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ABSTRACT

American Municipal Power is developing the Smithland Hydroelectric Project adjacent to the U.S. Army Corps of Engineers' existing Smithland Locks and Dam on the Ohio River. The site geology is characterized by over 100 feet of variable alluvial deposits overlying karstic limestone. The investigation program revealed loose to medium dense sands and sandy gravels along with some interbedded clay. Large voids were encountered in rock and ground loss into solution features was believed to have created 'chimneys' of loosened soil.

A project-specific seismic hazard evaluation determined that the maximum credible earthquake (MCE) is a 7.5M event associated with the New Madrid Seismic Zone. Site response analysis indicates that a peak ground acceleration of 0.52 g at the ground surface could be expected for the MCE, largely due to amplification within the soil column. The liquefaction and cyclic softening assessment indicated that the granular soils at the site would be susceptible to liquefaction, if left untreated. There was also concern about potential ground loss below the powerhouse and the migration of soil into rock under the powerhouse's sheet pile cut-off wall.

A ground improvement program was conducted to address these issues. Vibro-replacement (stone columns) was performed under the hardfill closure structures to minimize settlement and mitigate against liquefaction. Compaction grouting was performed in overburden and rock to mitigate against liquefaction and ground loss below and adjacent to the powerhouse. Consolidation grouting was also performed along the cut-off wall.

INTRODUCTION

The Smithland Hydroelectric Project is one of four new run-of-the-river hydroelectric projects on the Ohio River being developed by American Municipal Power, Inc. (AMP). AMP is a nonprofit wholesale power supplier for municipal electric systems. AMP serves 129 members – 128 member municipal electric communities in the states of Ohio, Pennsylvania, Michigan, Virginia, Kentucky and West Virginia, as well as the Delaware Municipal Electric Corporation. Combined, these publicly owned utilities serve approximately 625,000 customers.

The project is located on the Kentucky side of the Ohio River, adjacent to the existing Smithland Locks and Dam owned and operated by the U.S. Army Corps of Engineers (USACE) (Figure 1). The project will divert water from the existing Locks and Dam through bulb turbines to generate an average gross annual output of approximately 379 GWh. The construction of the project started in September 2010, and is expected to begin generating power in 2014.

The Smithland Hydroelectric Project consists of a reinforced concrete powerhouse, a riverside closure structure, a landside closure structure, a sheet pile cut-off wall, an approach channel, and a tailrace channel (Fig. 2). The powerhouse will house three horizontal 25.3 MW bulb-type turbines and generating units with an estimated total rated capacity of 76 MW at a gross head of 22 feet. A riverside closure structure will be constructed between the powerhouse and the existing cellular fixed weir of the Smithland Locks and Dam. A landside closure structure will be constructed between the powerhouse and the left bank. Both structures will be constructed of concrete-faced hardfill and founded on soil. A new sheet pile cut-off wall will be constructed along the upstream toe of the closure structures and powerhouse. The approach channel will be excavated in the existing bank and riverbed to conduct water to the powerhouse. The tailrace channel will be excavated in the existing bank and riverbed to conduct the powerhouse discharge from the draft tube exits back into the river.

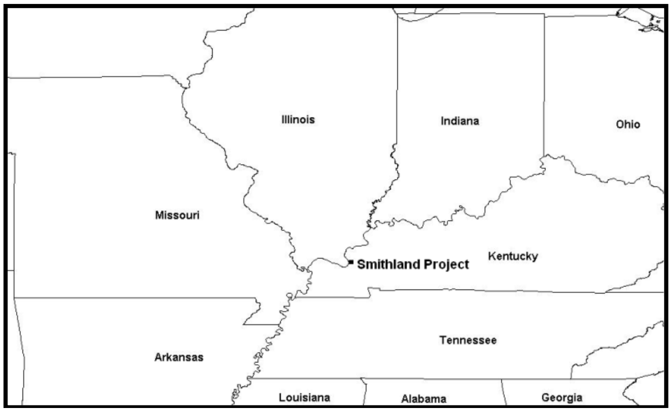

Fig.1. Location of Smithland Hydroelectric Project

Geotechnical investigation and analysis indicated that the soil foundations for the powerhouse and closure structures would likely to experience liquefaction, large earthquake-induced settlement, and post-earthquake instability under the Maximum Credible Earthquake (MCE), if left untreated. Foundation soil of the powerhouse could also possibly migrate into the voids in the bedrock resulting in differential settlement. To address these foundation issues and reduce under-seepage through rock, a ground improvement program was performed. This work included stone column installation in the closure structures foundation, compaction grouting in the powerhouse foundation, and consolidation grouting along the sheet pile cut-off wall. This paper presents technical information on this ground improvement program at the Smithland Hydroelectric Project.

SUBSURFACE CONDITIONS

The project site is located on the east side of the Ohio River near Smithland, Kentucky, on a flood plain of the locally

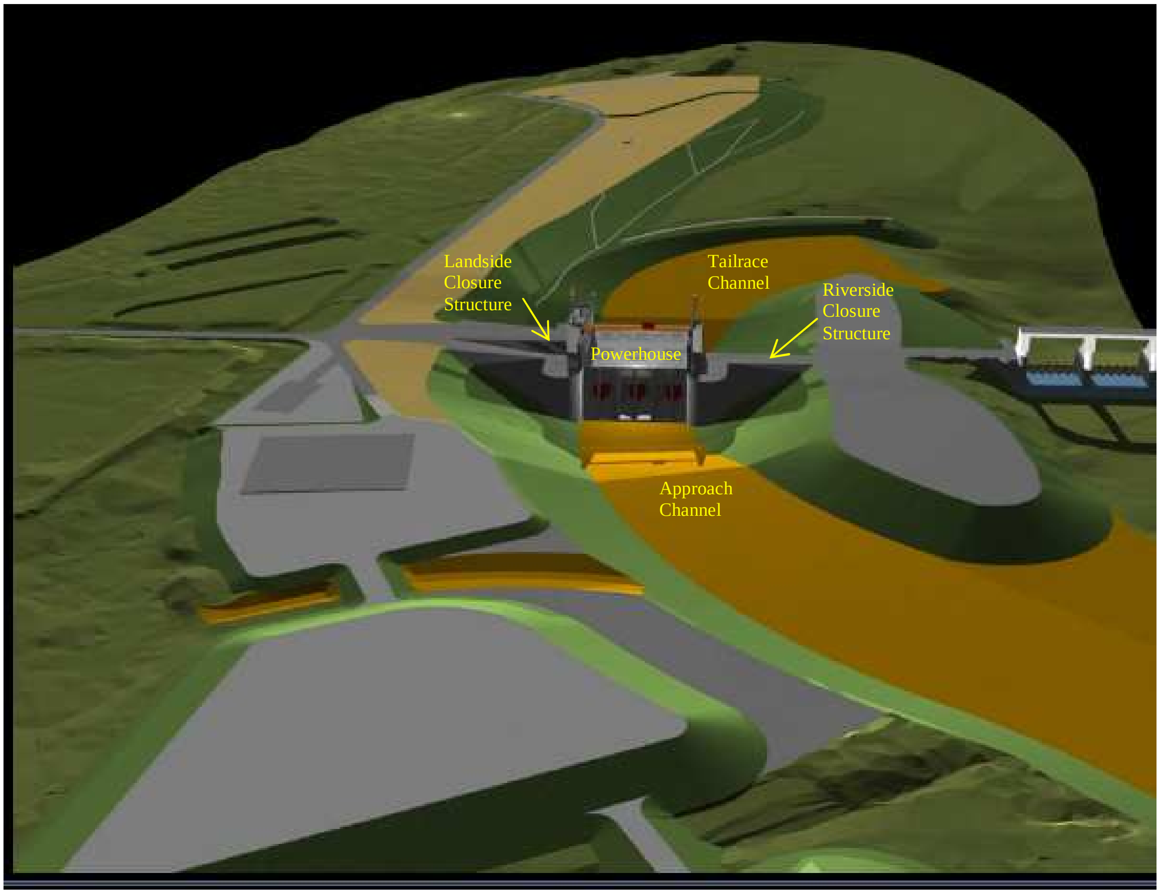

Fig.2. Project Components of Smithland Hydroelectric Project

south-flowing river. The subsurface materials consist of approximately 140 to 160 ft of alluvium overlying karstic limestone. Overburden consists of approximately 20 ft of clay that overlies loose to medium dense sand, which in turn overlies medium dense sand and gravel with occasional clay layers and lenses. Underlying the alluvial deposits is Mississippian age limestone of the Ste. Genevieve Formation. This formation consists of oolitic to cherty dolomitic limestone which occurs in generally flat-lying but slightly undulating thin to massive beds at the site with encountered voids ranging up to in excess of 10 ft deep in the upper 20 ft of rock. Many of the voids encountered by the exploration program were in-filled with soil, but large portions were either open or only partially in-filled. The subsurface materials were subdivided into the following layers (Table 1).

Table 1. Subsurface Materials and Characteristics

Layer	Approx. Elev.	Characteristics
Stiff Clay	El. 330 to 310	Lean clay with trace silt. SPT N-values ranged from 2 to 43; majority between 6 and 15. Plasticity index ranged from 5 to 31.
Loose to Medium Dense Sand	El. 310 to 260	Fine to coarse sand with trace fine gravel. SPT N-values ranged from 0 to in excess of 50; majority between 15 and 24.
Medium Dense Sand and Gravel	El. 260 to 180	Medium dense sand and gravel. SPT N-values ranged from 0 to in excess of 50; majority between 24 and 50.
Clay Seams	<El.280	Lean clay with trace silt. SPT N-values ranged from 0 to in excess of 50; majority between 8 and 19. Plasticity index ranged from 4 to 46.
Bedrock	Top of rock between El. 200 and 160	Oolitic to cherty dolomitic limestone of Mississippian age with solution features the varying soil in-filling. Average RQD was 69 with average recovery of 80%.

CHALLENGING FOUNDATION ISSUES

Seismic Hazards

The project site is close to two seismic zones: the New Madrid Seismic Zone (NMSZ) and Wabash Valley Seismic Zone (WVSZ) (Figure 3). Therefore, seismic hazards such as strong ground motion, liquefaction, earthquake-induced settlement and post-earthquake instability are challenging issues to the foundation of the powerhouse and closure structures.

A site-specific seismic hazard assessment was performed to develop design earthquakes. Probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) were performed in this study. Since the project is classified as a Significant Hazard Structure based on USACE (1995) and FERC (1991) criterion, the Maximum Design Earthquake was taken as the MCE. From the seismic hazard analysis, the MCE was established as:

- Design Earthquake Magnitude: 7.5 (Mw) at 43 km
- Peak Ground Acceleration (PGA): 0.29g (theoretical rock outcrop)

The MCE was used for the design and analysis of all water retaining structures of the permanent works to protect against potential loss of life and major economic losses that could be associated with an uncontrolled loss of the navigation pool. Under the MCE, the water retaining structures are to perform without catastrophic failure, such as uncontrolled release of the reservoir, although significant damage or economic loss may be tolerated (FEMA, 2005). For the Smithland project, the permanent water retaining structures are the powerhouse and its foundation, the closure structures and their foundations, and the soil immediately adjacent to those structures.

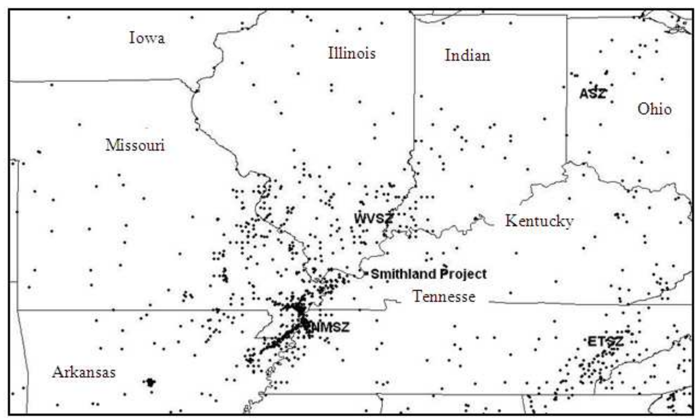

Fig.3. Seismicity of the Central and Eastern United States

Liquefaction Potential.

Liquefaction and cyclic softening analysis using the approaches developed by Idriss and Boulanger (2008) for the MCE indicated that most granular soil layers beneath the powerhouse and closure structures would likely have undergone liquefaction if left untreated and subjected to the MCE.

Post-Earthquake Instability and Dynamic Deformation.

The analyses for slope stability, earthquake-induced settlement, and deformation also indicated that ground improvement was needed for proper performance of the powerhouse and closure structures and the adjacent channel slopes in the event of the MCE or a similarly large earthquake.

Karst Foundation

Limestone recovered from rock coring exhibited features ranging from large voids to pinpoint vugular texture, dissolution enhanced fractures with crystal growth on the fracture walls to partially healed, healed and closed fractures and a range of texture from medium to coarse grain bioclastic to very fine grain or lithographic. Interpreted sizes of the larger voids typically ranged from about 1 to 6 ft with numerous voids smaller than 0.5 ft. Weathering condition of the intact recovered rock varied from fresh to moderate with severe weathering generally observed at or immediately adjacent to voids and fractures. The weathered condition, associated iron oxide staining and crystal formation observed along most open joints, along walls of voids and within vugs indicates the movement of groundwater through the rock mass.

Karstic solution features were encountered mainly in top 20 ft of bedrock and compose about 10% of the drilling in this zone. Many of the encountered voids were partially in-filled with soil of apparently open. Therefore, the design needed to address the potential for ground loss due to soil migrating into the karstic limestone bedrock under static and/or seismic conditions.

Therefore, according to the project site conditions and seismic hazard, ground improvement was needed in the vicinity of the water retaining structures to meet the general performance requirements:

- For the powerhouse foundation, ground improvement needed to minimize total and differential settlements, maintain earthquake and post-earthquake stability (primarily liquefaction), reduce earthquake-induced settlement and deformation, and prevent loss of ground into karstic features under static and seismic conditions.
- For the closure structure foundations, ground improvement needed to minimize total and differential settlements, maintain earthquake and post-earthquake stability, and reduce earthquake-induced settlement and deformation.
- For seepage cut-off features, ground improvement needed to reduce seepage and control exit gradients, prevent piping in soil and rock, and provide a durable cut-off. Of particular concern is the potential presence of erodible materials in solution channels within the bedrock.

GROUND IMPROVEMENT DESIGN

Stone columns were selected to mitigate liquefaction under the foundation soil of the closure structures, and compaction grouting was selected to mitigate liquefaction of the overburden under the powerhouse and fill karstic features in the bedrock; consolidation grouting would also be performed along the cut-off wall. Fig. 4 presents the ground improvement areas at the powerhouse and closure structures and surrounding areas (about 70 ft beyond the limits of the powerhouse and the closure structures). Fig. 5 shows the cross section of the ground improvement through the landside closure structures, powerhouse and riverside closure structure. The detailed information on selection of ground improvement techniques and design criteria is presented in the following section.

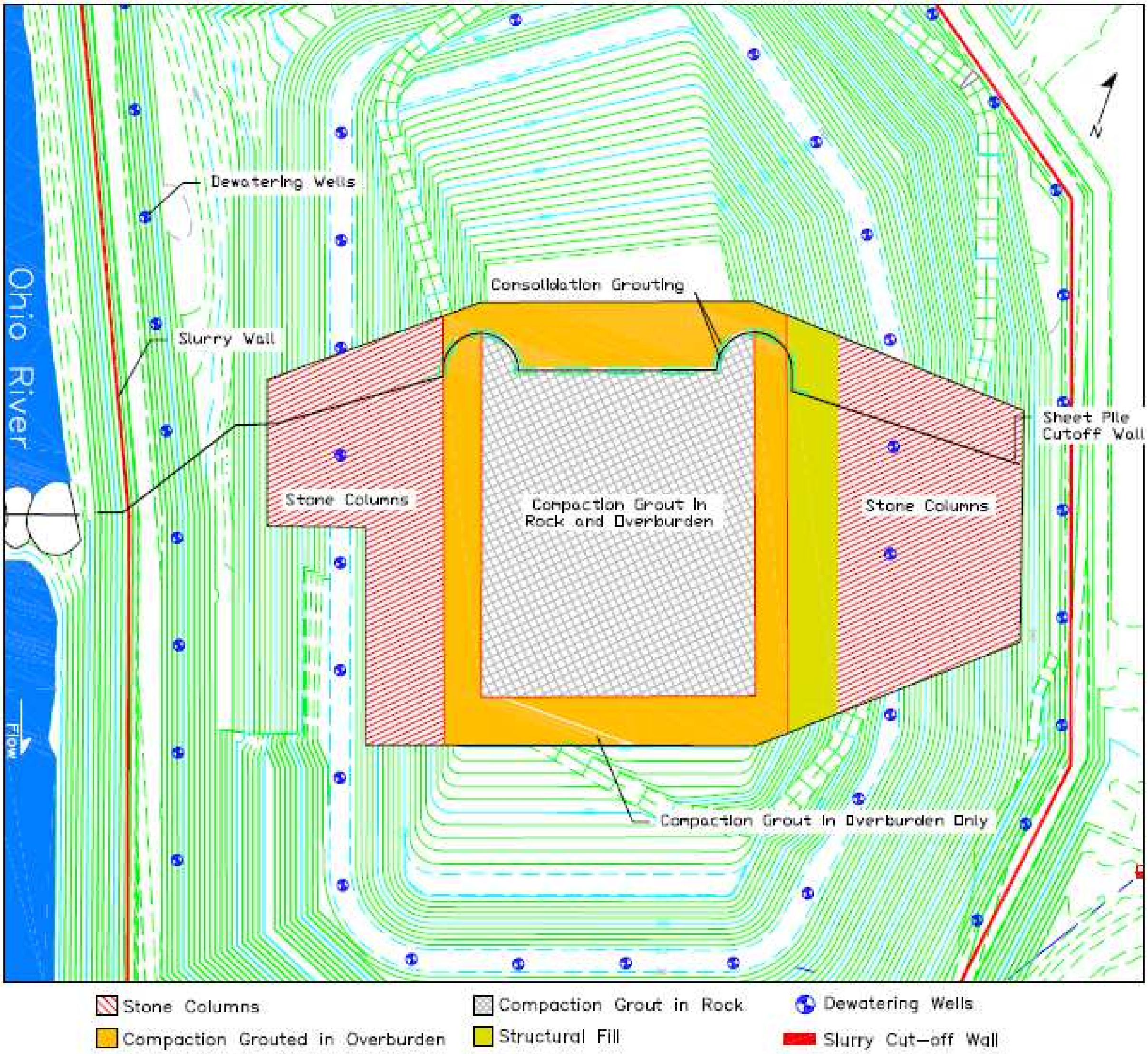

Fig 4. Plan View of Limits of Ground Improvement

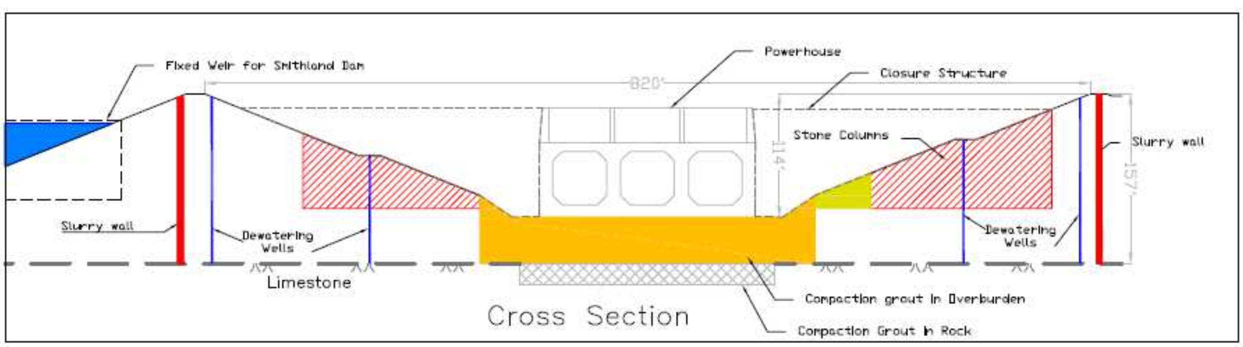

Fig 5. Cross Section of Ground Improvement

Closure Structure Foundation Improvement

The available techniques for liquefaction mitigation include deep dynamic compaction, deep soil mixing, jet grouting, compaction grouting, and vibro compaction. Upon a review of the above techniques, vibro replacement bottom-feed method was chosen to perform ground improvement and install stone columns for the foundation of the closure structures due to its relatively lower cost and capacity to treat soil at depth and obtain near-surface densification (ASCE, 1995; Kirsch and Kirsch, 2010; Barksdalel and Bachus, 1983; Moseley and Kirsch, 2004; and Xanthakos et al. 1994). In addition providing a means of densification, the stone columns were also selected to form dense elements that provide addition bearing and lateral reinforcement.

The target maximum depth of treatment is 60 ft, which is in the range for this technique. During the operation of vibro replacement bottom-feed method, a pre-drill rig was generally used to loosen soils in a pilot "hole" which enables the vibrator to penetrate to depth without excessive water jetting.

During the design phase, the most probable spacing of vibro probes was estimated to range from 8 to 9 ft, a minimum stone column diameter of 3.0 ft was specified, and an equilateral triangular installation pattern was preliminarily selected.

A verification testing program was proposed to determine if the required densification was achieved. Acceptance criteria were developed to evaluate CPT soundings and SPT blow counts of ground treated by stone columns based on the assessed ground motions for the MCE and the liquefaction analysis approaches presented by Idriss and Boulanger (2008).

Powerhouse Foundation Improvement

Compaction grouting was selected to treat overburden and bedrock under the powerhouse. Since this work could be performed from an elevation approximately 30 ft above the general powerhouse foundation level, it was believed that sufficient confinement would be available to achieve the required densification of relatively loose zones. This technique also provided means of filling voids in rock to lessen the potential for soil to migrate into to solution features.

Verification testing using SPT was used to assess that the required densification was achieved. Similar to the criteria used to evaluate the improvement achieved by the stone columns, the criterion for compaction grouting was developed based on the liquefaction analysis approaches presented by Idriss and Boulanger (2008).

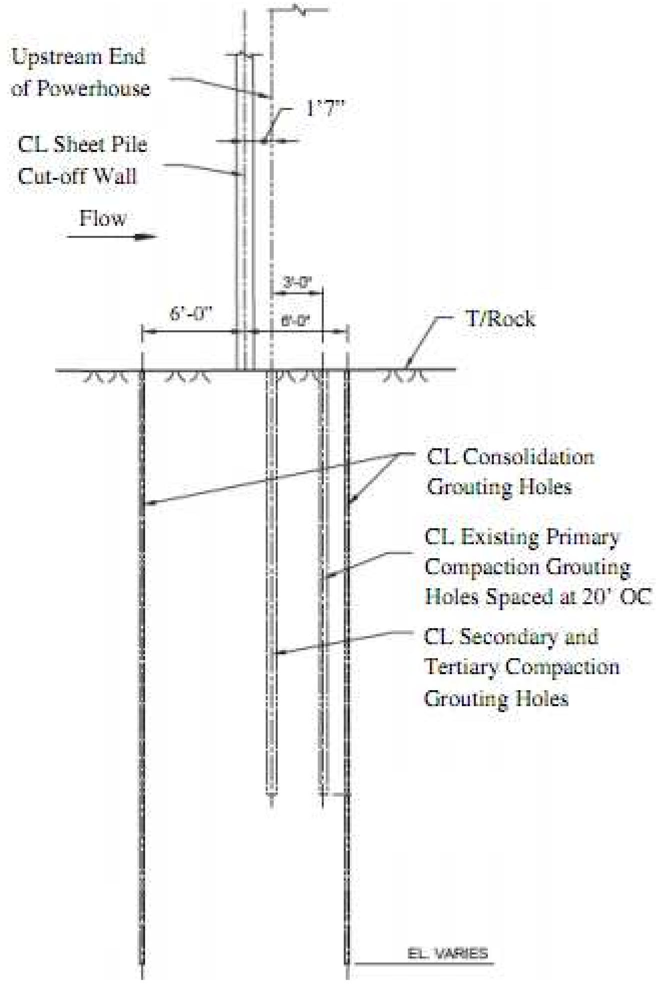

Fig. 6. Cross section of Consolidation Grouting Along Cut-off Wall

Grouting along Cut-off Wall

Following the completion of compaction grouting, consolidation grouting was performed along the sheet pile cut-off wall along the upstream face of the powerhouse and adjacent closure structures where the sheet piling extends to the top of rock. Consolidation grout in rock was designed to fill open voids and fissures in the bedrock and reduce the overall amount of under-seepage through rock.

GROUND IMPROVEMENT CONSTRUCTION

Stone Column Installation At Closure Structure Foundations

Stone columns were installed to densify and reinforce the in-situ soils to mitigate against potential earthquake-induced settlement and potential instability of the landside and riverside closure structures. The areas improved by stone columns include the footprint of the landside and riverside closure structures and portions of the adjoining slopes.

Test Section of Stone Column Installation.

Before stone column production, sections were undertaken in the landside and riverside closure structure areas to determine the column-to-column spacing for production work. The landside test section included 37 stone columns installed with center-to-center spacing of 8 and 9 ft. A total of 8 CPT soundings (4 pre-treatment and 4 post-treatment) and 6 SPT borings (2 pre-treatment and 4 post-treatment) were completed in this test section. The riverside test section also included 37 stone columns with 8 and 9 ft center-to-center spacing and 9 CPT soundings and 6 SPT borings. Based on the stone column test program results, a 9 ft center-to-center spacing was established for the landside production stone columns and an 8 ft spacing was established for the riverside production stone columns.

Stone Column Production.

Based on the established spacing, a total of 653 production stone columns were installed in the landside stone column treatment area and 556 production stone columns were installed in the riverside treatment area including the stone columns installed in test section.

Verification CPT and SPT were conducted and evaluated against the stone column acceptance criteria. A total of 19 CPT soundings and 28 SPT borings were completed on the landside. 19 CPT soundings and 13 SPT boring were completed on the riverside.

The post-treatment test results indicated that some areas within the footprint of the landside closure structure experienced only minimal improvement using a 9 ft triangular center-to-center spacing, such that areas within 20 to 30 ft of the excavation surface did not meet the acceptance criteria. To improve post-earthquake stability and minimize possible earthquake-induced settlement, additional stone columns were installed to extend the depth at which the acceptance criteria had previously been achieved. The additional stone columns doubled the number of stone columns within this zone.

The effectiveness of ground improvement was evaluated based on liquefaction potential and earthquake-induced settlement, and Newmark permanent displacement assessed based on post-treatment CPT and SPT results. The results of these analyses indicate that post-improvement closure structure foundations would be stable under the MCE and that the stone column installation program has achieved the design intent.

The ground improvement as implemented satisfies the design intent and is considered to provide suitable foundations for the landside and riverside closure structures.

Compaction Grouting In Powerhouse Foundation

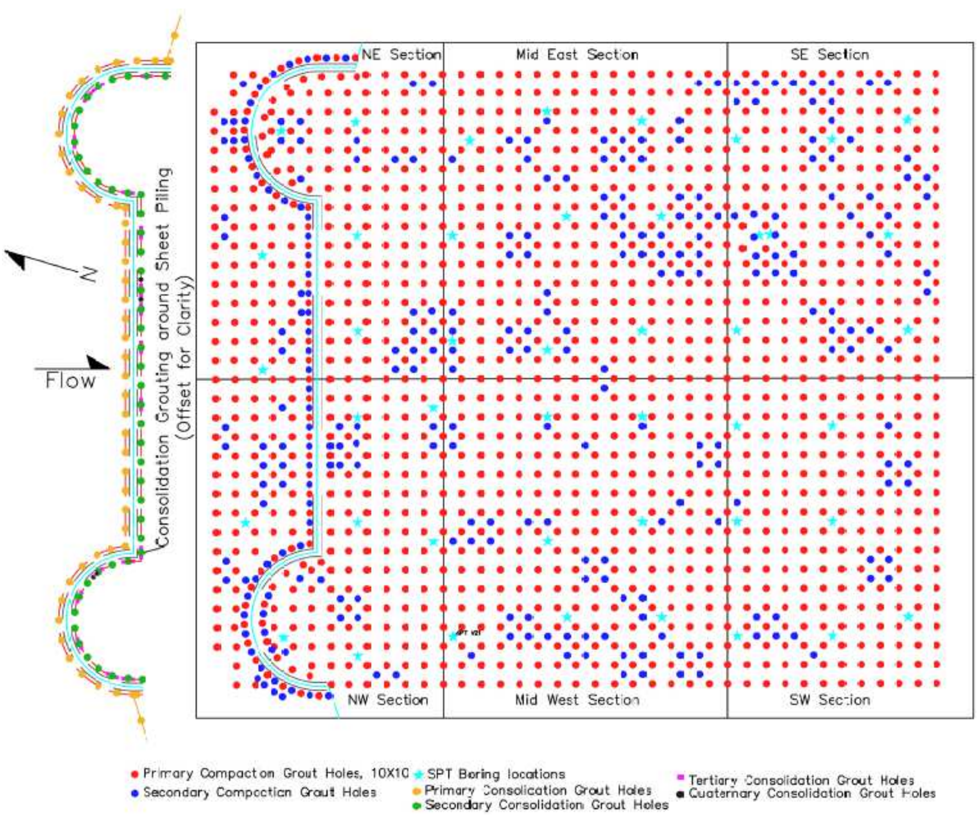

Fig. 7. Layout of Compaction Grouting Holes in the Footprint of Powerhouse and Cut-Off Wall

Compaction grouting has been performed in areas within and immediately adjacent to the powerhouse footprint to (1) fill open voids in the karstic limestone bedrock and densify loose material within solution features in the rock to mitigate against future migration of overburden soils into voids and (2) densify granular soils above the top of rock to mitigate against potential earthquake-induced settlement and strength loss of granular foundation soils. The layout of compaction grouting holes is presented in Fig. 7.

A compaction grouting test program was conducted to establish the spacing between primary grout holes and grouting procedures, including target (maximum) pressures, grout take limits and split-spacing criteria. Compaction grouting in rock generally used grout with an 8 to 10 in. slump. However, the grout with slumps as small as 2 in was used if large voids were encountered. Compaction grouting in overburden generally used grout with a slump not exceeding 3 in.

Based on the results of the test program the following grouting procedures and split spacing criteria were adopted for the production work:

- All drilling and grouting was performed from a work platform at El. 258.
- A 10 ft × 10 ft center-to-center rectangular grid was used to layout all primary grout holes.
- Compaction grouting in overburden was performed from the top of rock to El. 228, the general foundation level of the powerhouse.
- All primary holes within the limits of rock grouting were drilled and grouted to a depth of 25 ft below the apparent top of rock (the depth in which large solutions features were encountered by the investigation borings).
- In grout holes where drill tool rotation was arduous (high rotation hydraulic pressures) during grouting in rock or overburden, a lower target pressure was used to avoid seizing up the casing.
- All primary and secondary holes were grouted using the target pressures and grout take limits presented in Table 2.

Table 2 – Target Pressures and Grout Take Threshold

Matl.	Zone	Target Pressure	Max. Grout Take per 2 ft Stage	Other Criteria
Rock	Top 25 ft of Rock	450 psi	405 ft^3	If volumes exceed 405 ft^3 per 2 ft stage then pump 13.5 ft^3 per stage for remaining rock profile. Maximum total volume is 540 ft^3 per hole.
Over-burden	Top of Rock to El. 208	600 psi	30 ft^3	The 30 ft^3/stage cutoff is not applicable in the first 2 ft stage above apparent top of bedrock.
	El. 208 to 228	400 psi	6 ft^3	---
	El. 228 to Surface	---	---	Backfill / non-pressure grouting only.

- Split spaced holes were grouted with the same criteria as used for primary holes (listed above). Secondary grout holes were drilled through overburden (to the top of rock) if (1) one or more primary holes in a quadrant of four adjoining holes had an average take over the interval from 2 ft above the top of rock to El. 208 that exceeded 5 ft^3/ft or (2) two or more holes in a quadrant of four adjoining holes, had average takes over the interval from 2 ft above the top of rock to El. 208 that exceeded 2.5 ft^3/ft.
- Secondary holes were taken 25 ft into rock if (1) one or more holes in a quadrant of four adjoining holes had a total take in rock exceeding 500 ft^3 (18 yd^3) per hole or (2) two or more holes in a quadrant of four adjoining holes, had total takes in rock exceeding 250 ft^3 (9 yd^3) per hole.
- Where secondary holes were required, they were drilled at the centroid of the square grid, so as to split space the primary holes.

The post-treatment verification borings were conducted at center points between quadrants of four adjoining grout holes and, thus, represent areas receiving relatively reduced compactive effort. Fig. 8 and Fig. 9 present pre-treatment and post-treatment SPT plots, respectively. A comparison of the above two figures indicated that significant ground improvement have been achieved after compaction grouting in the powerhouse foundation. The results of the verification SPT borings (See Fig. 9) indicate that the evaluation criterion was generally exceeded at all depths except in areas where clay layers were encountered, generally between El. 220 and El. 230, especially in portions of the south east and mid-east sections.

In these areas, the clay layer will be removed by over-excavation if they are found to be soft. In a few of the SPT borings the apparent top of rock was below the median top of rock elevation, SPT blow counts were less than the evaluation criteria in the area above the top of rock. Secondary compaction grout holes were drilled at the location of these SPT borings to provide additional densification in these areas.

Overall, the post-treatment SPT borings show that the compaction grouting program significantly densified the powerhouse foundation and achieved the design intent.

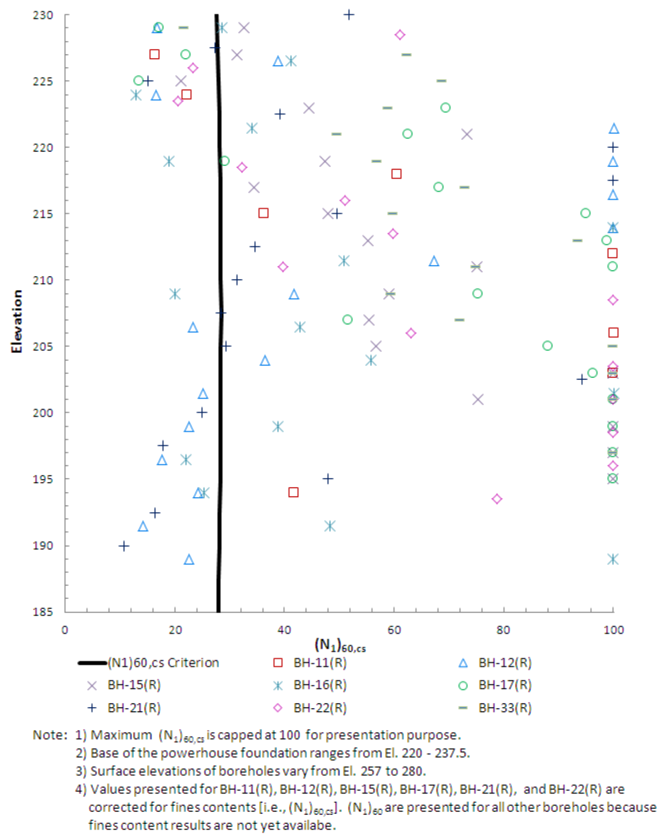

Fig. 8. Pre-Treatment Results for Powerhouse Foundation

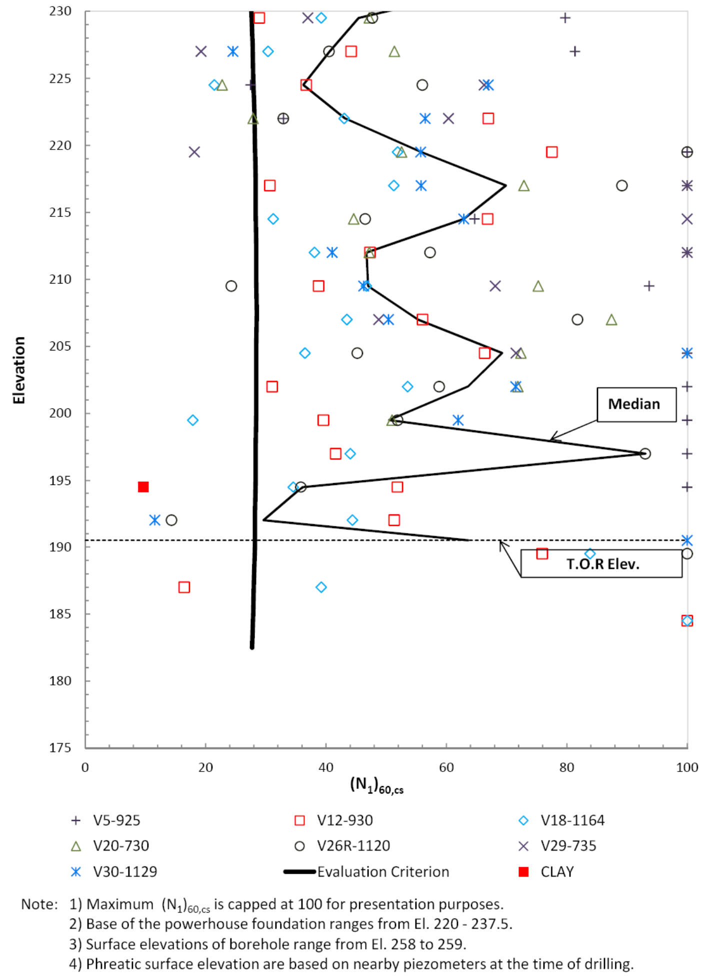

Fig. 9. Post-Treatment Results for Southwest Section in Powerhouse Foundation

Consolidation Grouting Along Cut-Off Wall

Consolidation grouting was performed in areas adjacent to the sheet pile cut-off wall along the upstream face of the powerhouse and adjacent areas of the closure structures where the sheet piling extends to the top of rock. Consolidation grouting was performed to fill open voids and fissures in the bedrock and minimize the overall amount of under-seepage through rock. It should be noted that consolidayion grouting was performed within areas that had already been treated with compaction grouting.

Primary consolidation grout holes and mandatory secondary consolidation grout holes were located upstream and downstream of the sheet pile wall, respectively. Primary and secondary holes were split spaced by tertiary holes and in turn quaternary holes in areas where the grout take in the rock measured exceeded 1.0 ft^3/ft in the proceeding hole(s). The spacing between the mandatory primary and secondary grout holes, as measured along the cut-off wall, was typically 5 ft. Where tertiary and quaternary holes were used, the spacing between grout holes was typically 2.5 ft and 1.3 ft, respectively.

Three sonic cores were drilled at locations along the periphery of the upstream sheet pile cut-off wall to obtain a continuous sample of soil and rock after completion of consolidation grouting. The recovery in the limestone bedrock was nearly 100% and voids in bedrock had been filled with grout (Fig. 9). Sonic coring indicates that compaction and consolidation grout successfully filled open voids and fissures in the bedrock.

Fig.9 Grout-Filled Seams in Core Sample

CONCLUSIONS

A ground improvement program has been completed at the Smithland Hydroelectric Project, one of several new hydroelectric projects being developed on the Ohio River at existing Corps of Engineer Locks and Dams. The ground improvement program consisted of stone column installation at the closure structures, compaction grouting at the powerhouse foundation, and consolidation grouting along the cut-off sheet pile wall. Without ground improvement, the foundation of the powerhouse and closure structures could potentially have experienced liquefaction, large earthquake-induced settlement and deformation, and post-earthquake instability as a result of the design earthquake (MCE) and powerhouse foundation soils could have potentially migrated into open voids in the bedrock resulting in differential settlement..

The post-treatment verification test results indicate that some areas within the footprint of the landside closure structure experienced only minimal improvement after the installation of stone column on a 9 ft triangular center-to-center spacing, such that areas within 20 to 30 ft of the excavation surface did not meet the established SPT acceptance criteria. To improve post-earthquake stability and minimize possible earthquake-induced settlement, additional stone columns were required. The additional stone columns doubled the number of stone columns within this zone.

Liquefaction potential and earthquake-induced settlement and Newmark permanent displacement assessments were undertaken based on post-treatment test results. These analyses indicated that improvement closure structure foundations would be stable under the MCE and that the stone column installation program achieved the design intent.

Compaction grouting was performed in areas within and immediately adjacent to the powerhouse footprint to (1) fill open voids in the karstic limestone bedrock and densify loose material within solution features in the rock to mitigate against the potential future migration of overburden soils into voids and (2) densify granular soils above the top of rock to mitigate against potential earthquake-induced settlement and strength loss of granular foundation soils. The post-treatment SPT borings show that the compaction grouting program significantly densified the powerhouse foundation and achieved the design intent.

Consolidation grouting was performed along the sheet pile cut-off wall where it extends to the top of rock. Consolidation grouting was performed to fill open voids and fissures in the bedrock and reduce the overall amount of under-seepage through rock. Sonic drilling performed after consolidation grouting indicates that the grouting program was successful in filling this features.

Based on post-treatment verification test results and re-evaluation of post-earthquake stability and deformation, the ground improvement program has successfully addressed the foundation issues encountered at the project site and will provide suitable foundation conditions for the powerhouse and closure structures.

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