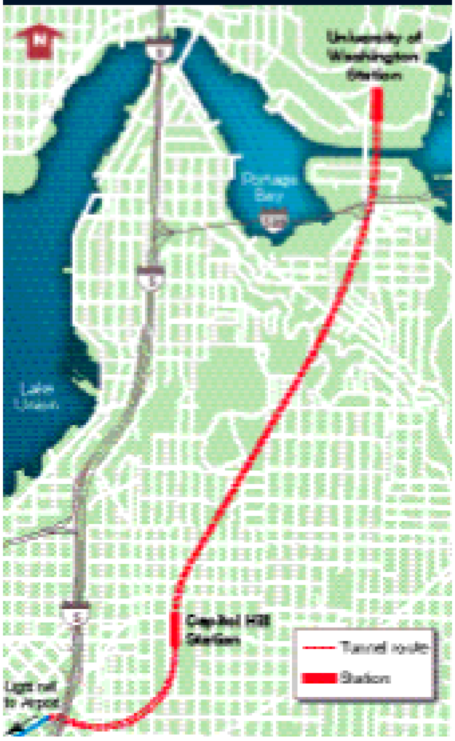
Making it sound for Seattle's U-Link

Richard Hanke, PE, of Malcolm Drilling Company, Phaidra Campbell and Colin Lavassar, PE, of Jacobs Associates, and John Sleavin PE, of Sound Transit, discuss the ground improvement program required for the construction of Seattle's University Link (U-Link) U230 contract, to assist with tunneling operations at multiple locations

THE COMPLEX GEOLOGY and congested underground utilities of Seattle's Capitol Hill have posed some of the greatest challenges on Sound Transit's University Link (U-Link) light rail U230 tunneling contract, part of the city's developing Link Light Rail system. The U-Link project will extend Seattle's existing light rail service from the downtown core of the city to the University of Washington via Capitol Hill. The extension consists of a series of tunnels that are being built under multiple contracts. This article focuses on one of the project's major components, the U230 Contract, which includes construction of twin tunnels from the Capitol Hill Station (CHS) south to the existing rail within the downtown transit tunnel on Pine Street (Figure 1); a 550ft (168m) long station on Capitol Hill; and a TBM retrieval shaft at the existing Pine Street Station and Stub Tunnel (PSST). The Capitol Hill Station interfaces with another component of the U-Link project, the U220 Contract, at the north end, along East John Street, between Broadway and 10th Avenue. The U220 Contract includes construction of twin tunnels between Capitol Hill and the University of Washington to the north.

Figure 1: Map showing the U-Link tunnels and contracts



Tunnels enter and exit at both ends of the Capitol Hill Station (CHS), requiring construction of two large ground improvement zones to assist with TBM launch and retrieval. Two smaller ground improvement zones were also needed to receive the TBM at the Pine Street Stub Tunnel (PSST). The ground improvements were achieved by constructing overlapping large-diameter jet grout columns to provide a solid mass of strengthened impermeable ground. The following article explains the need for ground improvement and examines several major components of the U230 Contract ground improvement program: surface and site conditions, initial design considerations, construction challenges, quality control measures, verification testing, and in situ conditions post construction.

Subsurface and site conditions

The geology at the north end of the CHS consists of glacial till and diamict (Qvd) from ground surface elevation 327ft (100m) to elevation 300ft (91m); nonglacial fluvial deposits (Qpnf) between elevation 300ft (91m) to 260ft (80m); and underlying layers of nonglacial 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 TBM drives break into the CHS between elevation 263ft (80m) and 283ft (86m).

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 ground surface elevation 325ft (99m) to elevation 270ft (82m); fluvial deposits from elevation 270ft (82m) to 265ft (80.5m); 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 because of the highly variable nature of the given geology.

The subsurface conditions at the Pine Street Stub Tunnel (PSST) consist of artificial fill (Af) from ground surface elevation 176ft (53.5m) to elevation 140ft (42.5m); landslide (Qls) and wetland deposits (Qw) between elevations 140ft (42.5m) and 115ft (35m); and underlying layers of nonglacial lacustrine deposits (Qpnl) and Pre-Vashon diamict (Qpgd). The U230 tunnels will break into the PSST excavation between elevations 124ft (37.5m) and 104ft (31.5m), 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 to slow raveling when unsupported. While the static groundwater table was estimated to be at elevation 104ft (31.5m), ground improvement was still required at both break-in locations.

Design considerations & rationale

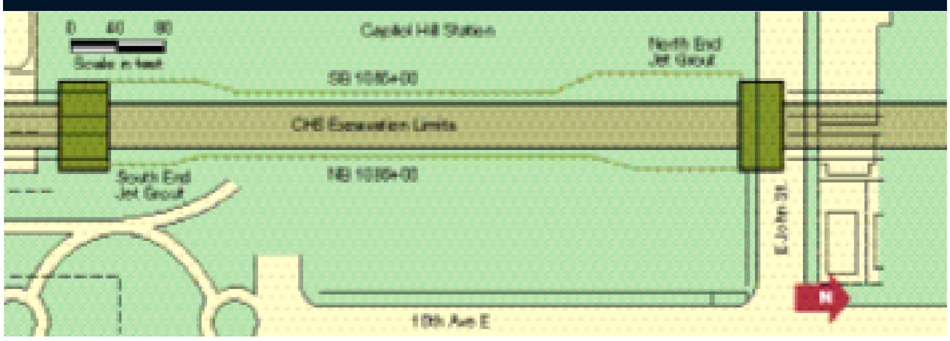
The U-Link tunnels are being mined with earth-pressure balance (EPB) TBMs (see p7). While EPBMs can provide continuous support to the native soils along the majority of the tunnel alignments, it is difficult to maintain face pressures at the break-in and break-out locations at the CHS and PSST sites, hence the need for localized ground improvements.

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

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 50ft high, 80ft long and 40ft wide (15m high, 24m long and 12m wide), normal to the tunnel alignment. The dimensions of the blocks provided approximately 20ft (6m) of improved soils above the tunnels and 10ft (3m) below and to the sides. The width was based on assumed TBM lengths of 30ft (9m) and the desire to have at least 10ft (3m) of the final tunnel lining sealed and securely grouted within improved soils before the cutterhead of the TBM breached the safety of the improved zone. A secondary goal of the ground improvement at the CHS was to improve the performance of the excavation support systems to limit shoring displacements (see 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

Figure 2: Locations of Jet Grout Zones for the Capitol Hill Station (CHS)

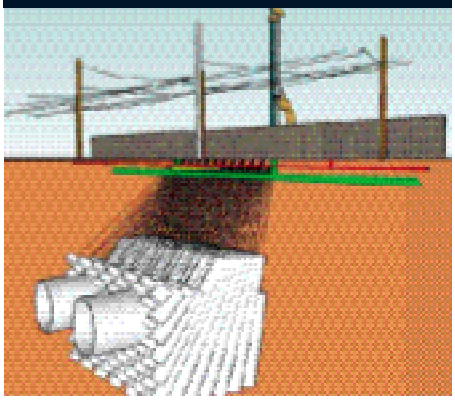


two zones posed significantly different levels of risk. The south zone is within the confines of the U230 CHS construction site and is not adjacent to any major buildings or streets. However, the north block is constructed beneath the public right-of-way on East John Street and is within 25ft (7.5m) of a large apartment building. The north zone thus posed a greater geotechnical challenge, it 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 those at the south.

The construction of the north jet grout block was also complicated by the presence of multiple buried and overhead utilities and high traffic volumes on East John Street. Although the jet grouting operation could encroach within the sidewalk of East John Street traffic lanes had to remain open. 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.

At the PSST, the dimensions of the two jet grout blocks are significantly different because of the buried utilities and anticipated

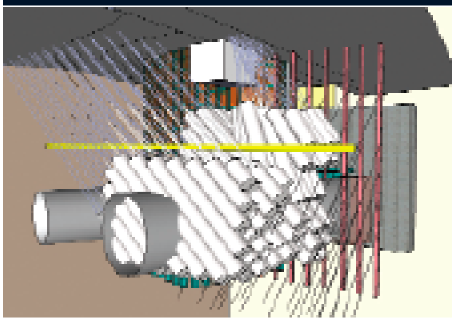
Figure 3: Preliminary design model of possible locations and angles of jet grout columns at the north side of the CHS



construction operations. For the northbound tunnel, the jet grout zone dimensions were 40ft wide by 20ft long by 40ft high (12m wide by 6m long by 12m high), and the block was designed to provide a secure face to allow the steel soldier piles of the TBM retrieval shaft to be removed after the TBM was received. For the southbound tunnel, the jet grout zone is 40ft wide by 75ft long by 40ft high (12m wide by 23m long by 12m high), with a step to clear an existing deep combined sewer utility. This block not only provides a secure face to allow the steel soldier piles to be removed from the TBM alignment, but it also provides 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 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 lifted during the construction phase.

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 4. However, these restrictions were lifted during construction and one lane and the sidewalk were taken out of public service. Being able to encroach on the roadway reduced the number and inclination of the angled jet grout columns, which resulted in a more efficient installation.

Figure 4: Model of possible locations and angles of jet grout installation at the PSST



Jet grouting

Jet grouting started in March 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. Proprietary MEGA-Jet technology was used, which incorporates 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 flow rate to efficiently construct large-diameter structural elements. Despite the very dense ground conditions, column diameters ranged from 8ft (2.5m) to over 13ft (4m).

Test programs were performed at the south end of the CHS box and at the PSST. Each of these test programs included six full-depth production columns with varied withdrawal rates (pull rate) and primary grout mix designs. Taking into account the very dense ground conditions at each site, a large jetting flow rate was maintained throughout, without the need to consider 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. Intercommunication between adjacent columns installed fresh-on-fresh is the first form of feedback in terms of erosional performance or geometry achieved. Continual monitoring of grout return specific gravity eventually yields a site-specific database with respect to soil type and depth that can later be correlated to column diameter.

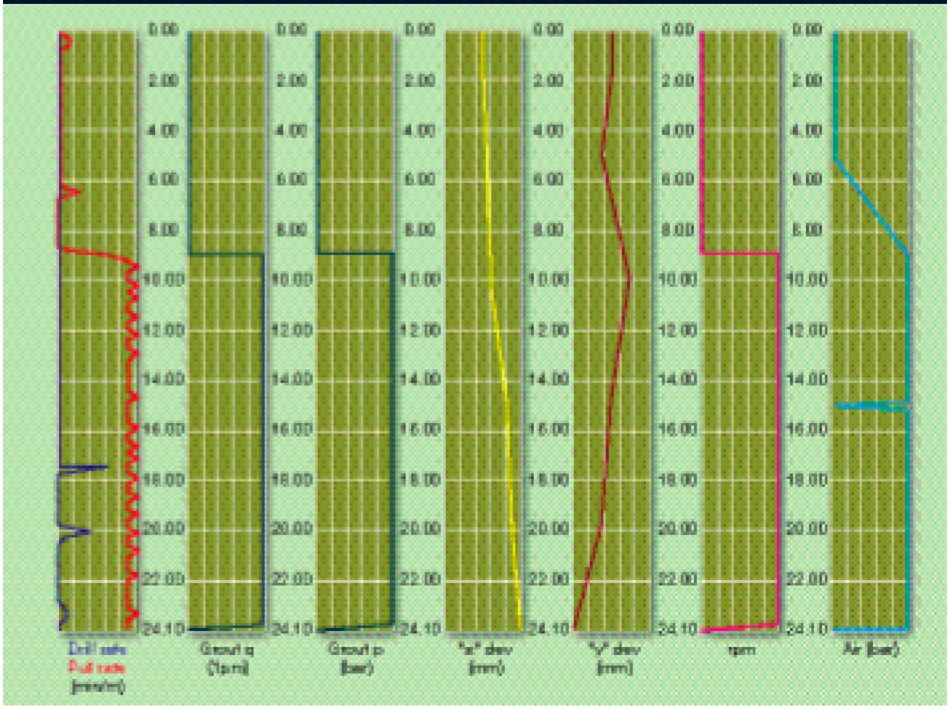
Quality control measures

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 2in (50mm) by 4in (100mm) cylinders for testing of unconfined compressive strength (UCS) per ASTM D4832 and determination of unit weight. The UCS grab samples yielded 28-day strengths of between 170psi (0.8MPa) and 1,700psi (11.2MPa), with an average of 1,000psi (6.9MPa), which was greater than the specified requirement of 200psi (1.4MPa).

The specific gravity of the grout return or reflow was also monitored during column installation. This was done 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. A database was created for each ground improvement zone to correlate specific gravity with column diameter within each soil stratum. Verification coring was then used to investigate zones of potentially unimproved soils and to further refine the model of in situ conditions.

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 flow rate, pressures and withdrawal rate, as shown in Figure 5. However, specific to the MEGA-Jet tooling system is an advanced method of monitoring drilling deviations in real time. 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 "x" and "y" orientations (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

Figure 5: Typical electronic data acquisition output plotted vs depth (m)



atop 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 +50ft (+15m), or for inclined drilling where deviations are more prominent and column overlaps become more critical. The electronic data were 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 unimproved 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 ungrouted 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 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 5ft (1.5m) 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 were 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, 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 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. 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 excavation progressed.

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 6) was taken to document the condition of the jet grout face for each lift.

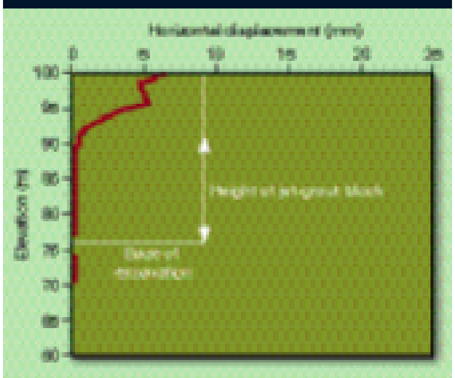
The photographs of the exposed face(s) suggest that the jet grouting achieved a relatively uniform level of ground improvement at the south face of the block in the fluvial soils. No pockets or layers of ungrouted soils were observed within the face. A piezometer installed to the north of the jet grout block indicated that the groundwater level was approximately 10ft (3m) above the base of the CHS as it reached the design depth; however, no seepage was observed coming through the face of the block. Some groundwater seepage was noted 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

Figure 6: Condition of Jet Grout Face for Lift 5

approximately 35ft (10.5m) to 40ft (12m) north of the wall, as anticipated. As the soil nail wall progressed downwards, the anchors occasionally encountered moist to wet sands 25ft (7.5m) to 35ft (10.5m) behind the face of the excavation and to the north of the block after 40ft (12m). In general, the observations of the soil nail installations suggested 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, 25ft (7.5m) to 35ft (10.5m) 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

Figure 7: Lateral displacement of soil nail wall at north end of the CHS vs depth



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 7.

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

The first TBM mined through the jet grout block at the north end of the Capitol Hill Station in late March 2012 (see p7). The TBM had a length of 37ft (11.3m), which is greater than the length assumed during the design of the jet grout block. This reduced the length of the tunnel liner that would be grouted against the jet grout block prior to the face of the TBM breaking through the soil nail wall and into the station. To mitigate the risk due to the increased length of the TBM an inflatable ring was installed on a steel frame mounted to the face of the soil nail wall as an exit seal. However, this did not turn out to be a significant issue and no soil or groundwater flow was observed around the perimeter of the TBM as it entered the station.

As the TBM mined through the jet grout block, face pressures and advance rates were reduced. This was done to limit lateral forces on the soil nail support system and to prevent overbreak of the shotcrete facing as the TBM reached the headwall. This approach was effective and instrumentation on soil nails adjacent to the tunnel eye only indicate that strains in the ground anchors increased by less than 50μ during mining.

Summary and conclusions

One of the most difficult challenges posed by the U230 ground improvement program 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 ungrouted 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, which was used to evaluate the overall consistency of the block and to map areas of potentially ungrouted 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 had been encountered. However, it was noted that remedial work in these applications does not typically accept additional jet grouting, but instead is more amenable to other grouting techniques performed from the face of the excavation.

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