BEACON HILL STATION DEWATERING WELLS AND JET GROUTING PROGRAM

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ABSTRACT

Sequential excavation of the Beacon Hill Station has been accomplished in highly complex and inter-layered sand, silt, and clay soils with multiple perched groundwater tables. Successful excavation has required the installation of a deep dewatering well system used to depressurize sand layers above and within the excavated tunnel faces. In addition, the presence of sand layers in the crown of the Platform Tunnels required jet grouting at depths of up to 49m (160-ft). This paper discusses the design intent, construction, and effectiveness of the dewatering system and the selective jet grouting of sand horizons and shaft break outs for the successful construction of the deepest SEM station excavation ever completed in the United States.

INTRODUCTION

The Beacon Hill Station and Tunnel project is a $280 million section of the Central Link Light Rail Line that extends for approximately 24 km (15 miles) between downtown Seattle and the SeaTac Airport. The project consists of an aerial station with 1.2 km (¾ of mile) aerial-guideway; twin 1.4 km (4,300-foot long), 6.4 m (21-ft) diameter running tunnels; 3 mined cross passages between the running tunnels; portal and tunnel approach structures with retained-fill and cut-and-cover structures; and a deep-mined underground binocular station with an entrance structure and surface plaza, station elevator access shaft and ventilation facility with a separate emergency egress and ventilation shaft at Beacon Hill.

STATION EXCAVATION DESIGN

The Beacon Hill Station will be the deepest station excavated in soft ground in the United States and will utilize the state-of-the-art Sequential Excavation Method (SEM; a.k.a. NATM in Europe) for construction of the station. The Station access shafts and head houses are supported with slurry walls, and the initial tunnel lining for the underground Station excavation consists of lattice-girder reinforced shotcrete. See other chapters on the Beacon Hill Station for additional figures with tunnel labels and dimensions. Station excavation has progressed through a complex sequence of glacially overridden deposits consisting of very dense and hard clay, silt and sand, gravel and cobbles with multiple ground water levels in granular deposits, typically due to perched groundwater overlying clay and till units of cohesive soils. Deep jet grouting and a system of dewatering wells have been used to solidify and drain the larger granular deposits in advance of SEM mining. This paper describes the installation jet grout columns and deep dewatering wells, and describes the effectiveness of these two primary pretreatment tools with regard to SEM tunnel excavation to date.

SITE LAYOUT CONSTRAINTS

The Contract-defined, Noise Wall-enclosed Beacon Hill Station work site is a one-acre lot, and as is likely to happen on tight-schedule, tightly constrained work site, construction activities overlapped each other with regularity. Five months after Notice-to-Proceed there were exploratory drill rigs, dewatering well drill rigs, jet grout drill rigs and plant equipment, a Hydrofraise and a Clamshell slurry wall excavator, and slurry wall desanding plant and equipment all vying for space on site. The slurry wall excavators were alternating between deep shafts and shallower head house walls. Instrument drilling and utility work was also on-going both inside and outside the site, so initial jet grouting work began on the Northbound Platform Tunnel (NBPT) and proceeded into the adjacent El Centro parking lot in an area unto itself, while dewatering wells began along the Southbound Platform Tunnel (SBPT) and south of the Main Shaft. Original jetting limits also included work for the West Longitudinal Vent Adit (WLVA) west of the Main Shaft, however with slurry wall work concentrated at this same central shaft, WLVA jet grouting was deferred, and ultimately scaled back to just two rows of jet grout columns next to the Main Shaft for the WLVA break-out. Jet grout work in the intersection of 17th Av. and Lander St. was precluded from starting until late 2004, however with the discovery of sand in the platform tunnels jetting priorities shifted to the platforms to allow for TBM passage through the station. With all this on hand it was quite a task to orchestrate all the moving pieces and still get work underway. Figure 1 shows the congested site in plan view and Figure 2 shows an aerial view of the site looking southward during early jetting and dewatering well installation work.

PROBE AND INSTRUMENT DRILL HOLE PROGRAM

Due to real estate easement acquisition and design timing constraints, only a limited number of bore holes could be completed for the station footprint during the design and preconstruction phase prior to bidding. As a result of these timing constraints, and in particular due to the highly variable nature of the soils encountered in the pre-bid design phase (which included a test shaft on-site), an exploratory drilling program was added to the scope of the construction contract which encompassed both exclusive soil-sampling holes, and sampling of geotechnical instrument holes for ground condition monitoring around the BH Station. More than 40 instrument holes, 20 probe holes, and cuttings from 20 of the 39 dewatering wells were sampled and logged within the first 18 months of construction from the vertical street surface level alone. Horizontal in-tunnel cores and probe holes from SEM production mining have also been logged as SEM mining has progressed, however the vertical probe holes were the basis for the newly defined jet grout limits and the revised locations for additional dewatering and observation wells. A plan view of the instrument and bore holes is shown in Figure 3.

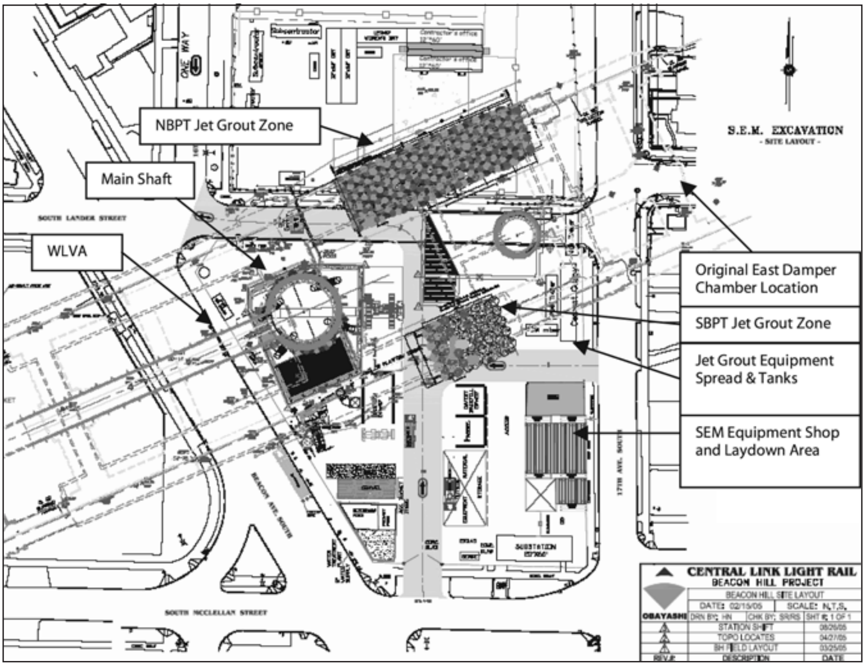

Figure 1. BH site plan

Figure 2. BH aerial photo

DEWATERING WELL INITIAL DESIGN

As part of the Geotechnical investigation for this contract, a test shaft was excavated at the location of the proposed access shaft to provide a detailed evaluation of ground conditions. The test shaft was excavated and lined with shotcrete and ring beams for excavation support, relying upon dewatering as an integral element of that scheme. Several non-cohesive granular soil zones were encountered which were not adequately dewatered and demonstrated significant instability in spite of well-points

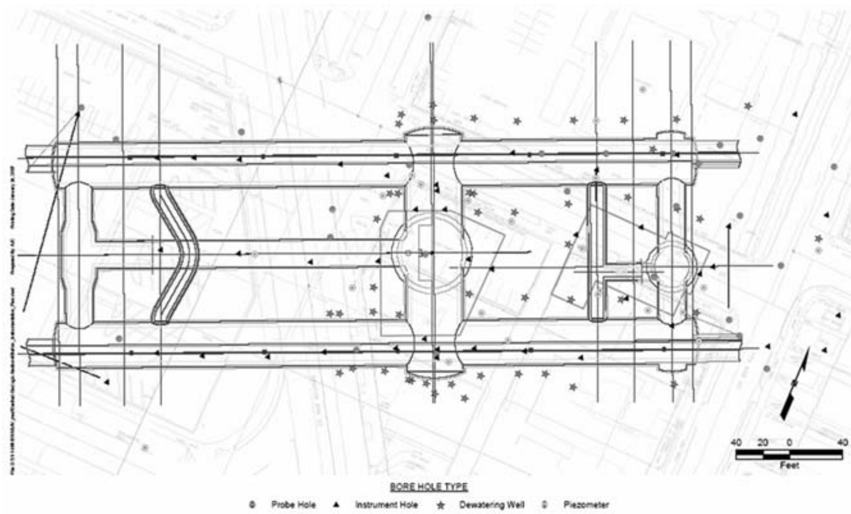

Figure 3. Instrumentation plan

and other drainage measures being implemented from within the shaft. The excavation of the test shaft by conventional methods was terminated at a depth of 30.5 m (100 ft) due to the unstable conditions, and further investigation was continued by large diameter drilling beneath. In response to the unstable ground conditions experienced in the shaft and the projected difficulties anticipated within the SEM mined tunnel, an initial array of 26 base and 12 optional vacuum assisted deep wells was laid out, heavily concentrating the dewatering effort immediately around the access shaft where the contract borings revealed non-cohesive granular soils, and the greatest potential for unstable conditions. The base 26 and additional 12 wells were designed with 9 m (30 ft) of well-screen between approximately 30–40 m (100–130 ft) depths, where the problematic soil material was anticipated.

In addition to the test shaft and initial exploratory boreholes, several aquifer pumping tests were performed in order to evaluate the hydro-geologic characteristics of this granular soil deposit and evaluate whether the material should be dewatered or modified. Well yields of approximately 75 lpm (20 gpm) were realized from the granular soils within the horizon of the northbound platform indicating permeable, non-cohesive ground which warranted jet grouting. The pump tests also indicated that there was communication between the multiple water bearing soil zones which reflected different groundwater table levels. This groundwater behavior was significant because the soils within the mined tunnel horizon were apparently recharged directly from the overlying soil zones.

DEWATERING WELL PRODUCTION WORK AND SYSTEM MODIFICATIONS

With the apparent source of recharge from overlying water bearing soil layers, there was a concern that even with the jet grouting, exterior water pressures of up to 1.7 bar (25 psi) could result in excessive external pressures on the SEM lining, as well as the potential for flowing sands through contacts between columns and through inadvertent windows in the jet grout. The originally contemplated array of 26 and 12 dewatering wells was expanded. In addition to the 18 of the first 26 wells that were already in place as part of the original scheme, an additional 21 wells (for a total of 39 on-site) were installed around the accessible areas of the whole mined station footprint to provide a more “global” or “areal” dewatering effort. These 21 new wells (some remaining original, some new/extra) were located based on localized, high pressure sand layers found in the exploratory holes, and were spaced to lower the water in the northbound platform sand to within 1.8 to 3 m (5–10 ft) of the bottom of the water bearing layer. The additional wells were constructed to depths of up to 58 m (190 ft) with screens as long as 30.5 m (100 ft) in order to tap and relieve any possible soil zones which might recharge the underlying water bearing soils in the tunnel horizon. A vacuum was applied to each well-head via dedicated piping from a surface mounted vacuum station. An additional 47 l/s (100 cfm) of vacuum capacity was provided for the expanded well system, which would have theoretically increased well discharge volumes quite substantially, however given the relatively low flow volumes for most of the wells installed to date has proved be an excellent backup device. The completed well locations and piezometers in Figure 3.

The initial 18 original “contract” wells were installed by mud rotary drilling techniques, using a polymer drilling fluid to stabilize the open 25 mm (10-in) drill-holes. The well installation technique was subsequently changed to a dual rotary, air flush technique to complete wells at greater depths as compared to the initial 18 wells, and to provide the Engineer with fail-proof quality assurance during well screen installation. With either drilling technique, well installation proved time consuming due to the very dense soil conditions. Each well required approximately four days for set-up, drilling, and well construction. Wells were toed a minimum of 1.8 m (5-ft) into competent clay and/or till layers, and cased with slotted 15 mm (6-in) Schedule 80 PVC well casing packed in filter sand, with bentonite chips or a neat cement grout plug at the bottom. Wells were furnished with 1.5 kW (2 hp) pumps capable of a maximum discharge of 115 lpm (30 gpm), with a 3 mm (1.25-in) vacuum assist line connected to one of two on-site vacuum manifolds.

Although the Station itself was to be constructed at considerable depth, a tremendous amount of underground activity had taken place within shallower depths. The system of deep wells had to mesh with the tiebacks installed for the shallower structures, the drilled barrel vault pipes over the crown of the larger SEM drifts, the jet grouting of the unstable soil zones, and of course the slurry walls and the footprint of the mined Station. Three wells were destroyed by tiebacks, four wells were damaged by the barrel vault holes, one well was grouted up, and seven additional wells were plugged by unknown means. Additionally, to protect the well system from the site activities, the well-heads had to be recessed in buried vault boxes and the well system discharge piping, vacuum piping, and electrical distribution was buried in trenches across the site. The entire well system discharge was directed to a storm drain connection on-site, however the plumbing also included piping through an automated pH adjustment (CO_2) treatment plant system for added environmental compliance assurance.

GROUNDWATER TABLE ELEVATION AND DRAWDOWN DATA

Upon startup of the dewatering system, an initial 11 dewatering wells were pumping at a combined rate of about 265 lpm (70 gpm), with most of the flow volume coming from 5 particular wells averaging 40 lpm (10 gpm) each. The remaining 6 wells produced generally less than 20 lpm (5 gpm) total. Subsequently, the remaining dewatering wells were installed and brought online, and the total flow rate gradually decreased over a 2-year period to about 40–80 lpm (10–20 gpm), as is the case at the time of writing.

Although individual well pump rates were generally low, the dewatering system has been extremely effective in lowering groundwater levels in the sand units near the

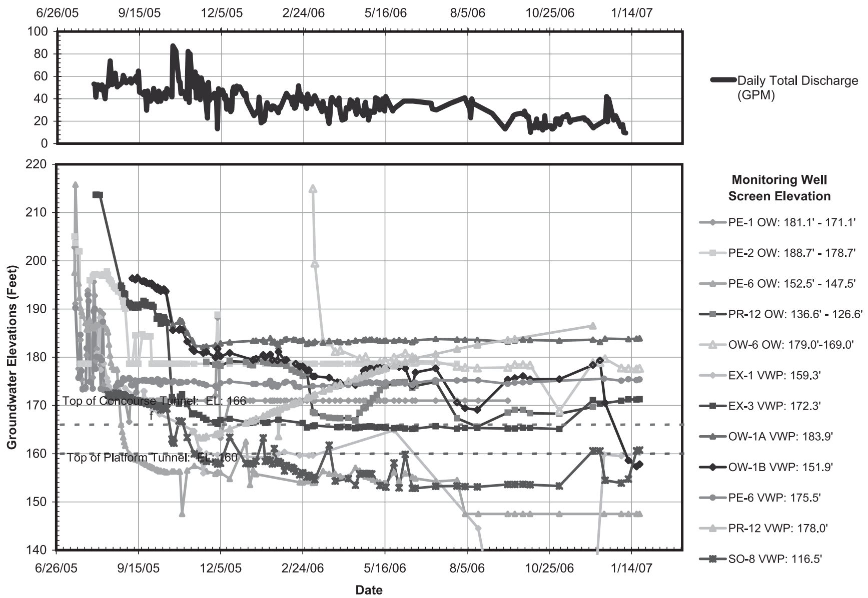

Figure 4. Well drawdown and discharge volumes

crown of the tunnel. The dewatering system reduced groundwater levels to "steady-state" conditions over about a 4 to 8-week period. After about 3 months of dewatering system operation 8 of the 19 piezometers used to observe groundwater levels were 'dry'. Groundwater levels were lowered up to approximately 18 m (60 ft) prior to excavation of the station and tunneling. It is also noteworthy that when some of the higher producing dewatering wells were shut down, such as for maintenance, the nearby piezometers showed a fairly rapid rise in groundwater levels, some as much as about 1.8 to 3 m (5 to 10 ft) in a several-week period. Refer to Figure 4 for an illustration of well drawdown and flow volumes for the well system.

GROUNDWATER IN EXCAVATED SOILS

Sequential Excavation of the Station has encountered very little groundwater in the tunnels mined to date. The first sign of groundwater was found in the South Concourse Cross Adit (SCCA) eastern sidedrift top heading where the sidewall sprung a leak 4.6m (15 ft) into tunneling. This turned out to be connected to dewatering well SN-06 and was subsequently stemmed by capping the bottom 9m (30 ft) of well casing and moving the pump up over the crown elevation. Subsequent excavation in this same heading encountered first a trickle of water and then a small 50 mm (20 in.) diameter "dike" of flowing sand from above the crown that found its way through the protective barrel vault layer. The dike was plugged with wood-wool, welded-wire-fabric and shotcrete with the addition of grouted pipe spiles in the center top heading crown. Subsequent to that only minor pockets of water in the SBPT East, SBPT West Headwall, the East Transverse Vent Adit (ETVA) north of the Ancillary shaft, and the NBPT at the north and bottom edge of the jet grout zone have been encountered. Individual drain pipes and hoses have been plumbed into the shotcrete lining to drain these features, none of which have amounted to more than a few gallons per minute initially

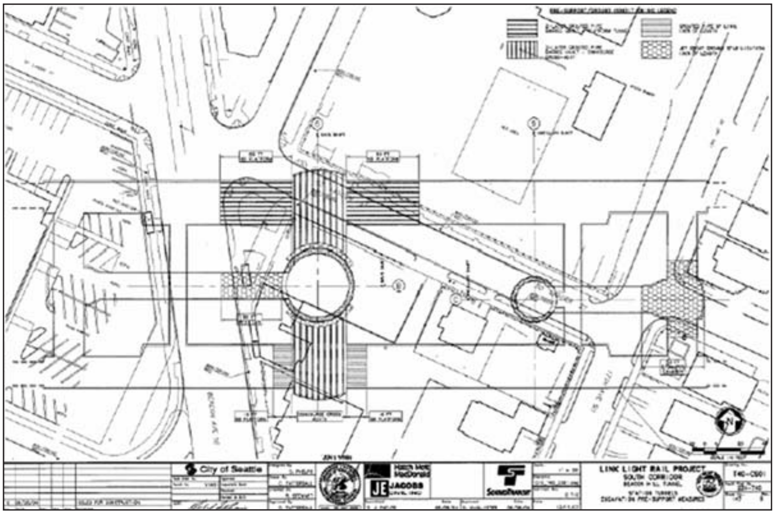

Figure 5. Original jet grout zone

before decreasing to quarts and cups per minute. Waiting on standby, but yet to be used, vacuum assisted well points have been contemplated twice but deemed unnecessary after weekend gravity draining of various well points in the SCCA and the ETVA proved to be effective.

JET GROUT INITIAL DESIGN AND LAYOUT

Jet grouting was depicted on the Contract Drawings for the Main Shaft break-out for the CCAs, along the WLVA break-out extending westward from the Main Shaft, and for the East Damper Chamber adjacent to the Ancillary Shaft. Jet grouting was intended to "pre-stabilize potentially unstable water-bearing sand layers expected in the crown of…" these various tunnels [GBR 10.3.3] and to "allow excavation with minimal water ingress…" [Spec. 02343 Par. 1.01 B]. The jet grout zones originally specified in the bid documents for the WLVA and EDC consisted of stabilizing the entire section of the tunnel face with jet grouting. In addition to the predefined zones, additional areas of jet grout were to be identified based on the exploratory drilling program. The jet grout limits were described with a plan, profile and section view (grout pay envelope), however individual column spacing and target column diameter was left up to the Contractor to determine. The Contractor-designed layout was also to include angled holes with a potential deviation from vertical of up to 20 degrees for the WLVA below Beacon Ave. west of the Main Shaft. A plan view of the original contract jet grout zones is shown in Figure 5 and the revised final jet grout zones are shown in Figure 6.

JET GROUT TEST PROGRAM

Prior to beginning the production jet grouting initially planned for the WLVA and the Main Shaft Tunnel eyes, a test program was performed to verify the relationship between jetting energy and the resulting jet grout column diameter. On most projects with working depths less than 12–18 m (40-ft to 60-f), the pre-production test columns are typically jetted near the ground surface and exposed by excavation to allow visual

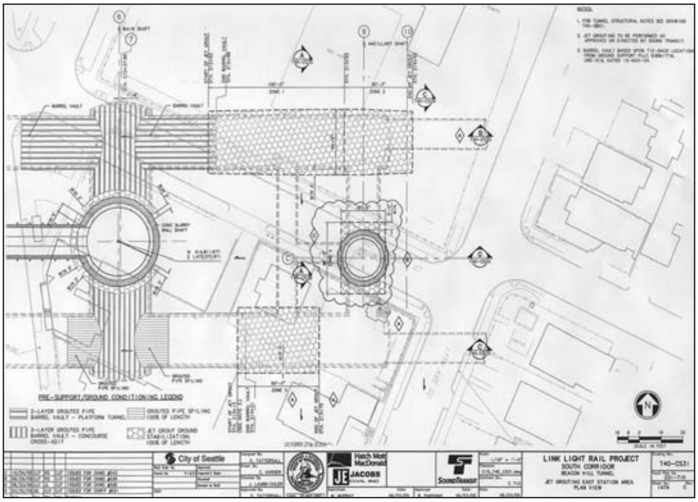

Figure 6. Revised jet grout plan

measurement and inspection of the columns to develop the jetting energy versus column diameter relationship.

Due to the jetting depths involved on this project, the use of shallow columns would have very limited applications to the production depths for two reasons. First, the near-surface soils in the available test area were very dense glacial tills with a substantial fines content that was not representative of the granular materials to be treated at depth. Second, the ground water pressure and the overburden stresses at a depth of 42.5–49 m (140-ft to 160-ft) differ so substantially from the near-surface conditions that an effective correlation can not be made between shallow test columns and the deeper production columns.

Normally, in cases where near surface ground conditions are not representative of the production work depths and where excavation and visual inspection of columns at the required depths is not practical, a pattern of columns would be installed at the working depth and cored at the intersections of the columns to verify closure of the columns. When coring is used to verify closure of a pattern of columns, there is less opportunity to minimize column overlap and refine the jetting energy. On this project, the Contractor decided to use a geo-physical testing method, "electronic cylinder testing," in conjunction with test column coring. The electronic cylinder test method attempts to determine the column diameter by measuring the difference in electric resistivity between a fresh jet grout column and native soils.

The resulting test program consisted of three shallow columns and four deep columns. The diameter of the shallow columns was measured using the electric cylinder followed by excavation and visual examination. The diameter of the deep columns was measured using the electronic cylinder method with limited coring to verify compressive strength. Due to the uniqueness of the electronic cylinder testing, the Contractor chose to perform a shallow test section to convince the project team of the usefulness of the geo-physical testing method by visually verifying the electric cylinder diameter projections on a shallow column. In the case of the shallow columns, visual observation was the primary method of measuring column diameter and the electric cylinder was the secondary means. Once the effectiveness of the testing method had been proven, it could then be used on deeper columns where the electric cylinder would be

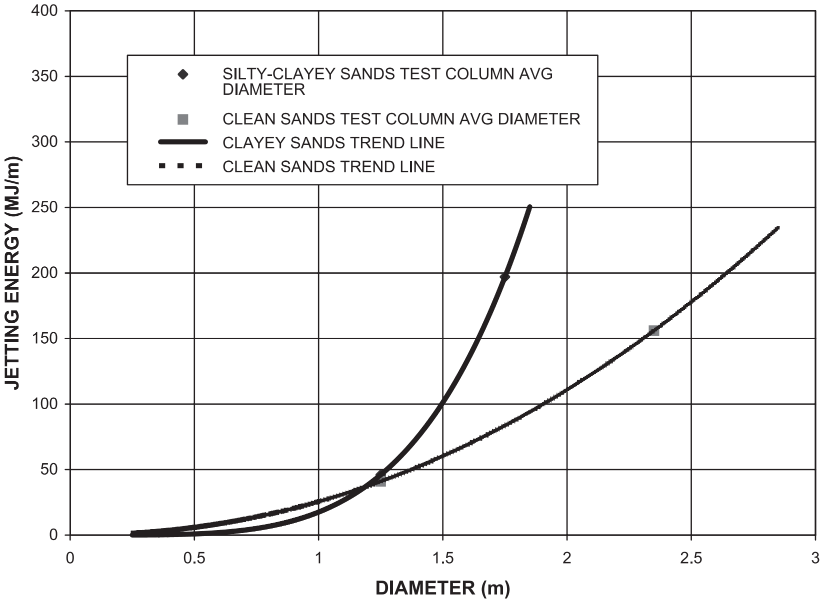

Figure 7. Jetting energy plot

the primary method of verifying the column diameter and coring would be the secondary means. The results of the shallow and the deep test program are presented below in Figure 7, with jetting energy per meter plotted against column diameter for the different soils encountered.

PRODUCTION WORK STARTING AT NBPT— PREDRILLED PATTERN DEVELOPMENT

After analyzing the results of the test columns to determine a reasonable range of expected jet grout column diameters for the target soils, a preliminary pattern of columns was laid out with a triangular spacing. At shallower depths, column layout pattern typically is based on expected diameter and the maximum drilling deviation measured or expected. The tolerance for potential gaps in the treatment is a function of whether the goal of the jet grouting is soil stabilization or groundwater control. Based on these three factors, a layout is chosen and the work typically proceeds without borehole surveys.

In the case of the depths involved at Beacon Hill Station, the potential drilling deviation associated with depths of 42.5–49 m (140-ft to 160-ft) would have resulted in an excessively tight column layout. Since the contract specifications required that every hole be surveyed, it would have been possible to base the layout on an expected maximum deviation and then perform some remedial work in the cases where the allowable drilling deviation was exceeded. However, it was determined that pre-drilling with hole surveying followed by adjustment of the column pattern and/or jetting parameters prior to performing the jetting would be a more efficient and controlled method of carrying out the jet grouting. In order to effectively utilize this method of pre-drilling, borehole surveying, and layout adjustment, a smaller working area within the larger column layout needed to be used. The actual working pattern within the global layout consisted

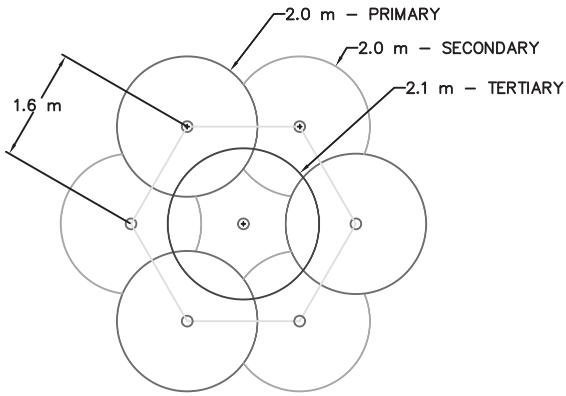

Figure 8. Grout column layout

of a hexagonal pattern consisting of three primary columns and three secondary columns at the nodes of the hexagon and one tertiary column inside the six perimeter columns. Figure 8 shows a typical hexagonal pattern for the jet grout columns taken from within the global triangular-column spacing layout.

Downhole Surveys

The contract specifications required that all jet grout drill holes be surveyed. While there are numerous means of surveying open boreholes using magnetic based systems, there are essentially just two means of surveying a drill hole through ferrous metal casing or drill rods. One method consists of lowering a survey probe consisting of inclinometers and gyroscopes down a hole with a cable. The probe is then stopped at regular intervals to take measurements and the information is then downloaded and processed by a PC to generate a drill-hole profile consisting of discrete measurement points connected by straight lines. A second method consists of lowering a survey probe consisting of x–y inclinometers down a hole via a set of indexed rods. The probe is then stopped at regular intervals to take measurements and the information is downloaded and processed by a PC to generate a drill-hole profile. While both systems are essentially the same, the first utilizes a gyroscope to determine the orientation of the inclinometers at each measurement point while the second uses a rigid rod system to maintain the orientation of the inclinometer in the same direction during the survey. The Boretrak MKII rigid-rod system was used for drill-hole surveys at BH Station.

Pre-Drilling Methods

Jet grout hole pre-drilling was performed using a dual-rotary cased-hole drilling system. Although the use of mud-rotary drilling for the pre-drilling was considered, the use of the cased-hole drilling offered three advantages. First, the contractor was familiar with the drilling method from conventional micro-pile and tieback drilling such that the cost per foot for the cased-hole system was substantially lower than mud rotary. Second, the use of dual-rotary drill has the potential to produce consistently straighter holes than conventional mud-rotary methods. Third, the use of casing allowed the pre-drilling crew to immediately proceed to the next hole without having to wait for the drill-hole survey to be performed since surveying would be performed inside the casing and not through the drill rods. If mud-rotary without casing had been used, the drill crew would have had to wait for the survey to be performed through the center of the drill steel before tripping out the drill rods and moving to the next drill-hole.

Pre-Drilling Equipment. The pre-drilling was performed using a Klemm 806-4 drill rig with dual-rotary drilling motors and an air/water flush system for the drill cuttings. Pre-drill holes were cased with 15 mm (6-in) flush joint threaded casing in 2-m (6.5-ft) lengths, and the drill holes were advanced with tri-cone bits due to the presence of cobbles and boulders within the glacial till and outwash encountered in many of the pre-drill holes. The pre-drill holes were advanced to a depth of 2-m (6.5-ft) above the top of the jet grout zone in most cases. The holes were not advanced through the treatment zone to avoid potential communication of the high energy jets with the adjacent pre-drill holes. Once the pre-drill holes had been advanced to the depth of roughly 40m (130 ft), the holes were then surveyed and tremie-filled with a weak cement-bentontie grout to prevent the hole from collapsing prior to jetting and to avoid any potential for communication during jetting before removing the casing. Depending on site constraints, the pre-drilling crew was typically able to advance, survey, and tremie backfill approximately 3–4 holes per 10-hr shift.

Jet Grouting Methods

All jet grouting on the BH Station was performed using a double fluid jetting system. The double fluid system consists of a high-energy jet of fluid grout surrounded by an air shield to increase cutting efficiency. The grout mix consisted of water and cement with a specific gravity of 1.46 to 1.48. Jetting grout flows were on the order of 380 liters per minute (100 gpm) with line pressures of approximately 400 bar (5,800 psi) at the drill rig. Air pressure was varied with depth of treatment with a maximum pressure of 15 bar (200 psi) and a flow rate of 5200 liters per minute (185 cfm). The air pressure was set slightly above the fluid pressure of the column of jet grout spoils in the annulus of the drill hole above the treatment depth. By keeping the air pressure above the fluid pressure of the spoils at the bottom of the drill hole, the air shield is able to effectively increase jetting efficiency of the grout jet.

Jetting Parameters and Hole Deviations

Jet Grout Equipment. The jet grouting was performed using a Klemm 3012 which has a standard maximum single stroke drilling and jetting depth of 26 m (85 ft). The Klemm 3012 on this particular project was equipped with a three-rod magazine which increased the maximum drilling depth to 44 m (145 ft) although the rod magazine was replaced by manually adding rods with an auxiliary winch for safety reasons during the course of the project. Drill rods were 114-mm (4.5-in) in diameter due to the drill depths and the relatively high grout flow rate through the center annulus of the double system rods. The cement grout for the jetting was provided by a Recirculating Continuous Mixer (RCM), and the grout density was monitored with a Coriolis-type mass flowmeter. Once mixed, the grout was pumped at the high pressures required for jet grouting with a Soilmec ST-500J pump.

Jetting Parameters and Hole Deviations

As indicated previously, the procedure for performing the jet grouting consisted of pre-drilling and surveying the holes at the triangularly spaced locations and then constructing a plan view of the as-drilled hole locations at the top of the treatment depth. Once this plan view was constructed, the jet grout column layout and work sequence was then developed. In the cases where the drill-hole deviation precluded the use of the original design pattern, the column diameter and sequence was then adjusted to achieve the desired treatment based on the actual drill-hole pattern at the top of jetting column. When the drill-hole deviations were on the order of 0.5%, the only adjustment necessary was usually just a resizing of the target column diameter. However, larger drill deviations could result in altering the jetting sequence, jetting adjacent columns fresh-and-fresh, and possibly pre-drilling another hole. All abandoned holes were either jetted with minimal energy or redrilled and backfilled with cement grout to avoid leaving a potential preferential groundwater conduit into the future SEM tunnel horizon. The jet grout column diameters typically ranged from 1.8 m (6 ft) to a maximum of 2.6 m (9 ft), and it was often necessary to use fresh-and-fresh jetting sequences to create a continuous mass of soil-cement from the resulting drill hole geometry. The fresh-and-fresh method involved jetting two adjacent columns rather than following a primary/secondary/tertiary sequence with tertiary columns filling the remaining intersection between already jetted (hardened) primary and secondary columns. In total 565 columns were pre-drilled with a redrill rate of less than 5% required due to vertical tolerances. As a result of the pre-drilling, surveying and re-drilling prior to jetting, the number of jet grout columns actually jetted was within 2% of the theoretical design quantity of 565 columns.

Production Quality Control

Wet Spoil Density and Compressive Strength Testing. During the initial test columns and the production jet grouting, the wet spoils coming from the annulus around the drill steel during the jetting were collected for testing. Immediately upon sampling, the density of the wet spoils were measured in a mud balance and compared to baseline density measurements as a measure of the efficiency of the jetting which is indirectly an indication of the column diameter and soil type assuming that the jetting energy is relatively constant. In the sands, more efficient jetting resulted in high spoil densities which is indicative of larger diameters while the same jetting energy in the silts and clays would produce lower spoil densities which was indicative of smaller diameters as expected.

The spoil samples were then cast into 75mm × 150mm (3 × 6-in) cylinders for curing and compressive strength testing. Although the compressive strength measured by core samples was the specified acceptance criteria, the contractor took daily wet samples to provide more timely feedback that the jet grouting was achieving the minimum specified strength of 3.5 MPa (400 psi). Due to the variability of the soils at the site and the tendency for the strongest core samples to survive the coring operation in tact with a minimum L/D of 2, it is difficult to make any specific observations on the relationship between core sample strength and wet sample cylinder strengths. However, in general the wet spoils sample strengths were typically in range of 5 to 7 MPa (600–800 psi), and the in-tact core samples ranged from 5 to 14 MPa (600–1,400 psi). This higher maximum strength from the in-situ samples is a function of the larger gravel aggregate which is less likely to be expelled with the spoils at the top of the drill hole, the fact that core samples were typically well older than 28-days prior to testing, and the effect of consolidation of the in-situ soil-cement due to the weight of the spoil column above the treatment zone during initial set and curing.

Data Acquisition. The Bi-Tronics data acquisition system on the Klemm 3012 drill rig provided a continuous readout of all drilling and jetting parameters in the cab of the drill rig. Both the drill operator and field engineering staff could monitor the grout density, pressure, and flow; air pressure and flow; pull speed; rotation speed; and depth. All of these jetting parameters were also recorded and downloaded daily to create a graphical print out of each column jetted for review by the Contractor and submission to the Owner. The Bi-Tronics system was programmed to automatically control of the drill rod rotation speed and withdrawal speed during the jetting. Although the use of data acquisition for smaller projects is often cumbersome without replacing the need for a diligent operator, on a project of the size and nature of this the data acquisition records, and more importantly the daily review of those records was directly responsible for catching several field mistakes which had they gone unchecked could have had unfortunate consequences for the subsequent SEM tunnel operations in the station headings.

Quality Assurance

Core Drilling and Compressive Strength Testing. The contract required that a confirmation core hole and permeation test be performed for every 150 cubic meters (200 CY) of ground jet grouted. In order to prevent excessive core damage and increase recovery during coring, the in-situ soil-cement was typically allowed to cure and gain strength for 3 to 4 weeks following jetting before coring. The core locations were selected at both the intersection point and the mid-point of completed jet grout columns. Prior to mobilizing the core drill, cased holes were advanced by the pre-drilling operation and surveyed to insure that the core was taken from an appropriate location. Coring was performed using a triple tube PQ system which produces cores with a diameter of 85 mm (3.5-in). Of the thirty cores performed during the course of the project, the core recovery criterion was achieved in all but one of the cores in the targeted sands and gravels, however several of the cores contained sections of hard yet unjetted silts, tills and clay. Although it is difficult to truly verify the exact quality and strength of the in-situ material since the coring process tends to destroy and thereby remove from consideration weaker samples, the cores tested met the minimum strength criteria in all cases. Of the 360 holes over the NBPT only five required rejetting (two from the early learning curve and three as added insurance based on less-than-ideal core spoils recovery). None of the columns for the shaft breakouts or the SBPT required rejetting.

Falling Head Piezometer Test. The contract documents required that a falling head permeability test be performed in the treated jet grout zone to verify that the maximum permeability was less than 3×10^{-6} cm/sec. With the original jet grout work areas encompassing the entire height of the EDC, the method of assessing the results of the falling head test were more straight forward than in the station platforms where the jet grout treatment zone might only be 3 to 4 meters thick. In the end, the core holes themselves were typically used to run the permeability test. An inflatable packer was seated at the top of the borehole, and a falling head permeability test was run until a consistent flow rate was measured which indicated a steady seepage pattern. The results were then analyzed using a Hvorslev's equation for a well point filter at an impervious boundary. The equation that was used produced a range of permeability values depending on assumptions relative to horizontal and vertical permeability coefficients, and although it is difficult to justify or verify all the assumptions for this equation in this application, the calculated permeability ranges met the contract requirements and provided an indication of good quality (low permeability) jet grouted soil mass.

EXCAVATION OF TREATED SOILS

Sequential Excavation break out work for the WLVA from the Main Shaft began in late June 2005 exposing the finished jet grout product for the first time. Evidence of the jetted column was seen clearly two rows deep and with the exception of a very small unjetted pocket at the outskirt of the crown, the exposed ground stood up perfectly. In September 2005 CCA break out work from the Main Shaft began, and again a solid mass of jet grout was found behind the slurry wall across the entire opening. Platform tunnel jet grout was first encountered in June 2006 in the SBPT and again was found to be a competent mixture of jetted columns mixed with till and stiff clay that was too hard to penetrate by jetting. A Liebherr 932 excavator with a two pronged bucket attachment was used to excavate through a majority of the jet grout, however a milling road-header attachment was also used for about 20% of the SBPT by volume. Jet grout in the NBPT was encountered in November 2006 and is currently being mined (at the time of writing) with similar conditions to those found in the SBPT. In all the jet grouting program has provided a very competent soil stabilization mass, and has been mined with little incident and virtually no water influx to date.

CONCLUSION

The two methods of ground improvement, jet grouting and dewatering, combined with careful and systematic excavation and support methods in the Beacon Hill Station tunnels have proven to be very effective in limiting ground deformations and ground losses during station SEM excavation. Station SEM excavation has resulted in very small ground losses, of less than 0.1% of the excavation face volumes, corresponding to 0.8 to 1.6 inches of settlement, as measured in borehole extensometers with measurement rings located 1.8 to 3.6 m (5 to 15 ft) above tunnel crown. With the Station excavation over 80% complete, maximum surface settlements over the station are only 16.5 mm (0.65 in.). These surface settlements are a fraction of the over 100 mm (4 in) of surface settlement predicted using an assumed ground loss of 1%. Obviously the ground, both within and outside of the jet grouted and dewatered areas, has behaved better than anticipated, and resulted in much less ground loss than was predicted. However, much of the success in excavating the Station to date can be attributed to the proper and high quality field installation of these two ground improvement tools.

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