

## Ground Improvements for Seattle's U230 U-Link Tunnel Construction

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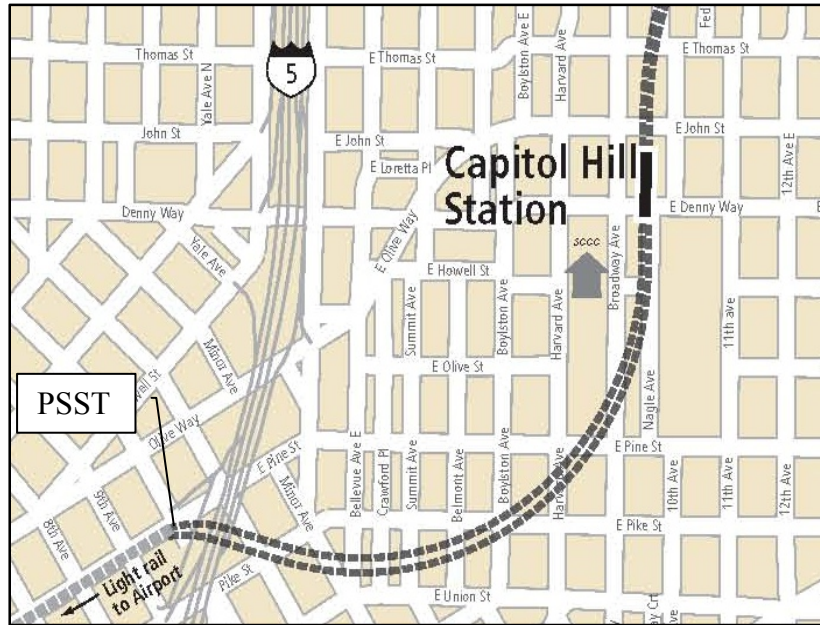
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**ABSTRACT:** Construction on Seattle's Sound Transit University Link U230 contract required ground improvement of existing site soils to assist with tunneling operations at multiple locations. The ground improvement program included installation of jet grouted blocks at the tunnel boring machine (TBM) launch and reception portals. The complex geology and congested underground utilities of Seattle's Capitol Hill posed the project's greatest challenges. City street right-of-way constraints were also a significant issue to overcome. This paper presents the rationale for the ground improvement design and current capabilities of large diameter jet grouting for ground improvement in the glacial soils of the Pacific Northwest. Moreover, this paper also discusses challenges that arose during construction and the results of the quality control and quality assurance testing of the ground-improved areas.

### INTRODUCTION

Sound Transit (*Puget Sound Regional Transit Authority*) has contracted to build a series of tunnels under multiple contracts to extend existing light rail service from the downtown core of Seattle to the University of Washington. The U230 Contract, which is the focus of this paper, will include construction of twin tunnels from the Capitol Hill Station (CHS) to the existing rail within the downtown transit tunnel on Pine Street (Figure 1). The U230 Contract includes construction of a 168 meter [550 ft] long station on Capitol Hill, which interfaces with the U220 Contract at the north end, along East John Street, between Broadway and 10<sup>th</sup> Avenue. A TBM retrieval shaft at the existing Pine Street Station and Stub Tunnel (PSST) is also required. The U220 Contract will see construction of twin tunnels between the University of Washington and Capitol Hill.

Tunnels enter and exit at both ends of the CHS requiring construction of two large ground improvement zones to assist with tunnel launch and retrieval. Similarly, two smaller ground improvement zones were also needed to receive the TBMs at the



**FIG. 1. Alignment of the U-Link Tunnels from CHS to PSST.**

PSST. The required ground improvements were achieved by stabilization of the soil mass by constructing overlapping large diameter jet grout columns to provide a solid mass of strengthened impermeable ground.

The following sections outline the need for ground improvements, initial design considerations, construction challenges, quality control measures, verification testing and in-situ conditions post construction.

## **SUBSURFACE & SITE CONDITIONS**

### **Capitol Hill Station**

The subsurface conditions at the north end of the CHS consist of: glacial till (geologic unit: Qvt) and diamict (Qvd) from the ground surface elevation 100 meters [327 feet] to elevation 91 meters [300 feet], non-glacial fluvial deposits (Qpnf) between elevation 91 meters [300 feet] to 80 meters [260 feet], and underlying layers of non-glacial and glacial lacustrine deposits (Qpnl and Qpgl). The fluvial deposits are generally described as slightly silty, gravelly sands to sandy gravels and will flow when wet, even under low hydrostatic heads. The fluvial deposits are below the static groundwater table and a dewatering system is necessary to allow the excavation to progress. This dewatering system consists of deep wells around the perimeter of the box and vacuum well-points in the northwest corner of the excavation, where the largest ground water inflows are expected. The TBMs will break into the CHS between approximately elevation 80 meters [263 feet] to 86 meters [283 feet].

The subsurface conditions at the south end of the CHS box varied considerably compared to those at the north in that the layer of fluvial deposits was much thinner. At the south end of the CHS box the soils consisted of: glacial till and diamict from the ground surface elevation 99 meters [325 feet] to elevation 82 meters [270 feet], fluvial deposits from elevation 82 meters [270 feet] to 80.5 meters [265 feet], and

underlying layers of glacial lacustrine deposits. Despite the relatively thin layer of fluvial deposits, ground-improvement was still required and the dimensions of the improved block were identical to those used at the north. The primary reason for ground improvement in this zone was uncertainty about ground conditions due to the highly variable nature of the given geology.

### **Pine Street Stub Tunnel**

The subsurface conditions at the Pine Street Stub Tunnel (PSST) consist of: artificial fill (Af) from the ground surface elevation 53.5 meters [176 feet] to elevation 42.5 meters [140 feet], landslide (Qls) and wetland deposits (Qw) between elevations 42.5 meters [140 feet] to 35 meters [115 feet], and underlying layers of non-glacial lacustrine deposits (Qpnl) and Pre-Vashon diamict (Qpgd). The U230 tunnels will break into the PSST excavation between elevations 37.5 meters [124 feet] to 31.5 meters [104 feet] and the tunnel eyes should be within the landslide and glacial lacustrine deposits. These deposits have relatively high fines content and are anticipated to be firm and exhibit slow raveling when unsupported. While the static groundwater table was estimated to be at elevation 31.5 meters [104 feet] ground improvement was still required at both break-in locations at the PSST.

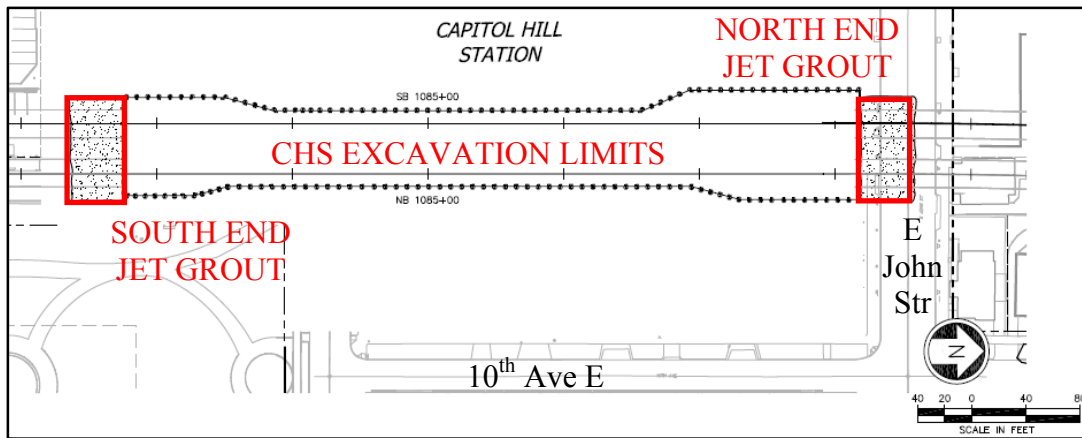
## **DESIGN CONSIDERATIONS & RATIONALE**

The U-Link tunnels will be mined with earth-pressure balance (EPB) tunnel boring machines. While EPB TBMs can provide continuous support to the native soils along the majority of the tunnel alignments, it is more difficult to maintain face support at the break-in and break-out locations at the CHS and PSST sites, hence the need for localized ground improvements.

The primary geotechnical concern at the break-in and break-out locations is that material may flow around the TBMs and into the excavated box as mining progresses. This uncontrolled loss of material can result in surface or sub-surface voids manifesting into settlements. These concerns are partially addressed by dewatering the soils within the tunnel zones but layers of saturated soil or perched water may still exist. As such, additional safety is ensured by providing ground improvement at each break-in/out location with the primary goal of increasing the strength of the native soils and reducing their permeability.

### **Capitol Hill Station**

The design considerations of the two jet grout blocks at the CHS were relatively similar. The dimensions of the blocks were calculated based on the anticipated strength of the jet-grout improved soils, the size and length of the TBMs and the pillar width between the twin-tunnels. The blocks were 15 meters [50 feet] high, 24 meters [80 feet] long and 12 meters [40 feet] wide (normal to the tunnel alignment). The dimensions of the blocks provided approximately 6 meters [20 feet] of improved soils above the tunnels and 3 meters [10 feet] below and to the sides. The width was based on assumed TBM lengths of 9 meters [30 feet] and the desire to have at least 3 meters [10 feet] of the final tunnel lining sealed and securely grouted within improved soils before the head of the TBM breached the safety of the improved zone. A secondary



**FIG 2. Locations of Jet Grout Zones for the CHS.**

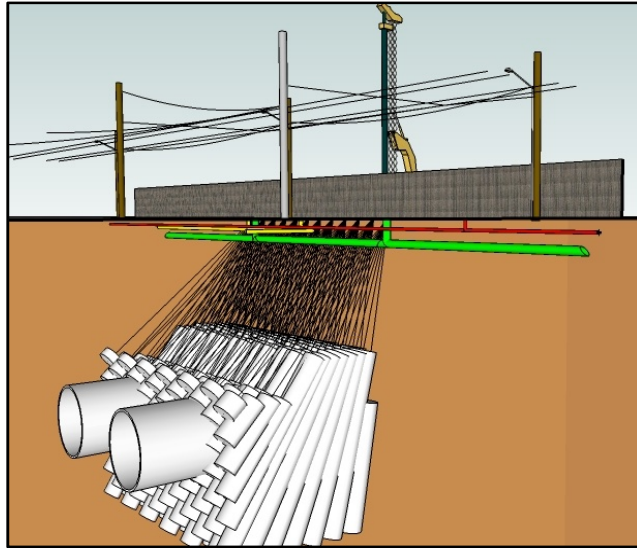
goal of the ground improvement at the CHS was to improve the performance of the excavation support systems to limit shoring displacements. Refer to Figure 2 for the locations of the jet grout zones at the CHS site.

While the design considerations for the two CHS jet grout blocks were similar, the two zones posed significantly different levels of risk. The south zone is within the confines of the U230 CHS construction site and is not adjacent to any major buildings or streets. The north block is constructed beneath the public right-of-way in E John Street and is within 7.5 meters [25 feet] of a large apartment building. The north zone poses more geotechnical challenges and is also the interface between the U220 and U230 contracts. As such, the challenges associated with the tunnel break-in at the north end of the station were considered to be much greater than at the south.

The construction of the north jet grout block was also complicated by the presence of buried and overhead utilities, and high traffic volumes on E John Street. Although the jet grouting operation could encroach within the sidewalk of E John Street, traffic lanes had to remain clear and active. A 3D model was subsequently created to determine the potential locations and angles of the jet grout columns that would be necessary to avoid the utilities, stay within the project site, and achieve the desired level of improvement, as shown in Figure 3.

### **Pine Street Stub Tunnel**

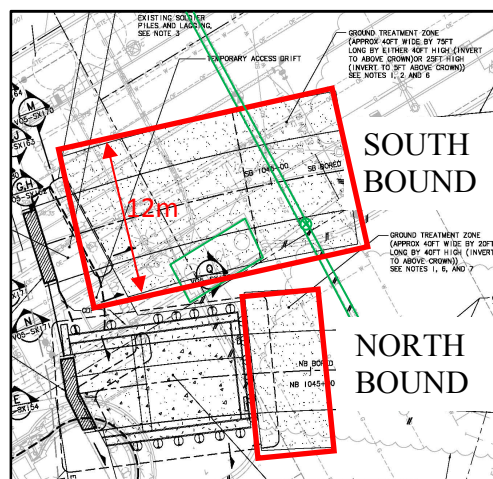
At the PSST, the dimensions of the two jet grout blocks are significantly different due to the buried utilities and anticipated construction operations. For the northbound tunnel, the jet grout zone dimensions were 12 meters [40 feet] wide by 6 meters [20 feet] long by 12 meters [40 feet] high and was designed to provide a secure face to allow the steel soldier piles of the TBM retrieval shaft to be removed after the TBM is received. For the southbound tunnel, the jet grout zone was sized 12 meters [40 feet] wide by 23 meters [75 feet] long by 12 meters [40 feet] high with a step to clear an existing deep combined sewer utility. This block not only provided a secure face to allow the steel soldier piles to be removed for the TBM alignment, but also to provide a stable tunnel heading to allow removal of the existing tiebacks from within the face of the TBM, which could potentially run in open face mode. The southbound tunnel



**FIG. 3. Preliminary Design Model of Possible Locations and Angles of Jet Grout Columns at the North Side of the CHS.**

jet grout geometry was also controlled by the existing deep sewer line that ran perpendicular to the alignment. During the design phase this utility was to remain operational during the production of the ground treatment; however, this restriction was able to be lifted during the construction phase. Refer to Figure 4 for a diagram of the northbound and southbound jet grout blocks.

During the design and tendering phases, jet grouting operations were not envisioned to be permitted to encroach onto Pine Street or on the city sidewalk. Using 3D modeling, typical locations and angles for the jet grout columns were determined, as shown in Figure 5. However, these restrictions were able to be lifted during the construction phase and one lane and the sidewalk were taken out of public service. Being able to encroach in the roadway enabled the jet grout operation to limit the amount of angled jet grout columns, which resulted in a more efficient installation.



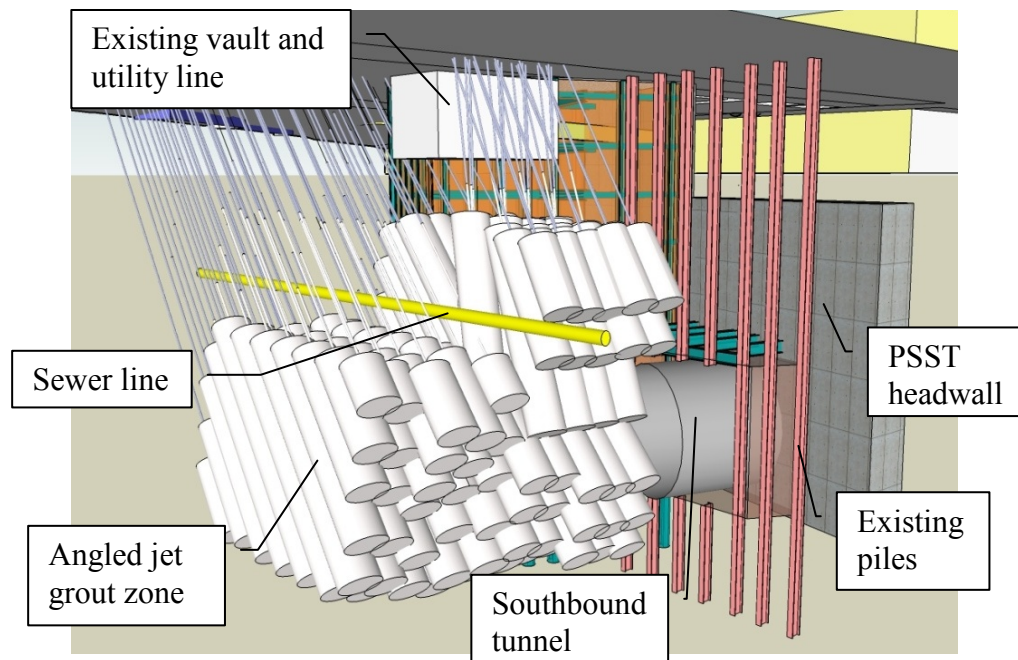
**FIG. 4. Locations of Jet Grout Zones at the PSST.**

## JET GROUTING

### Construction of Jet Grout Zones

Jet grouting started in March of 2010 at the south end of the CHS site and in late April at the north end. Work on the PSST began in September 2010 with all work being completed before year's end. At the onset of operations at each site consideration was given to the unique geotechnical conditions and the specific site constraints. The proprietary MEGA-Jet technology was used which utilizes a unique array of jetting nozzles in a modified triple fluid configuration. While normal jet grouting processes fall victim to severe fluid head losses, the MEGA-Jet system utilizes a highly refined balance of pressure and flowrate to efficiently construct large diameter structural elements. Despite the hard and very dense ground conditions described above, design column diameters ranged from 2.5 meters [8 feet] to over 3 meters [10 feet] while select installed columns had diameters exceeding 4 meters [13 feet].

Test programs were performed at the south end of the CHS box and at the PSST. These test programs each included six full depth production columns with varied withdrawal rates (pullrate) and primary grout mix designs. Taking into account the very dense ground conditions at each site a large jetting flowrate was maintained throughout without consideration of reducing injection rates and energy levels. The six interconnected test columns were installed with specific attention given to observation of grout return specific gravity and communication between adjacent fresh columns.



**FIG. 5. Model of Possible Locations and Angles of Jet Grout Installation at the PSST.**

Intercommunication between adjacent columns installed fresh-on-fresh will be the first form of feedback in terms of erosional performance or geometry achieved. Continual monitoring of grout return specific gravity will eventually yield a site specific database with respect to soil type and depth that can later be correlated to column diameter.

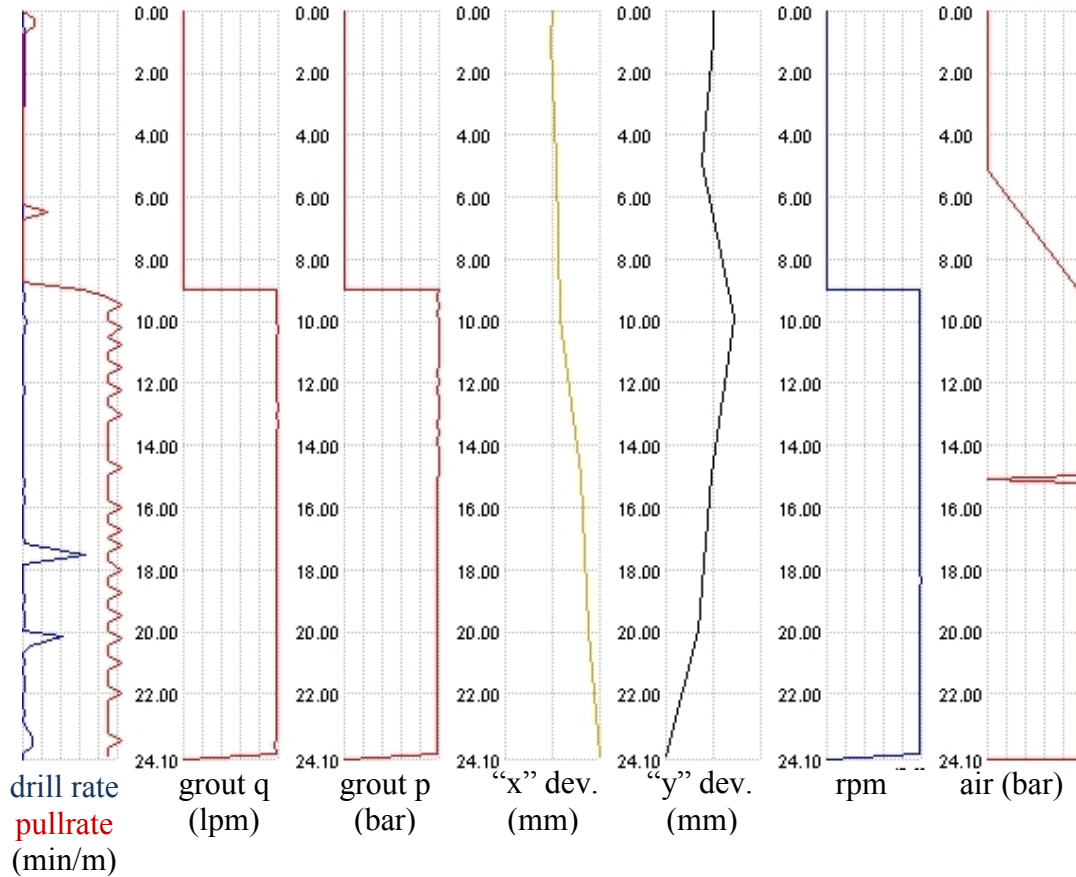
### **Quality Control Measures**

In-situ wet samples were taken from various depths of the fresh jet grout columns on a daily basis. A soilcrete retrieval tool was fashioned onto the drill string and lowered into a fresh column to obtain a discrete sample at a given depth. Samples were cast into 5 centimeter [2 inch] by 10 centimeter [4 inch] cylinders for testing of unconfined compressive strength (UCS) per ASTM D4832 and determination of unit weight. The UCS grab samples yielded 28-day strengths between 0.8 MPa [170 psi] to 11.2 MPa [1700 psi] with an average of 6.9 MPa [1000 psi], which was greater than the specified requirement of 1.4 MPa [200 psi].

Specific gravity measurements of grout return or reflow was obtained throughout a majority of the columns at random depths in order to map the in-situ conditions across the production area. Heavier reflow would indicate column construction was within an erodible strata and the resultant column radius would be greater than that of areas where the specific gravity of the reflow was much less. Specific to each project site or geologically unique area, a database was created to correlate specific gravity with column diameter within each soil stratum. Verification coring was used to calibrate the correlation.

Electronic data acquisition equipment was used to collect grouting parameters for each column. The typical full suite of electronic data was collected, which included all relevant construction parameters such as grouting flowrate, pressures, and withdrawal rate, as shown in Figure 6. However, specific to the MEGA-Jet tooling system is an advanced method of monitoring drilling deviations realtime. The jet grout monitor (lower nozzle/jet assembly) is outfitted with a GPS and inclinometer set to measure, display and record drilling deviations in both the “x” and “y” orientation (referenced to standard polar coordinates) as they occur. The GPS/inclinometer data signal is transmitted along the drill string through special E-rods (drill casing with internal wiring and electronic signal coupled joints). Mounted atop of the drill string is an electric rotary swivel to allow continuous signal response during all drilling operations. Monitoring of drill deviations becomes particularly important for deep work, approximately +15 meters [+50 feet], or for inclined drilling where deviations are more prominent and column overlaps become more critical. The electronic data was combined with the specific gravity measurements to create an estimate of as-built column locations and radii with respect to depth. This information was then imported into Solidworks, a 3D modeling software, to look for areas of un-improved soils within the jet grout blocks.

The primary element of the quality assurance (QA) program for the jet grouting operations was a series of horizontal and vertical cores. The purpose of coring was to obtain continuous samples through the jet grout blocks to look for layers of un-grouted soils and to obtain samples for UCS testing. As expected, the cored samples exhibited slightly greater strengths than the wet grab samples. In addition, the



**FIG. 6. Typical Electronic Data Acquisition Output Plotted vs Depth (m).**

horizontal core holes were to be filled with water so that in-situ permeability testing of the jet-grout blocks could be performed.

At the south end of the CHS box a total of eight cores were advanced through the soilcrete block. The cores were drilled with a HQ3 barrel and produced average sample recovery rates of 65%. The highest recovery rate was 98% while the lowest was 48%. The recovery rates in the fluvial and lacustrine deposits were not significantly different and slightly lower recovery rates were observed in the first and last 1.5 meters [5 feet] of each core run.

At the north end of the CHS box a total of four cores were advanced through this jet grout block. The cores were drilled with the same HQ3 tooling and produced an average sample recovery rate of only 31%. The highest recovery rate was 47% and the lowest was 19%. The sample recovery rates increased with depth through the fluvial deposits and were greater in the fine-grained lacustrine deposits than in the fluvial sands. The low recovery rates in the upper soils are likely due to the amount of gravels present within the soilcrete matrix and the relatively high strength of these clasts compared to the fresh grout.

At the PSST, the first two cores were advanced with HQ3 tooling and produced an average sample recovery rate of 29%. In an effort to increase the amount of recovery,





**FIG. 7. Condition of Jet Grout Face for Lift 5.**

the coring procedure was subsequently converted to continuous SPT sampling, which produced an average recovery rate of approximately 50% but retrieved a physical sample at consistent intervals throughout the depth profile. Sonic drilling was also considered but never performed.

### **Observational Method for the North CHS Jet Grout Block**

As the cores through the jet grout zones did not produce sufficient recovery to fully evaluate the performance criteria an observational approach was adopted. The observational approach focused on the north jet grout block at the CHS site. The performance of this jet grout block was considered critical as the ground improvement was necessary to reduce ground movements due to tunneling and station excavation and to prevent the fluvial deposits from piping during construction. The observational approach had three primary components: documenting the condition of the exposed jet grout face as the CHS was excavated; observing soil/block conditions as ground anchors were installed; and measuring ground deformations in and around the block as the excavation progressed.

The excavation of the CHS site began in the middle of July 2010 and was completed in late January 2011. The shoring system for the north wall of the excavation consisted of soil nails with a shotcrete facing. This system was installed in a top-down progression and the south face of the jet grout block was observed as each lift was constructed. A panoramic photograph (Figure 7) was taken to document the condition of the jet grout face for each lift.

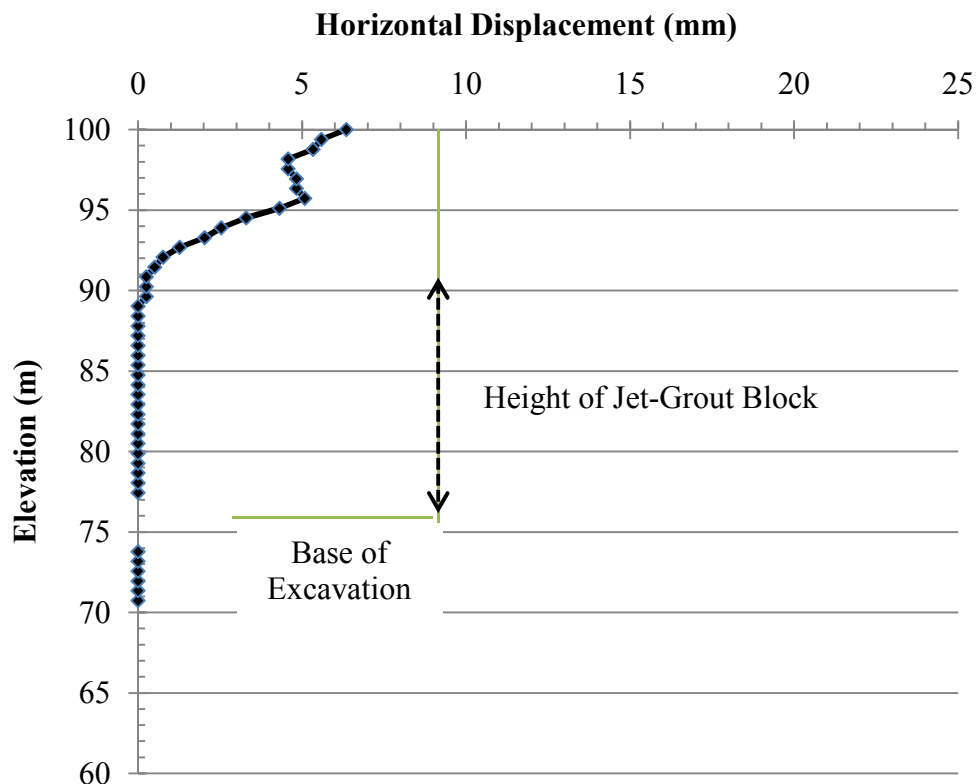
The photographs of the exposed face(s) suggest that the jet grouting achieved a relatively uniform level of ground improvement at the south face of the block in the fluvial soils. No pockets or layers of un-grouted soils were observed within the face. A piezometer installed to the north of the jet grout block indicated that the groundwater level was approximately 3 meters [10 feet] above the base of the CHS as it reached the design depth however no seepage was observed coming through the face of the block. Groundwater seepage was noted however in the northwest corner of the excavation flowing around the edge of the ground improvement zone.

Soil nails supporting the north headwall were advanced with an impact hammer through the jet grout block. The nails in the top half of the block encountered jet grouted soilcrete from the face of the excavation to approximately 10.5 meters [35 feet] to 12 meters [40 feet] feet north of the wall as anticipated. As the soil nail wall progressed downwards the anchors occasionally encountered moist to wet sands 7.5 meters [25 feet] to 10.5 meters [35 feet] behind the face of the excavation and to the north of the block after 12 meters [40 feet]. In general, the observations of the soil nail installations suggest that the upper half of the jet grout block is relatively continuous but that a wedge of ungrouted material may be present near the base of the

block, 7.5 meters [25 feet] to 10.5 meters [35 feet] from the face of the CHS excavation. The ungrouted wedge is envisioned to be attributed to the lack of column overlap within the deeper reaches of the battered column array where the efficiency of the array diminishes with increasing depth. This finding was consistent with the results of the Solid Works modeling created from the as-built data.

The overall performance of the jet grout zone can also be seen in the lateral displacements of the soil nail wall as the excavation was advanced. This movement was monitored by an inclinometer installed at the center of the wall. The inclinometer was read daily during excavation and the displacement profile over the depth of the instrument is presented in Figure 8.

The lateral displacement at the top of the soil nail wall was approximately 6 millimeters [0.25 inches] when the CHS excavation was completed in January 2011. This displacement developed as the excavation was advanced from the ground surface to elevation 89 meters [293 feet]. No significant movement was observed as the excavation progressed from this point to the final design elevation of 77 meters [252 feet], approximately 23 meters [75 feet] to 24 meters [78 feet] below the adjacent grade. The displacement of the inclinometer indicates that almost no movement occurred within the jet grout zone.



**FIG. 8. Horizontal Displacement of Soil Nail Wall at North End of the CHS vs Depth.**

## **SUMMARY AND CONCLUSIONS**

One of the most difficult challenges posed by the U230 ground improvement programs was evaluating the effectiveness of the jet grout based on the QA/QC cores. These cores showed that the strength requirements were satisfied whenever testable core was recovered. However, the cores produced so little recovery that no conclusions could be supported as to the overall consistency of the block. The in-situ permeability testing would likely have helped to determine if un-grouted zones were present but these tests had not been performed.

The observational approach for the north jet grout block provided a significant amount of data that was used to evaluate the overall consistency of the block and to map areas of potentially un-grouted soils. This approach required a degree of flexibility between the expectations of the owner, contractor, and designer. While not an issue on this project, the potential drawback of this approach was that waiting until excavation started precluded any remedial jet grouting from being performed if significant problems were encountered. However, it was noted that remedial work in these applications do not typically accept additional jet grouting but are more amenable to other grouting techniques performed from the face of the excavation.

## **ACKNOWLEDGEMENTS**

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