SOIL NAIL AND JET GROUTED EXCAVATION SUPPORT WALL AT PEIRCE MILL DAM

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As part of the Woodrow Wilson Replacement Bridge project several damaged wetland and environmental areas were to be mitigated. One of the mitigation projects consisted of constructing a Denil fish ladder around Peirce Mill Dam in Rock Creek National Park, Washington, D.C. The dam is a historic structure operated by the National Park Service. The fish ladder is located between the east abutment of the dam and Beach Drive, a major collector road. Construction of the fish ladder required removal of a portion of the east abutment and an excavation that extended about 22-ft below the pavement of Beach Ave. It was not possible to close Beach Drive except for very brief intervals or at night; therefore, a support of excavation system that would minimize disruption to the public was required. The high groundwater required that the groundwater be cut off prior to excavation; the random rubble and soil fill, and shallow but erratic rock surface precluded driving sheet piling. A jet grout system was selected because the grout probe could penetrate the rubble fill, stabilize a variety of soil conditions, and cut off the ground water prior to excavation. It was not possible to construct a stable wall without tiebacks or soil nails. Given the schedule and tight working conditions it was decided that soil nails through the jet grout wall facing would be the most cost effective method, as it did not reduce the working space as rakers would and would be more cost effective than prestressed tiebacks.

1.0 INTRODUCTION

This enhancement activity is a portion of the environmental mitigation plan for the Woodrow Wilson Bridge Project. This and other projects in Maryland and Virginia are intended to offset environmental impacts to natural resources associated with construction of the new Woodrow Wilson Replacement Bridge over the Potomac River and its contributing interchanges. This project is a result of the cooperative efforts of the National Park Service (NPS), the Maryland State Highway Administration (MSHA), the Federal Highway Administration (FHWA), the Virginia Department of Transportation (VDOT), the District of Columbia Department of Health-Fisheries Division and the Smithsonian National Zoological Park.

The Rock Creek stream restoration goal is to restore upstream fish migration in Rock Creek by removing or modifying existing in-stream fish barriers. Restoration will include the removal of abandoned fords and sewer lines, modification of existing fords, creation of natural pools and the construction of a Denil fishway at Peirce Mill Dam.

Peirce Mill was built in the 1820's, and operated commercially until 1897. The United States Government acquired the mill as part of Rock Creek Park in 1892, and the NPS restored it as a working mill in 1936. Peirce Mill became famous as the only 19th century gristmill operating full time in the NPS system. The site operated off and on until April 1993. NPS (2006)

1.1 Site Description

Pierce Mill Dam is approximately 400 feet upstream and north of the Tilden Street crossing in Rock Creek Park as shown in Figure 1. Beach Drive, located on the east bank of Rock Creek, is a two lane collector road that runs north to south and carries heavy commuter traffic.

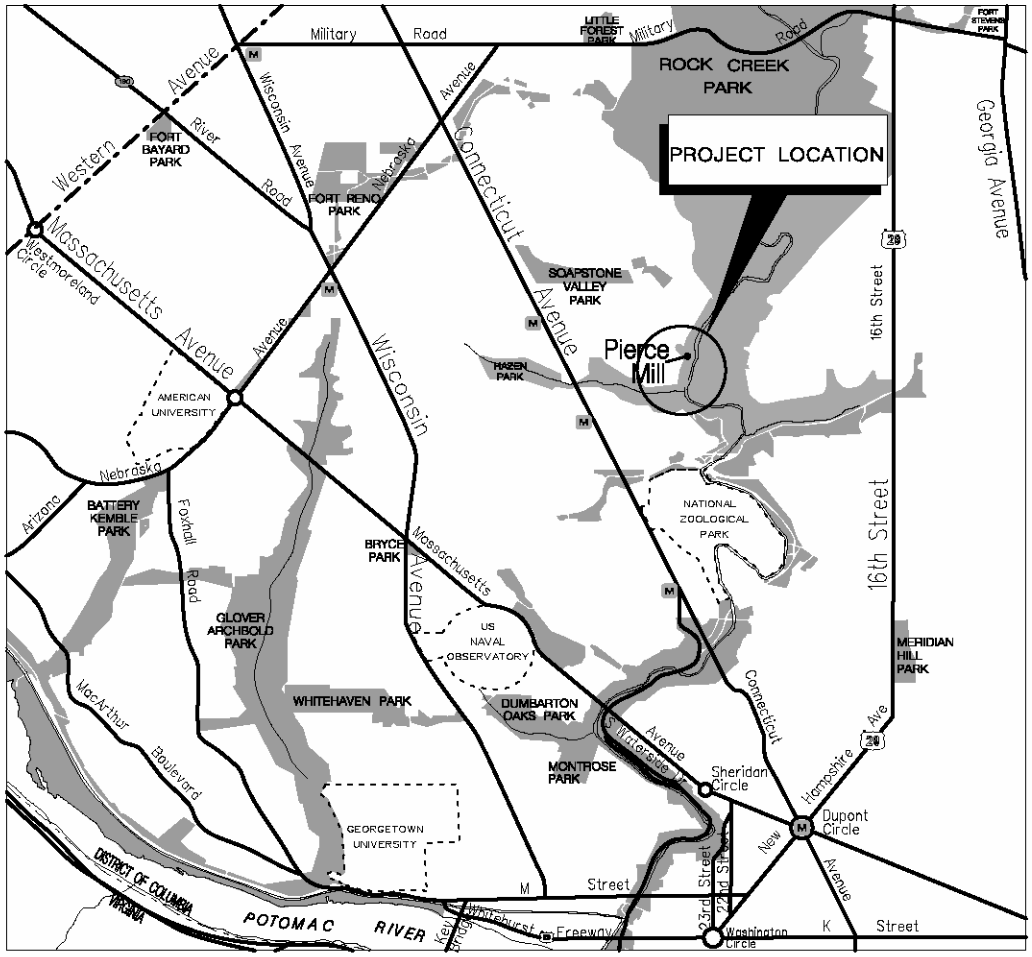

Figure 1 – Project Vicinity Map

Pierce Mill Dam is aligned east to west and is a stone masonry dam which is 8 feet in height, 8 feet in top-width, and 125 feet long between stream banks. Components of the dam include a 43 foot long jointed ashlar masonry spillway located near the middle of the dam, between two grouted boulder abutments. The creek flows from the north to the south, with the spillway at EL 57.7 with the tailwater near EL 53. All dam components have a concrete footing founded on bedrock, at or below EL 50.

The topography within the area of the temporary excavation support system and the Denil fishway ranges from about EL 50 in the creek basin to EL 69 along Beach Drive.

Upstream banks were eroded earth. The east downstream bank adjacent to Beach Drive consists of a stone masonry retaining wall extending approximately 10 feet south of the east abutment, which transitioned to a gabion retaining wall. Near the existing wing wall, the interface of the stone masonry wall and the gabion wall, an eroded void formed in the bank.

1.2 Project Description

The approved concept was to provide fish passage by constructing a Denil fishway between Beach Drive and the east abutment of Pierce Mill Dam. The fishway consists of a reinforced cast in place concrete open top culvert with a stone masonry veneer upstream of the dam, and a reinforced concrete wall with a concrete strut, stone masonry veneer and a transition to a dry stack stone masonry wall downstream of the dam. A Denil fishway uses a series of symmetrical close-spaced baffles in a channel to redirect and slow the flow of water, allowing fish to swim around the barrier.

To construct the fishway and dry stack retaining wall, a temporary excavation support system was required to support Beach Drive and allow vehicular traffic to be maintained during construction. The temporary excavation support system has a total length of approximately 115 feet. The excavation support system consists of two segments. Segment one is approximately 30 feet long, parallel to the proposed fish ladder, at a skew angle to Beach Drive. The second segment of the wall is approximately 85 feet long parallel to and directly supporting Beach Drive.

Figure 2 depicts the plan view of the proposed construction of the Denil fishway and the temporary excavation support structure. The wall height is approximately 17 feet along the first segment, but increases to 21 feet with a top of wall elevation near EL 69 where Segment one meets Segment two. The base of the second segment of the wall at the north end is near EL 52, but descends to EL 48 at the south end of the wall. The profile along the wall is shown in Figure 3.

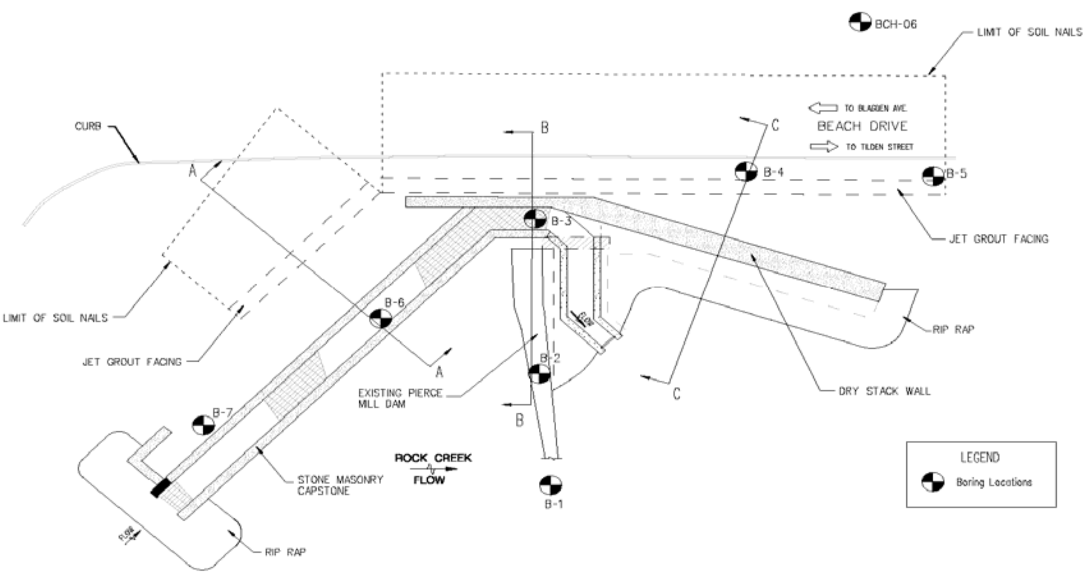

Figure 2 – Site Plan Showing Boring Locations and Conceptual Soil Nail Arrangement (KCI 2003)

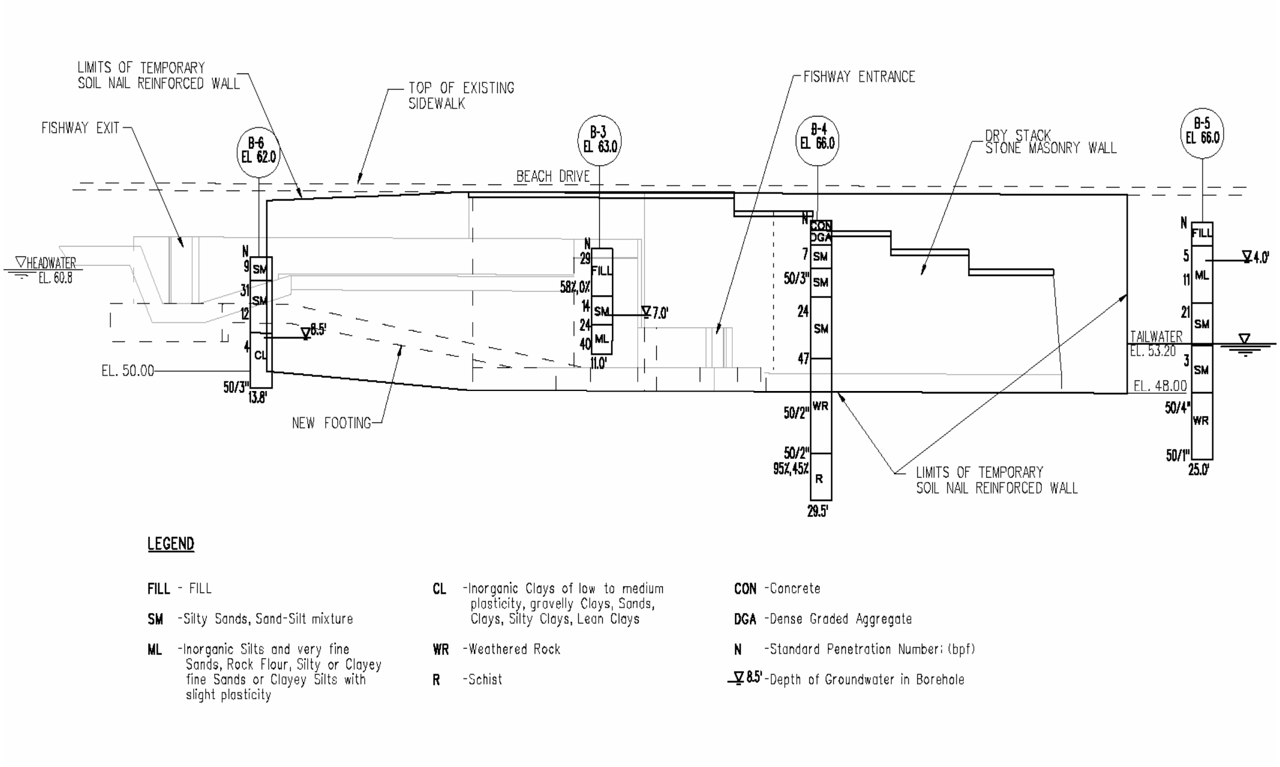

Figure 3 – Profile of Soil Nailed Wall (KCI 2003)

<u>2.0 GEOLOGY AND SUBSURFACE CONDITIONS</u>

2.1 Regional Geology

According to the *Geologic Map of Maryland*, 1968, and the *Geology and Groundwater Resources of Washington, D.C. and Vicinity, Geologic Water Supply Paper, 1776*, the project site is located in the Eastern Piedmont Physiographic Province. The natural soils of the region are residual soils that have formed in place by the weathering of the parent bedrock. Residual soils typically exhibit a profile characterized by a change from soil to decomposed rock to rock with increasing depth below the ground surface. The project site appears to be located in the Wissahickon Formation consisting of Lower Pelitic Schist. This formation is believed to be of the Late Precambrian Age.

The banks of Rock Creek have been graded several times in the past. This reach of the stream had been straightened to build Beach Drive and improve the mill race and dam site. Most recently, the elevations of Beach Drive were raised about 2 to 3-ft about 10 to 15 years ago. The old pavement was generally left in place under new fill.

<u>2.2 Subsurface Conditions</u>

The FILL material consisted of silty sand with gravel, cobbles, and boulders with occasional roots and wood fragments. It appears that the SPT N-values are exaggerated by gravel, cobbles, boulders and other obstructions and are not indicative of the relative density or consistency of the FILL material. A near continuous layer of pavement was encountered during construction, but was not encountered in the borings. The natural soils were residual soils and consisted of three strata. The first consists of very loose to medium dense silty sand; the second, medium stiff to hard sandy silt; and the third, soft to hard sandy, micaceous clay.

The residual soil gradually transitions with depth into Decomposed Rock. Decomposed Rock is defined as residual material which retains the relic rock structure of the parent bedrock and exhibits SPT N-values consistently in excess of 60 blows/foot and less than 50 blows/inch or spoon refusal.

The parent bedrock consisted of schist. The bedrock encountered was moderately fractured and slightly weathered. The RQD ranged from 0 to 60-percent. Rock was encountered at elevations ranging from EL 38.5 to EL 42 along the face of the excavation, but was at EL 57 in the north bound roadway indicating a steeply sloping rock line downward towards the creek. KCI (2003)

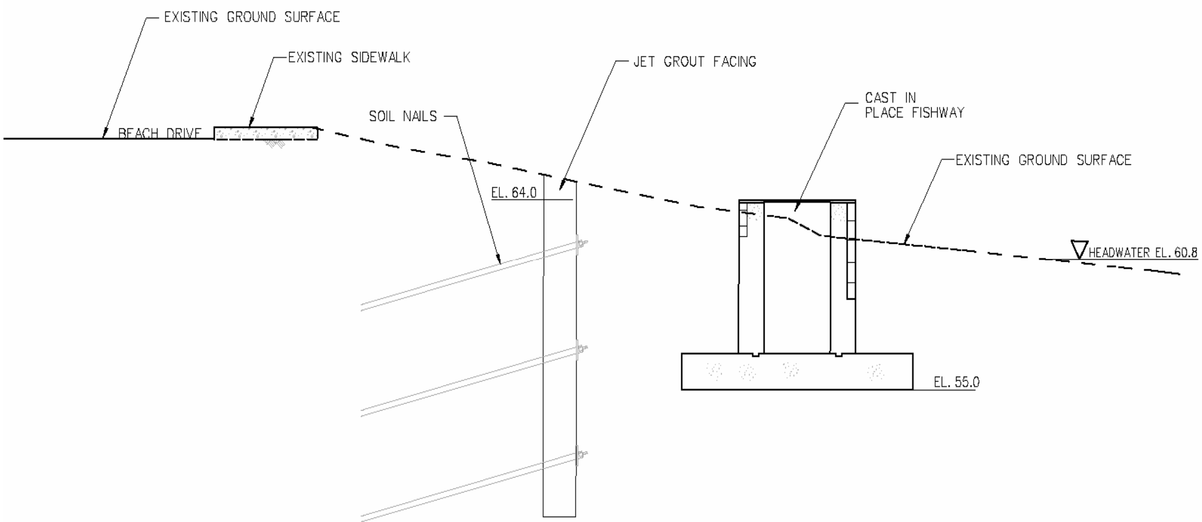

Figure 4 - Cross Section A through Soil Nailed Wall and Upstream Exit of Fishway (KCI 2003)

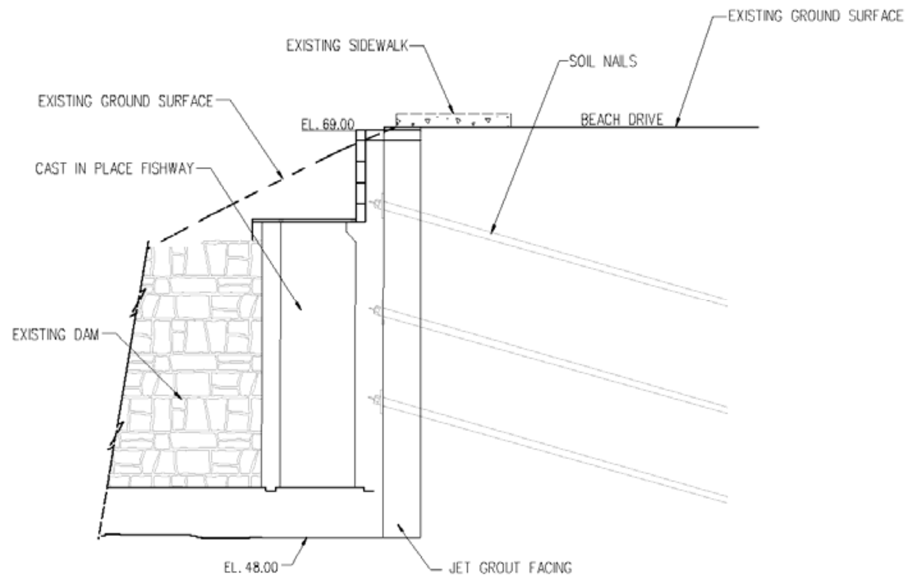

Figure 5 - Cross Section B through Soil Nailed Wall at Dam and Beach Drive (KCI 2003)

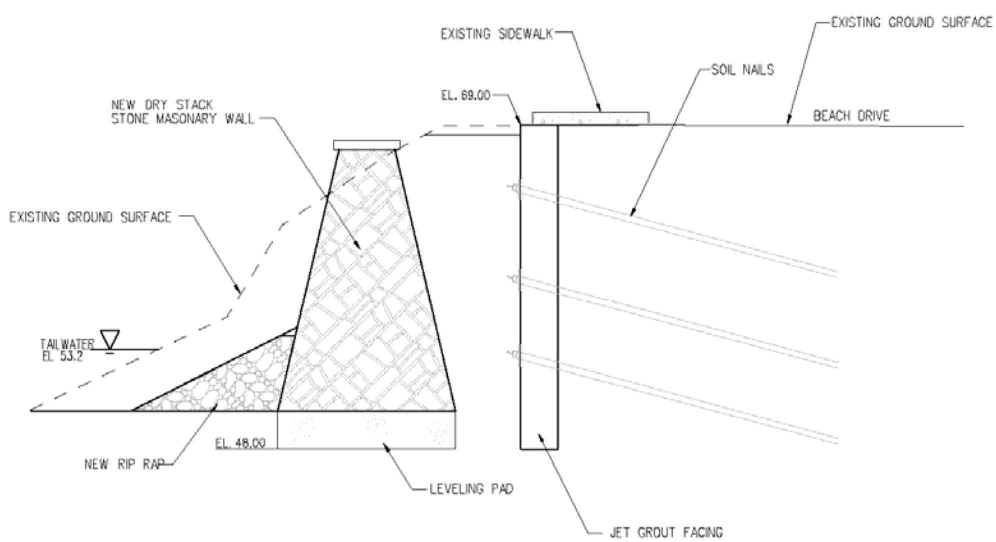

Figure 6 - Cross Section C through Soil Nailed Wall Below Downstream Entrance to Fishway (KCI 2003)

3.0 SITE RESTRICTIONS

Several project constraints were imposed by the NPS to limit the impact to the historic fabric and environment of the site so as not to impede the operations and public enjoyment of the park. The constraints affecting constructability at the site included:

- Since Beach Drive is a major commuter route to downtown DC, it was imperative that construction be limited to non-rush hour periods. Construction in the roadway was limited to the hours of 9:00 am to 3:30 pm weekdays; no holidays, weekends, or night construction was allowed. However, permission was granted to close the road from 9:00 pm to 5:00 am for the jet grouting operations from August 31 to September 8, 2005.

- Minor single lane closures could not be longer than 15 minutes.

- No large construction vehicles were allowed in the park, and those vehicles that the Contractor used had to meet certain size and weight restrictions and even then had to be limited to particular pre-approved routes through the park.

4.0 ENGINEERING CONSIDERATIONS

Because of the tight working area, the slopes could not be laid back: a temporary support of excavation was required. The following three temporary excavation support options were evaluated:

- Soldier Piles and Lagging with Tiebacks
- Secant Pile Wall with Tiebacks
- Soil Nail Reinforced Wall with Jet Grouting

The Consultant recommended that a temporary soil nail reinforced wall with jet grout facing be constructed along the proposed temporary excavation support alignment. Based on the subsurface conditions encountered, it was anticipated that groundwater difficulties and the resulting short standup time of the excavated wall face may create construction difficulties. The groundwater will likely flow through fractures in the decomposed rock and relic discontinuities in the residual soil as well as possibly along the base of the fill. Given the highly variable nature of the rock, fill, silty soil and potentially wet conditions, it was felt that the facing needed to be installed prior to excavation. The cobbles and the rock layer precluded steel sheeting from consideration and the lack of stand up time eliminated soldier piles and lagging. The jet grout probe could usually penetrate boulders, or if refusal was encountered additional columns of jet grout soil-crete could be added surrounding the obstruction. Once below the obstruction the jet grouting could then grout under the obstruction. This, in effect, would securely incorporate the boulder or obstruction into the jet grout facing avoiding a potential window under the obstruction. Even if the seal between the obstruction and the jet grout was not water tight, and water leak would be localized and not likely to cause piping or underground erosion.

The secant piles would require a rather large auger cast pile or drilled shaft rig to penetrate the cobbley fill and the tight work area would make the large pieces of equipment problematic. There was also concern that the cobbles might create alignment difficulties, and there was not enough room near the dam abutment to allow for mis-aligned piles. If a smaller sized auger cast-in-place pile rig was used, it will increase the risk of mis-aligned piles and the risk of refusal before the bottom of the excavation was reached. For these reasons, jet grouting was selected for the wall facing.

Jet grouting can be accomplished using reasonably sized equipment suitable for the confined site and it can be configured for penetrating varied subsurface materials such as soil, and rock. Cobbles and boulders could still present difficulties, but they can be addressed in the field with out too much change in procedures. The grout could act as cementing agent to stabilize the unstable material in place and incorporate boulders or obstructions into the jet grout mass. The jet grout facing could be extended to the decomposed rock stratum therefore reducing dewatering during construction and creating a firm toe in of the wall facing. Weep holes were included to avoid the build up of hydrostatic pressures behind the wall.

4.1 Jet Grouting Described

Jet grouting is a specialized in situ mixing of soil and cement grout. After reaching the required excavation depth, the drill rods are rotated and raised to the ground surface at a constant rate, while injecting cement grout at high pressure, forming a soil-cement column. When placed in an overlapping fashion, the columns can be configured to provide, selective treatment or mass-treatment areas of soils; however, the efficiency of the columns is dependent upon the section modulus of any steel reinforcement that may be inserted, and the compressive strength of the grout. Using this method to construct the wall, much smaller equipment can be used than with soldier piles. Wide flange or H-pile members, or other reinforcement may be placed while the grout is still fluid to act as soldier piles to increase the moment and bonding resistance of the wall. The jet grout wall at this site did not contain any soldier piles or reinforcing since the soil nails and walers provided adequate reinforcement.

It was anticipated that the installation of the soil nails will require that the bore holes be drilled and the nails be cased for the entire embedment length. Typically, residual soils provide adequate support to keep the borehole open; however, at this site the soils are rather sandy and near the bottom of the wall and are also wet. In addition significant amounts of fill existed. An alternative installation method may consist of using hollow stem augers. This installation method allows the augers to be slowly withdrawn as the grout is injected from the bottom of the hole to the top.

Advantages of a soil nail reinforced wall with jet grouting include the adaptability to varying site conditions. There are no restrictions regarding the bottom of wall embedment because surface movement can be controlled by adding additional nails. Higher compressive soil strengths along the face of the excavation are created from the jet grout thus providing an overall soil improvement. The construction of the jet grout wall will create a groundwater cut off wall, thus reducing dewatering during excavations. Quality assurance can be well documented, as each nail may be proof tested and the jet grout compressive strength can be verified. A soil nail reinforced wall with jet grouting is well suited to sites with difficult access because of the relatively small size and mobility of the construction equipment; noise and vibration nuisances are minimized.

A high degree of system redundancy of this wall minimizes the affect of a failure of one inclusion. Given the highly variable SPT N-values and subsurface materials, redundancy from the installation of multiple nails provides a wall that allows the contractor flexibility in nail placement location and construction. This was critical around the existing dam where there is little clearance between the dam and Beach Drive. For a single row of anchors (tiebacks) to be installed, the anchors would need to be firmly bonded into rock and a much smaller margin of error is required for both placement and construction. By constructing a wall with one or two rows of tiebacks, the grout facing will become a structural member of the wall and not just a facing element; therefore requiring structural steel reinforcement. To provide structural support for this scenario, the grout facing would also be required to be firmly seated into firm rock requiring large diameter rock coring.

With the method of construction selected, it is possible to drill through an obstruction including a jet grout wall face or to relocate or splay a soil nail as required in the field. This method of construction is relatively quiet compared to alternative construction methods discussed. The equipment required for the installation of the soil nail grout and jet grouting wall face is relatively small and can be used for both applications, therefore reducing mobilization costs. The use of jet grout along the wall face eliminates the excavation stand up time typically associated with the construction of a soil nail wall.

Disadvantages of a soil nailed reinforced wall with jet grout include the requirement of a specialized contractor. Underground easements for nails generally are required, but the NPS owns and maintains the road so that is not an issue at this site. Long-term creep of soil nails may occur in fine-grained soils. Since there were few fine-grained soils in this area and this is a temporary wall, creep should not be a significant issue. Due to the variable soils indicated on site, the size of the jet grout columns and the required soil nail length will be difficult to predict, therefore verification and proof testing of the nails are very important.

A conceptual plan of the soil nails is shown in Figure 2. Cross sections for the jet grout with soil nails are shown in Figures 4 to 6 of this paper.

A construction monitoring program consisting of settlement survey points was established to monitor the horizontal and vertical displacement of the temporary excavation, roadway surface of Beach Drive, and the existing Pierce Mill Dam. Threshold and Limiting Values of 0.5 and 1.0-inches, respectively, were established for the monitoring points.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 Jet Grout Equipment and Procedures

Drilling and the controlled extraction of the jet grout rods were accomplished using a Casagrande M9-1 drill rig with automated jet grouting lifting controls and timers. Grout for the jet grout operations was mixed on-site in a 7 cubic yard high speed mixing tank. Cement was delivered in bulk pneumatic tankers and stored on-site in a vertical cement silo capable of holding in excess of two truckloads of material. The jet pump consisted of a GeoAstra high pressure, triplex plunger type pump housed in a sound suppression container to significantly reduce ambient noise.

The single phase (grout only) jet grouting technique was utilized for this project. Based on availability, tooling consisted of a two phase jet grout system (capable of injecting both air and grout) with the air ports turned off to reduce the risk of heaving the road or blowing out the side of the existing slope. Grout consisted if a 1:1 (by weight) Portland cement and water grout mixture injected at pressures of 5,500 to 6,000 psi. To construct each overlapping column, grout was injected at approximately 50 gallons per minute as the jet rods were extracted at an approximate lifting rate of 1 foot per minute.

A total of 59 columns were installed to drilled depths ranging from 11 to 21-ft and the grouted length ranging from 2 to 21-ft. Fractured rock was grouted at the bottom of some of the columns during initial jetting operations. The grout columns were about 3-ft in diameter. A photograph of an excavated test column is shown in Figure 6. Figure 7 is a photograph of the jet grout rig installing a jet grout column.

The working area was confined to a narrow strip limited by the temporary coffer dam and Beach Drive. This is illustrated in Figures 9 through 11. The equipment could not turn, it had to drive straight in and back out. The grout batch plant and staging area was the parking lot north of the proposed fishway.

Figure 7 - Excavated Jet Grout Column Partially Removed From the Ground

Figure 8 - Jet Grout Installation from Beach Drive Showing Water Jets, Operator and Protective Mats

Figure 9 - Soil Nail Installation Showing Old Pavements, Rubble and Soil Fill

Figure 10 - Soil Nail Installation Showing Proximity of Peirce Mill Dam, Temporary Coffer Dam and Beach Drive

Figure 11 - Finished Soil Nail Wall Showing Weep Holes, the East Abutment of Peirce Mill Dam and the Formwork for the Fishway.

5.2 Soil Nail Construction and Testing

Two sacrificial verification load tests were conducted on September 19, 2005. The test soil nail installed with tremied in-place grout failed the test: the jack could not maintain pressure after 75% of the design load was applied. The test soil nail with the pressure grout complied with the special provision requirements. The structural element of both soil nails were Grade 75 all-thread bars with a free length of 82 inches and a bond length of 132 inches. The design test load was 32 kips. The design of the soils was revised and three additional tests were performed October 24, November 8, and November 14, 2005. The new soil nail had a design load of 29 kips, a free length of 96-inches and a bond length of 120 inches. The last production soil nail was installed November 15, 2005. Figure 12 is a plot of all five of the sacrificial verification load tests and the minimum theoretical displacement.

Assuming an effective diameter of 7 inches, the tremied in place soil nail test results indicated an average ultimate resistance of about 1,200 psf (8.3 psi), whereas the pressure grouted soil nail obtained at least 3,200 psf (22psi), thus achieving the minimum required load factor for pullout resistance (0.50). Subsequent verification and proof tests of several additional production anchors confirmed that the minimum safety factors assumed in the design were maintained.

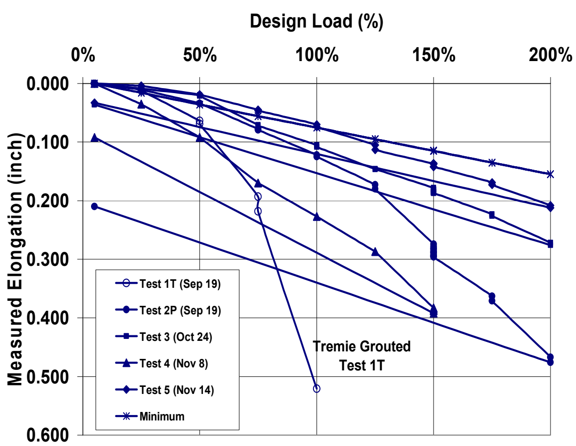

Figure 12 – Verification Load Test Results

These adhesion values compare favorably with the published case studies and recommended design values contained in FHWA (1996) and the Post-tensioning Institute (PTI) Soil Nail Manual. The adhesion for the tremied nails was very close to the value reported for nails installed in sandy silt with gravel and clay: 1.295-ksf. The pressure grouted nail was in the range reported for sandy gravel with boulders and decomposed rock. These reported values ranged from 2.861 to 4.553 ksf. FHWA (1996) did not describe in detail the installation method for the soil nails.

The PTI manual indicated a range of values for cohesion-less soils for gravity grouted anchors of 10 to 20 psi, slightly higher than the 8.3 reported for this project. For pressure grouted anchors, the PTI manual recommends between 12 and 55 psi for fine to medium, medium to dense sand. This is a wide range, but the values obtained from this testing program are with in this range.

The early tests were conducted on nails on the upper most row and the nails were therefore mostly in soil and possibly fill. The later nails were at a lower elevation and therefore mostly in decomposed schist. the difference in material is reflected in the measured elongation during the tests. The first pressure grouted tests resulted in a deflection of 0.476 inches, while the last nail tested resulted in a deflection of 0.212 inches: less than half of the first test.

An instrumentation program was initiated to monitor wall movements. If excessive wall movements were detected then remedial actions could be taken such as adding more soil nails. The elevations at the top of gutter in Beach Drive and the top of dam were monitored for elevation as the work progressed. The initial readings were taken August 18, 2005, and the final reading was obtained January 1, 2006. The top of gutter settled as much as 0.38-inches by the November 14, 2005: the last week of soil nail installation. A few days later it rebounded 0.048-inches for a total settlement of 0.34-inches during construction of the jet grouting and the soil nailed wall. By December 5, 2005 the gutter settled more for a total settlement of 0.41 inches. The final reading showed a significant rebound that was attributable to frost heave. The rebound events recorded were likely the result of thermal changes and solar heating of the concrete gutter pan as well as extreme rainfall events as described below. There were no noticeable cracks, gaps or mis-alignments noticed in the pavement, gutter pan or sidewalk during construction. Figure 13 plots the results of a monitoring point at the gutter location.

September 2005 was the driest September on record in Washington, D.C.; by October 22, 2005 October was already the wettest October on record. This extreme variation in rainfall could explain some of the unusual settlement and rebounds: they could be due to shrink/swell of the soils as the soils dried out and re-wetted.

Freezing and thawing of the soils in November, December and January could also have contributed to some of the movements.

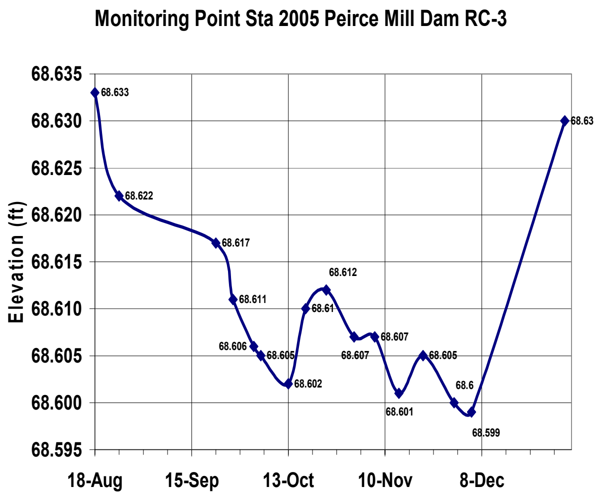

Figure 13 – Monitoring Point STA 2005

On June 25 and 26, 2006 the Washington, D.C. area experienced the two wettest days recorded: 9.41inches in 48 hours with 7.09 inches on the 25th NWS (2006). There was extensive flooding and 2000 people had to be evacuated upstream of the park for fear that an earth dam would fail. By this time the dry stack retaining wall near the fishway entrance and the backfill was complete. However, the floodwater overtopped the drystack wall and eroded the backfill, rip rap, the turf reinforcement matting and a portion of the soils supporting Beach Drive to the south of the jet grout facing. This exposed about 20 to 30 feet of the jet grout facing. The area of the jet grout facing was not eroded and the roadway was protected.

6.0 CONCLUSIONS

The Peirce Mill Dam Denil fishway project presented several unusual design and construction constraints. The high and variable groundwater and adjacent stream, the variable rock line, the old pavement and rubble fill combined with the soft and wet silty soils required that the installation methods be compatible with all of these wide ranging conditions and be adaptable to potentially abrupt changes in the actual conditions.

The weep holes shown in Figure 11 flowed continuously illustrating the need that a stable facing element to be in place prior to excavation. Also, the materials encountered contained more boulders and cobbles than expected as well as an old pavement layer. These materials would have made more conventional facing methods such as soldier piles and lagging or sheet piles difficult if not impossible to install.

The equipment needed to have a small foot print to reduce disturbance to the park environment and minimize any other impacts to the environment to an acceptable level. The above requirements combined with having to work within a highly visible and operating park, and directly adjacent to a major collector road and environmentally sensitive stream required a creative combination of techniques. For this unusual combination of conditions, the combination of jet grouting and soil nailing proved to be a successful approach.

7.0 REFERNCES

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