amounts of gravel within the stratum. An average SPT N-value of 10 blows per foot (bpf) was recorded within this stratum (Froehling & Robertson, 2010).

Coastal Plain

Coastal Plain materials are present underneath the fill materials to depths ranging from 52 to 75 feet below the surface (elevation -43.5 to -59.5 feet AMSL). Coastal Plain deposits consisted of sandy gravel, silty gravel, gravel with silt and sand, gravelly sand, gravelly sand with silt, sand with silt, silty sand, sand, clayey sand, sandy silt, silty clay, lean clay, and fat clay.

An average SPT N-value of 20 bpf was recorded for the granular soils in this stratum indicating a very loose to very dense state. An average SPT N-value for 14 bpf was recorded for the cohesive soils in this stratum (Froehling & Robertson, 2010).

Residual Soils

Residual soils form due to in-place weathering of the parent rock. They were encountered below the Coastal Plain soils in 6 of the 16 borings, ranging from 62 to 83 feet below the surface (elevation -48.5 to -71 feet AMSL). These soils consist of sandy silt, elastic silt, and silty sand. An average SPT N-value of 24 bpf was recorded for the granular soils in this stratum. An SPT N-value of 14 bpf was recorded for the cohesive soils in this stratum, indicating a stiff consistency (Froehling & Robertson, 2010).

[7] In the SPT test, a split-spoon sampler is driven into the soil by freely dropping a weight of 140 pounds from a height of 30 inches. The number of blows that are needed to drive the split-spoon sampler three consecutive 6-inch increments is recorded. The Standard Penetration Resistance (N-value) is the sum of blows from the last two six-inch increments. The N-value provides a general indication of in situ soil conditions (e.g., consistency) and is correlated with certain soil engineering properties.

Table 3.5.1 Site Soil Characteristics

Soil	Components	Bottom of Stratum (below grade)	Average SPT N-Value	Consistency
Organic soils	Containing roots, fibrous matter, and/or organic components	2 to 7 inches	n/a	Unsuitable for engineering purposes
Fill materials	sandy gravel, silty sand, clayey sand, sandy silt, and lean clay	5.5 to 17 feet	10 bpf	Very soft to hard, very loose to very dense
Coastal Plain	gravel, silty gravel, gravel with silt and sand, gravelly sand, gravelly sand with silt, sand with silt, silty sand, sand, clayey sand, sandy silt, silty clay, lean clay, and fat clay	Underlying fill materials to depths of 52 to 75 feet	20 bpf (granular soils) 14 bpf (cohesive soils)	Very loose to very dense (granular soils) Very soft to hard (cohesive soils)
Residual soils	sandy silt, elastic silt, and silty sand	62 to 83 feet	24 bpf (granular soils) 14 bpf (cohesive soils)	Very loose to very dense (granular soils) Stiff (cohesive soils)
Decomposed Rock	Sampled as silty gravel, sand, silty sand, and sandy silt	62 to 83 feet	In excess of 60 bpf	Very Dense
Rock	schist	NA	NA	Moderate to highly weathered, moderately to highly fractured

Note: Based on field data, a Seismic Site Class D was established for the site per Section 1613.5.3 of the 2006 International Building Code.
Source: Froehling & Robertson, 2010

3.5.2 What are the site's current geologic conditions and what is the depth to bedrock?

The general geologic conditions encountered at the site consist of decomposed rock and bedrock.

Decomposed Rock

The majority of test borings encountered decomposed rock below the Coastal Plain and/or Residual soils. Decomposed rock, for the purposes of this analysis, is defined as residual material with an average SPT-N value in excess of 60 bpf, indicating that it is very hard or dense. Decomposed rock can be more difficult to excavate than the residual soils. Decomposed rock encountered at the site consisted of silty gravel with sand, sand with silt and gravel, silty micaceous sand, sandy micaceous silt, and elastic silt with trace rock fragments

Weathering of the parent bedrock is generally more rapid near fracture zones, and therefore, the bedrock surface may be irregular. The difference in weathering may also result in areas of rock and decomposed rock appearing within residual soils. (Froehling & Robertson, 2010).

Bedrock

Rock was generally encountered at a depth of 62 to 96 feet below the surface. Seven borings were extended five to ten feet into rock. Bedrock encountered on site consisted of gray to olive brown, moderately to highly weathered, moderately to highly fractured, micaceous shist. A layer of quartz rock was encountered within one test boring. Rock recovery values recorded at the site ranged from 35 to 100 percent. Rock Quality Designation values recorded ranged from zero to 87 percent (Froehling & Robertson, 2010).

3.5.3 What is the depth to groundwater at the site?

Based on subsurface water observations conducted at wells and through test borings, Froehling & Robertson estimate the groundwater was at an elevation of five feet below sea level (elevation -5 feet) during the field exploration. This represents an approximately one foot rise in the ground water level recorded from their May 2009 exploration. This difference can be attributed to the elevated rain and snow totals experienced during the time of the field exploration in February 2010. Generally, seasonal and yearly fluctuations of the water table should be expected with variations in precipitation, surface runoff, evaporation, and other similar factors.

3.5.4 How would construction and operation of the NMAAHC affect Geology, Soils and Groundwater?

For the purpose of defining whether any of the proposed alternatives could potentially affect the geology and soils of the site, several criteria are considered.

> **No Impact:** The geology or soils of the site would not be impacted or the impact to these resources would be below or at the lower levels of detection.
>
> **No Significant Impact:** Impacts would be detectable. Mitigation would be needed to offset adverse impacts and would be relatively simple to implement and would likely be successful.
>
> **Significant Impact:** Impacts would be readily apparent and result in a change to the character of the resource over a relatively wide area. Mitigation needed to offset adverse impacts may or may not be successful.

Short-term impacts would occur during construction of the action alternatives and were addressed in the Tier I Final EIS (Smithsonian Institution 2008a). Long-term impacts would occur during operation of the NMAAHC.

For the purposes of analyzing the impacts of the four action alternatives, it is assumed that all four alternatives would extend to an elevation at about -32.0 feet AMSL. The building would be primarily column supported, with maximum column loads ranging from about 350 to 400 tons. Total settlements on the order of one inch with differential settlements of less than 0.5 inches were considered acceptable for design (Froehling & Robertson, 2010).

No Action Alternative

The No Action Alternative would not result in any changes to the project site or new development on the site. As such, there would be no short- or long-term impacts on geology, soils and groundwater.

Action Alternative 1: Plinth Concept

Geology and Soils

The primary concern with construction adjacent to the Washington Monument would be the potential to induce settlement of the structure's foundation. The Washington Monument has a shallow foundation system bearing at an elevation of approximately two feet AMSL. The foundation is based on a compressible clay stratum located between the monument foundation and bedrock, which extends to an elevation of approximately -60 feet AMSL. Additional loading of the compressible clay could cause additional settling.

The Washington Monument lies approximately 669 feet southwest of the NMAAHC site. A study conducted in 1962, the results of which have been reviewed and confirmed as recently as 2002, outlined soil loading parameters for areas near the Washington Monument (NPS, 1962; Lacy, 2002). The reports addressed allowable permanent net increase and allowable permanent net decrease of soil loading, as well as allowable excavation for areas within 200 feet of the Washington Monument. The study did not specify parameters for sites over 200 feet from the Washington Monument.

In addition, other structures, including NMAH and the Herbert C. Hoover Commerce building are located less than 500 feet from the NMAAHC site. Disturbance of soils on the NMAAHC could potentially cause settlement of any adjacent structures. However, since all building loads for the Plinth Alternative would be founded on deep foundations extended to bedrock, the construction would not cause load changes in the soils founding the Washington Monument or other nearby structures (Froehling & Robertson, 2010). As such, there would be no adverse impact on on-site soil stability or soils adjacent to the Washington Monument during construction of the Plinth Alternative. Impacts on geology would be less than significant because no significant geologic features were found on-site.

Groundwater

During site observation, subsurface groundwater registered at an elevation of approximately -5 feet AMSL within the observation wells. Therefore, groundwater levels were close to the surface level. Subsurface water levels and soil moisture would likely fluctuate due to changes in precipitation, runoff, and season. Groundwater fluctuations within five feet are considered normal. Changes in groundwater levels greater than 10 feet could also cause stress changes within soils on the project site and adjacent properties.

A temporary dewatering system would be used to isolate the building area and dispose of groundwater that would be encountered during construction.

Following construction, continuous dewatering of the site is not anticipated to be necessary due to the fact that groundwater flows on the site would be permanently diverted by diaphragm slurry walls[8] and because the soils beneath the groundwater table are primarily clays that prohibit rapid movement of groundwater. Due to the depth of the excavation, as well as the amount of groundwater expected to be encountered, a diaphragm slurry wall would be utilized. Groundwater would be captured and channeled to storm drains or other sewer disposal systems at a rate that would not induce settlement within the project site or at nearby structures.

---

[8] or another form of Support of Excavation (SOE) System that could use jet grout and secant.

The volume of the proposed structure occurring below the groundwater table would impede groundwater flows and could cause minor variations in the depth of groundwater to occur within the immediate vicinity of the proposed structure. The depth of the groundwater table would fluctuate and rise on the up gradient side and lower on the down gradient side. Variations in groundwater depth would return to normal levels as the water moves farther away from the structure. The overall quality of groundwater would not be degraded beyond its current condition. Groundwater within the project site would be isolated from groundwater surrounding the site from the use of slurry walls or another form of SOE System (Smithsonian Institution 2008a).

Diaphragm Slurry Wall System

The design of the Plinth Alternative could include a diaphragm slurry wall (slurry wall) system that would create a hydraulic break between the interior of the building site and the surrounding areas. No groundwater level change is expected beyond the limits of construction for the new building.

The diaphragm slurry wall system would be paired with a "dewatering" system. In this system, deep dewatering wells or a well point system would remove the groundwater to the desired level. The water would typically disposed of through a sewer system and/or water canals. However, due to the required drawdown level of -32 feet AMSL which would likely result in ground settlement, a slurry wall system could be used to isolate the building area from the surrounding soils (Froehling & Robertson, 2010).

A diaphragm slurry wall is a cast-in-place structural concrete wall, formed utilizing the slurry-supported trench method. Using this method, construction equipment is used to dig trenches, which are formed in individual sections called panels. Once a panel is dug, a bentonite and/or polymer slurry is deposited in the panel to provide support and avoid collapse of the panel's earthen wall. Reinforcing steel is placed into the slurry. Concrete is then funneled into the panel, which displaces the slurry and forms the wall.

The purpose of the diaphragm wall system would be to surround the site and buffer the Plinth Alternative from the surrounding soil and groundwater level. Diaphragm walls have been shown to provide a rigid earth retention system, as well as provide a hydraulic barrier between the surrounding groundwater and the interior building area (Froehling & Robertson, 2010).

Based on the recommendations in the Preliminary Geotechnical Engineering Evaluation completed by Freohling & Robertson, the slurry walls should be a minimum of 36 inches thick, extend a minimum of 5 feet into the decomposed rock stratum, and be designed to withstand the lateral earth pressure that would be encountered during construction. Once the diaphragm wall is installed, the building area would be dewatered without inducing groundwater drawdown of the surrounding areas (Froehling & Robertson, 2010).

For these reasons, neither construction nor operation of the Plinth Alternative would create settlement of on-site buildings or adjacent structures. No significant short-term or long-term impact on soils or groundwater would occur as a result of the Plinth Alternative. Impacts on geology would be less than significant because there are no significant geologic resources on-site.

The analysis conducted for the Tier I EIS assumed that the Plinth Alternative would be designed as a "bath tub" structure, meaning that once construction has been completed, the temporary dewatering system surrounding the site would be discontinued and water would be allowed to return to hydrostatic conditions. No groundwater level change would occur beyond the limits of construction (Froehling & Robertson, 2010).

Foundation Systems

Determination of an appropriate foundation system for a given structure is dependent upon the structural loads, soil conditions and construction constraints, such as proximity to other structures. A variety of foundation systems were evaluated for long-term support of the building, including driven piles and drilled shafts. Froehling & Robertson concluded that driven piles would be the most efficient and economical foundation system for the Plinth Alternative. However, this foundation system was dismissed because of public concerns about disruption to adjacent structures due to vibrations, as well as noise concerns from pile driving on the National Mall. As an alternative, a system of drilled shafts and subfloor drainage was recommended and is described below (Froehling & Robertson, 2010).

In order to address the foundation appropriately, shafts would be drilled into the bedrock. These shafts would extend into the rock material one shaft pile diameter, a minimum of 36 inches, or to caisson drill refusal level. Test borings indicate rock layers range from -62 to -96 feet AMSL.

Analysis indicated that total settlements for the drilled shafts would be less than one inch, with differential settlements up to about one-half of the estimated total settlement. They would vary based on the changes in excavation requirements across the building footprint, the distribution of loads, differences in column spacing and loads, and the variability of underlying soils (Froehling & Robertson, 2010).

The Plinth Alternative would also incorporate basement floor slabs in the building design. Basement floor slabs would be designed as a structural slab system supported by the deep foundation system and/or grade beams. The lowest slab would be constructed approximately 27 feet below the observed ground water level on the site. The floor slab would be designed to resist uplift pressures created by the groundwater. Horizontal waterproofing would be used below the floor slab to prevent cracking and groundwater intrusion into the building (Froehling & Robertson, 2010).

Additionally, a permanent under-slab drainage system would be installed on-site. This would address groundwater intrusion through the decomposed rock and rock stratums that would exert force to raise the building structure. Additionally, a drainage system would be incorporated into all surface retaining walls to prevent the unanticipated buildup of hydrostatic pressures (Froehling & Robertson, 2010).

The combination of drilled shafts, basement floor slabs and under-slab drainage system would prevent groundwater intrusion in the Plinth Alternative, settlement of the structure or floating during the long-term operation of the Plinth Alternative. No significant impact on the building structure from groundwater would occur. Additionally, there would be no significant impact on groundwater or groundwater quality. Potential impacts on geology would be less than significant from installation of the building as there are no significant geologic features on-site.

Action Alternative 2: Plaza Concept

Because the geologic, soil and groundwater conditions at the site would be the same with the Plaza Alternative as the Plinth Alternative, the Plaza Alternative would have the same effects as described above. Since building loads for the Plaza Alternative would be founded on deep foundations extended to bedrock, the construction of the NMAAHC would not cause load changes in the soils founding the Washington Monument or other nearby structures. Further, the Plaza Alternative would incorporate a SOE System (e.g., a diaphragm slurry wall system) and a dewatering system to ensure that building loads on-site and the adjacent properties would not be affected by groundwater induced settlement. There would be no adverse impact on the stability of on-site soils during construction or operation of the Plaza Alternative. There would be a less than significant impact on geology because there are no significant geologic features on-site.

As with the Plinth Alternative, the Plaza Alternative could be subject to groundwater intrusion or a floating building during long-term operation. The Plaza Alternative foundation would include drilled shafts, basement floor slabs, and permanent under-slab drainage system. This combination would ensure no significant long-term impact on the building structure from groundwater. Additionally, there would be no significant impact on groundwater or groundwater quality.

Action Alternative 3: Pavilion Concept

Because the geologic, soil and groundwater conditions at the site would be the same with the Pavilion Alternative as the Plinth and Plaza Alternatives, the Pavilion Alternative would have the same effects as described above. Since building loads for the Pavilion Alternative would be founded on deep foundations extended to bedrock, construction of the NMAAHC would not cause load changes in the soils founding the Washington Monument or other nearby structures. Further, the Pavilion Alternative would incorporate a SOE System (e.g., a diaphragm slurry wall system) and a dewatering system to ensure that building loads on-site and the adjacent properties would not be affected by groundwater induced settlement. There would be no adverse impact on the stability of on-site soils during construction or operation of the Pavilion Alternative. There would be a less than significant impact on geology because there are no significant geologic features on-site.

As with the Plinth and Plaza Alternatives, the Pavilion Alternative could be subject to groundwater intrusion or a floating building during long-term operation. The Pavilion Alternative foundation would include drilled shafts, basement floor slabs, and permanent under-slab drainage system. This combination would ensure no significant long-term impact on the building structure from groundwater.

Action Alternative 4: Refined Pavilion Concept

Because the geologic, soil and groundwater conditions at the site would be the same with the Refined Pavilion Alternative as the other Alternatives, the Refined Pavilion Alternative would have the same effects as described above. Since building loads for the Refined Pavilion Alternative would be founded on deep foundations extended to bedrock, the construction of the NMAAHC would not cause load changes in the soils founding the Washington Monument or other nearby structures. Further, the Refined Pavilion Alternative would incorporate a SOE System (e.g., a slurry wall system) and a dewatering system to ensure that building loads on-site and the adjacent properties would not be affected by groundwater induced settlement. There would be no adverse impact on the stability of on-site soils during construction or operation of the Refined Pavilion Alternative. There would be a less than significant impact on geology because there are no significant geologic features on-site.

As with the other action alternatives, the Refined Pavilion Alternative could be subject to groundwater intrusion or a floating building during long-term operation. The Refined Pavilion Alternative foundation would include drilled shafts, basement floor slabs, and permanent under-slab drainage system. This combination would ensure no significant long-term impact on the building structure from groundwater. Additionally, there would be no significant impact on groundwater or groundwater quality.

3.5.5 What efforts would be taken to minimize the impacts on Geology, Soils and Groundwater?

The potential impacts on geology, soils, and groundwater would be minimized to no impact or a less than significant impact with the incorporation of Best Management Practices to control soil erosion and stormwater runoff during site preparation and construction, and with the incorporation of design measures described above in this section. It is assumed that the following procedures would be adhered to during building construction.

Site Preparation

- Any surficial soils and other deleterious non-soil material, such as asphalt or concrete, should be removed from the proposed construction area. Positive surface drainage should be maintained to prevent the accumulation of water.
- Underground utilities should be re-routed to locations a minimum of 10 feet or greater outside of the proposed building footprint.
- Areas intended to support new fill and pavements should be evaluated by a geotechnical engineer. The potential need for and extent of undercutting and in-place stabilization required can best be determined by a geotechnical engineer at the time of construction.

Diaphragm "Slurry" Wall Construction

- Prior to construction, the project engineer and structural engineer should review all diaphragm wall design plans and specifications. Construction plans should include guide wall details, a panel excavation sequence, and slurry mix design.
- The guide wall is recommended to be at at least 4 feet above the groundwater level. The slurry in the panel door should be desanded prior to placement of the concrete so that concrete placement does not create pockets of sand.
- A minimum concrete strength of 4,000 psi should be used with concrete slumps ranging from eight to ten inches for slurry wall construction.
- The project geotechnical engineer should review all pile design plans and specifications. Compressive load tests should be conducted according to ASTM D-1143. The load test should be conducted prior to construction to confirm that the contractor's construction methods and installation equipment can produce a foundation that will perform satisfactorily.
- The project geotechnical engineer should be retained to observe and document all field activities and develop recommendations for production pile driving criteria.

Drilled Shaft Construction

- If non-slurry or "dry" drilling methods are utilized, temporary steel casing should be installed in the drill hole of each caisson to keep the hole from collapsing. This would also allow workers to excavate, clean, and inspect the drilled shaft prior to placement of concrete. Soft or loose soil should be cleaned out of the bottom of the caisson prior to placement of reinforcing steel and concrete.

- The steel casing should not be removed until there is a sufficient head of concrete at the bottom of the casing to prevent slurry, water, or loose material from entering the excavation and creating a zone of weakness in the shaft.

- Installation records including drilling effort and drilling times associated with the final three feet of installation should be recorded. The time should include the penetration rate of the auger to determine refusal on bedrock.

- After the hole is completed, concrete should be placed as soon as possible and result in complete filling of the excavation without segregation.

Support of Excavation System

- To limit the effects of excavation and construction on adjacent structures, the use of a rigid Support of Excavation (SOE) system should be employed. The SOE system would function two ways: (1) it would allow for excavation and construction of the building, and (2) it would provide a permanent groundwater cutoff between the building and the surrounding area. The intent of the cutoff wall would be to greatly reduce the amount of groundwater intrusion into the site, allowing for interior dewatering utilizing a conventional subdrainage pumping system. The SOE system would need to be keyed into the decomposed rock layer on site and would need to be comprised of very low permeable materials. Embedment into the decomposed rock would greatly reduce groundwater flow around the wall itself, while the low permeable materials would minimize groundwater flow through the core (Froehling & Robertson, 2010).

- In order to prevent excess pressure on surface retaining walls, heavy equipment should not operate within five feet of below-grade walls. Footings or other surcharge loads should be evaluated to ensure excessive stress is not exerted.

Controlled Structural Fill

- Because some landscape elements would likely use structural fill, either on-site soils or an off-site source having a classification of silty gravel, gravelly sand, sand, silty sand, clayey sand, lean clay, or sandy silt should be used. Controlled structural fill should be free of boulders, organic matter, debris, or other deleterious materials with a maximum particle size not greater than three inches. Fill soils should have a maximum liquid limit of 45 and plasticity of less than 20.

- If construction traffic or weather disturbs the subgrade, the upper 8 inches of soils used for structural support should be scarified and recompacted. Each lift of fill should be tested to confirm that the recommended degree of compaction is attained. In utility trenches and other confined areas, potable compaction equipment and thin lifts of three to four inches may be required to meet specified degrees of compaction.

- Moisture content of fill soils should be within two percentage points of the optimum moisture content as determined from the standard Proctor density test, ASTM D 698. The contractor should have equipment on-site during earthwork for both drying and wetting of fill soils.

Subsurface Water Conditions

- Subsurface water is water existing below the ground surface. Groundwater at the site should be maintained a minimum of two feet below the bottom finished floor elevation of the building. This would allow construction to be conducted in dry conditions. Installation of vertical and horizontal water proofing for the building should be installed.

- A system of monitoring wells would be installed and recorded during construction. These wells would be used to demonstrate that the dewatering activities would be constrained to the site area and would not induce stress changes below adjacent structures. Additionally, the slurry wall contractor would install a groundwater reinjection system. This system would be used if groundwater depressions are observed during construction.

Monitoring and Contingency Plan

- A monitoring and contingency plan would be developed to monitor the site and the surrounding areas during construction. These plans would include a preconstruction survey indentifying the current conditions of adjacent structures prior to NMAAHC construction activities. The plan would specify instrumentation to be used on site, as well as threshold levels and monitoring frequency associated with each instrument. At a minimum, optical survey points would be placed along the SOE so that if any movement in the area is recorded, actions may be taken. Inclinometers and other geotechnical instrumentation have previously been used in diaphragm wall projects to measure deflections of the wall during construction.

This page is intentionally left blank