Construction of Underground Facilities for the Narragansett Bay Combined Sewer Overflow Program, Phase I

An ambitious project overcame a rocky start and used innovative practices to meet significant construction challenges.

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By any measure, the Narragansett Bay Commission's (NBC) Combined Sewer Overflow (CSO) Tunnel was a large, demanding project comprised of several risky elements, and included some considerable challenges, delays and disputes involving ground freezing, shaft construction, excavation and support of a large rock cavern, tunnel boring machine (TBM) mining and adit excavation by blasting.

Project Background

The quality of sewage treatment for ten communities of 350,000 people in the Providence metropolitan area of northern Rhode Island has markedly improved since the formation of the NBC in 1982. Each day, the NBC provides secondary treatment for 66 million gallons of sewage at two wastewater facilities; however, the addition of rainfall to the system results in overflows into the Upper Narragansett Bay an average of seventy-one times annually. This condition is being addressed by implementing the CSO Abatement Program, which has been in design and construction since 1992.

During the development of the conceptual and preliminary design, significant hurdles were experienced that led to approximately

three years of delay prior to construction. Originally, a comprehensive plan was developed in 1993 and approved by the Rhode Island Department of Environmental Management (RIDEM). A preliminary design was then developed in 1995 that consisted of three deep storage tunnels and seven underground storage facilities. The cost of the facilities was considerable and beyond the limits that many wanted to fund.

A formal stakeholder process was implemented to maximize due process and encourage the development of general consensus on a chosen plan of action. Therefore, between 1996 and 1998 multiple alternative methods and strategies were revisited, leading to the selection of an alternative that reduced the amount of tunneling and that did not require underground storage tanks. The final scope of the program consisted of two deep off-line storage tunnels, five near-surface interceptors, twelve sewer separation areas, a wetland facility and a wastewater treatment plant upgrade. The construction would stretch over twenty years, with three sequential phases to reduce the immediate increases to the costs of capital improvement. In 1999, RIDEM issued a "Finding of No Significant Interest" based on the environmental analysis for the three-phase program. Phase I design was initiated in 1998 and completed in 2001.

As summarized in Figure 1, the Phase I system intercepts existing sewers at nine diversion/relief structures and transfers flow underground via seven gate and screening structures and 5- to 9-foot-diameter drop shafts. At the bottom of these shafts, 8.75- to 15.75-foot-diameter de-aeration chambers in turn connect to 8-foot-diameter adits. The adits total 4,057 feet in length and carry flows to the 16,284-foot-long, 26-foot-diameter, 230-foot-deep main spine tunnel (MST). At the north end of the project area, the 26-foot-diameter foundry shaft provides ventilation/access, while in the south the S1 screening shaft sits adjacent to the new 50 million gallons per day (mgd) Fields Point Tunnel Pump Station (FPTPS). The pumps are housed in a 300-foot-deep cavern, with dimensions 117 by 61 by 68 feet. This cavern is connected to the surface by an 11-foot access shaft and a 32-foot utility shaft. Following each storm event, flow from the tunnel will be transferred to the existing Fields Point Wastewater Treatment Facility (FPWWTF) for secondary treatment and release.

The majority of underground construction work was packaged together in a single contract referred to as Contract 6, which included the MST, adits, two work shafts (S-1 and the foundry), one drop shaft (OF067) and the FPTPS cavern and shafts. (The work performed under this contract is the subject of this article.) In the interest of minimizing disturbance to the surface, construction of each drop shaft was packaged geographically with each of the remaining six near-surface construction sites (WRI, OF032, OF009, OF006/007, MRI and OF004/061), the details of which are described in Castro *et al.*[1] Lastly, a contract was packaged to fit out the FPTPS and install system-wide controls.

A total drilled footage of 32,800 feet was gathered from 146 soil and rock borings conducted in 1994 through 1995 and in 1999 through 2000. It was found that lower surficial conditions generally consist of a basal glacial till overlain by discontinuous strata of glaciofluvial and glaciolacustrine cobbles, sands and silts. Nearer the surface are estuarine, alluvial and urban fill deposits. The bedrock is comprised of a Devonian- to Pennsylvanian-aged sedimentary series that is weakly metamorphosed and deformed by folding. In addition to relatively competent conglomerate, sandstone, siltstone and shale with average unconfined compressive strengths (UCSs) of 10 to 13 kilo-pound per square inch (ksi) and a maximum UCS of 25 ksi, there lay intermittent deposits of weak graphitic shale with a UCS of less that 1 ksi.

Bedrock topography is extremely variable, with soil depths of 26 to 170 feet along the main spine alignment. The profile was selected so that the tunnel crown would remain two diameters below the lowest apparent top of rock elevation and a constant invert slope of 0.0012 percent could be achieved. Apart from regional variability, it was also documented that extreme localized irregularity exists both in the elevation and composition at the soil/rock interface. Groundwater levels are

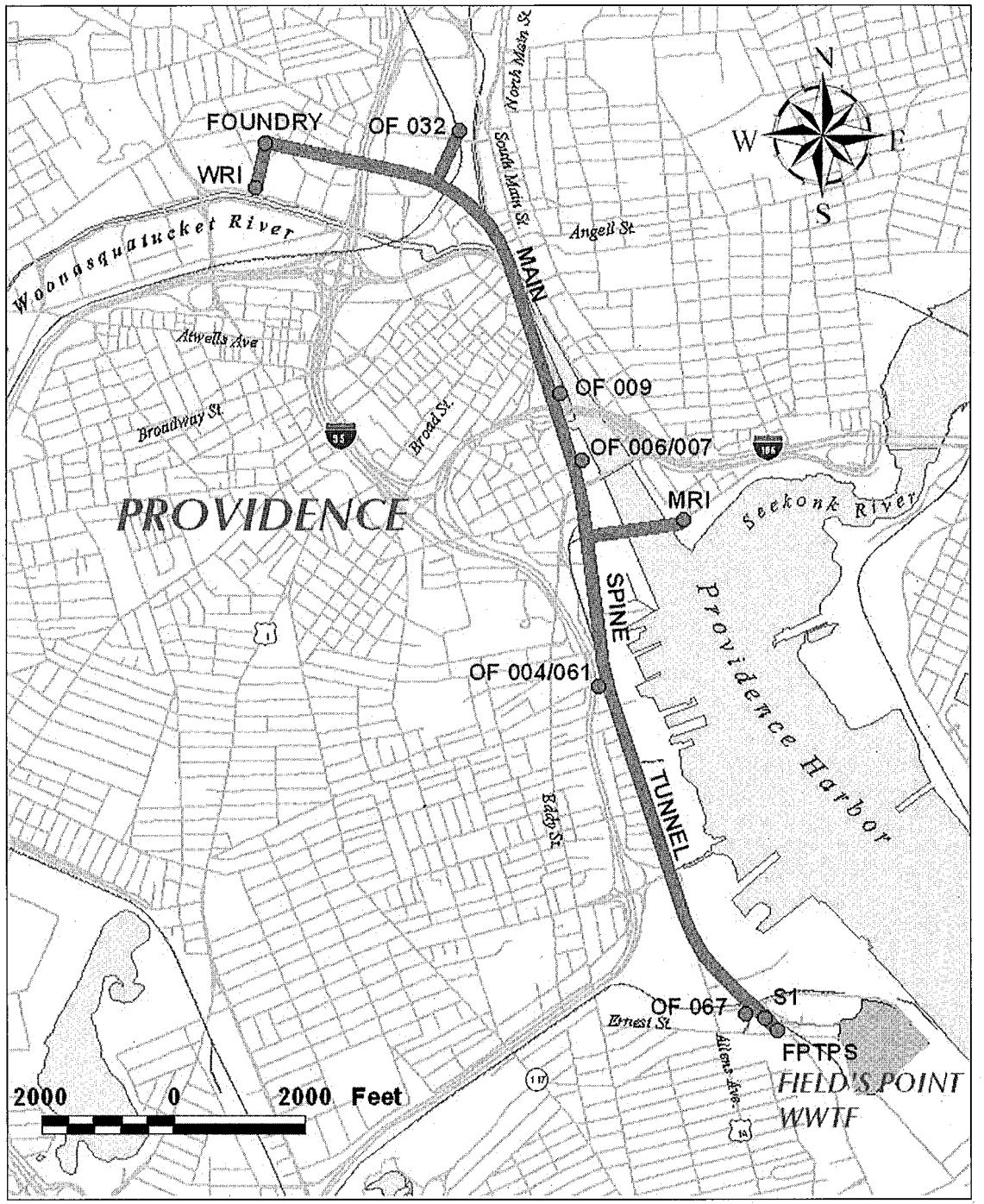

FIGURE 1. Project layout – MST alignment and shaft locations.

generally within 10 to 20 feet of the surface. Anticipated ground conditions were originally reported by Dill *et al.*[2]

The raw geotechnical data were summarized in data reports and ranges of soil and rock properties were categorically stated in a geotechnical baseline report (GBR). Three types of ground behavior were expected during tunneling: stable (65 percent) with fewer fractures and longer stand-up time, poor (25

percent) and very poor (10 percent). In the interest of simplicity and conservatism, the poorer ground was grouped together and designated Type II tunneling ground. For a two-pass tunneling operation, steel ribs and steel mats would be needed in Type II ground; whereas pattern rock-bolts and welded wire fabric were specified for Type I. The tunnel lining would consist of 18-inch-thick cast-in-place (CIP) concrete, reinforced with rebar in the Type II ground. The bid documents also included an option to use 15-inch-thick bolted precast segments if bidders wished to construct the tunnel using a one-pass method.

It was estimated that 3,300 gallons per minute (gpm) of groundwater ingress could be encountered during tunnel construction. Probing was specified to be conducted ahead of all excavations and substantial unit price quantities were included for pre-excavation grouting. Lastly, the possibility of encountering coal, and thus methane gas, was of sufficient concern that the contract documents required bidders to assume that construction would be performed in a gassy environment. This designation was later modified to "potentially gassy" upon receiving input from bidders during procurement.

Contract 6 was advertised in June 2001 and three bids were received on September 25, 2001: $163.5 million, $167.6 million and $227.4 million. The engineer's estimate was $175.0 million. The contract was awarded in December 2001 to the low bidder.

S-1 Shaft – Soil Excavation

Sinking large diameter shafts through 170 feet of unconsolidated, saturated soil constituted a significant challenge. The contractor had the option of supporting the soil by either slurry wall or freezewall. The contractor retained a subcontractor to install freezewalls at all six shafts in the contract. The method involved circulating –13 to –22°F refrigerated brine through a closed circuit system of pipes located within the ground, thereby freezing the water in the surrounding soil pores. Freezepipes were arranged in a ring around the shaft at 4-foot spacing and drilled to depths of 170 to 180 feet, terminating 10 feet into rock. At the S-1 shaft, the goal was to produce a 50-foot-diameter, 9-foot-thick frozen soil ring to serve as a hydraulic barrier and as a structural support surrounding the shaft during excavation.

Instrumentation to monitor the freeze consisted of observation and thermoprobe wells. Closure of the freeze ring would be verified and excavation of the shaft core could start once soil temperatures reached 30°F at a certain distance from the freezepipes and the water level inside the shaft had risen relative to the prevailing water table.

A 14-foot-deep collar (see Figure 2) at the S-1 shaft was constructed in May 2002, the subcontractor installed the freeze system and the freeze was initiated in late June 2002. The bottom of the collar was situated just above the groundwater table.

It was originally estimated that the freezewall would close in thirty-eight days from the start of the freeze and by fifty days the freezewall thickness would be sufficient for excavation to start. The actual duration for closure was 122 days and excavation started at 138 days. This delay led to considerable efforts to determine the cause and mitigate further delays.

Several types of monitoring techniques were employed to locate anomalous warm zones that, in turn, could shed light on unwanted "windows" of unfrozen soil in the developing freezewall. At first, it was discovered that the freezewall from the surface to a few feet below the groundwater table was not developing as planned. The contractors claimed that gasoline contamination was delaying the freeze by depressing the freezing point. Other theories on the cause of the delay were based on the premise that moving water put an extra heat load on the freeze system and delayed the formation of ice. Eventually, problems deep in the freezewall were also suspected.

In September 2002, bentonite and cement grout were pumped from inside the S-1 shaft in an attempt to close off suspected deep openings. Next, additional freezepipes were installed at shallow depths just below the collar from a ring exterior to the main vertical freezepipes. Neither effort resulted in closure.

Four pump tests were then performed within the shaft. The drawback of pumping is

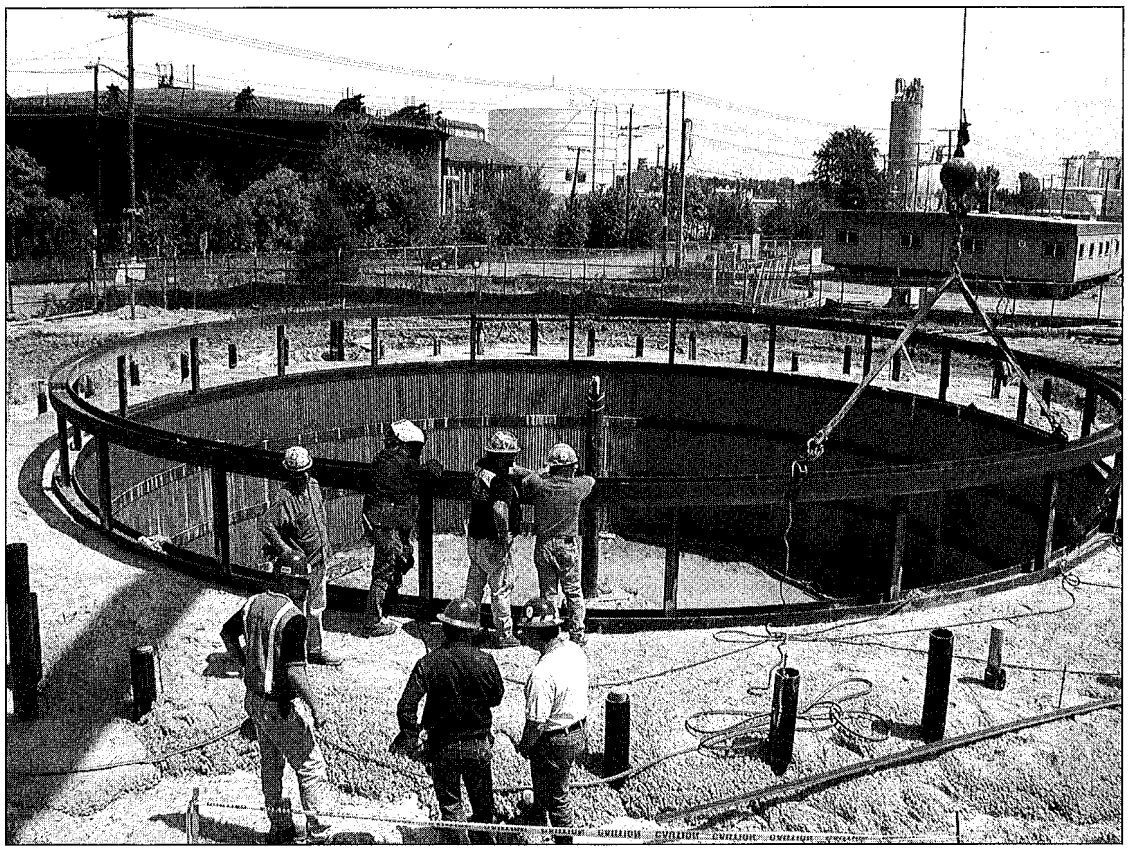

FIGURE 2. S-1 shaft collar construction in June 2002, with the freezepipe casings in the foreground prior to being plumbed and operated.

that the test itself may induce flow through a freezewall breach. However, the tests did provide valuable data that were used to locate windows in the freeze.

By mid-October, there was growing certainty that the top of the freeze had closed but two windows in the freezewall located 120 feet and 155 feet below the surface were preventing full closure. Additional observation wells and freezepipes were installed, and grouting was performed from outside the shaft using sodium silicate and cement/bentonite grout materials. Closure was verified in early November and excavation started by the end of the month.

Due to the extended freeze period, the freezewall grew considerably thicker than planned, which resulted in slower than expected excavation rates since the frozen material had to be chipped with a hoe ram (see Figure 3). Upon reaching the top of rock, a CIP concrete liner was placed using the slipform technique and the freeze was turned off.

The contractor submitted a claim that contended how three differing site conditions (DSCs) had interacted to cumulatively delay the freeze from the anticipated 38 days to 122 days. They contended that "free-phase" gasoline contamination delayed the freeze near the top of the shaft and was remedied by the installation of shallow freezepipes. This problem masked two deeper windows that were caused by higher than anticipated permeability and groundwater flow up from bedrock through pre-construction grout holes that were incompletely sealed by a previous NBC contractor.

In response, the owner's team investigated and concurred that the freeze had been delayed near the surface and later by two windows at depth. However, there was disagree-

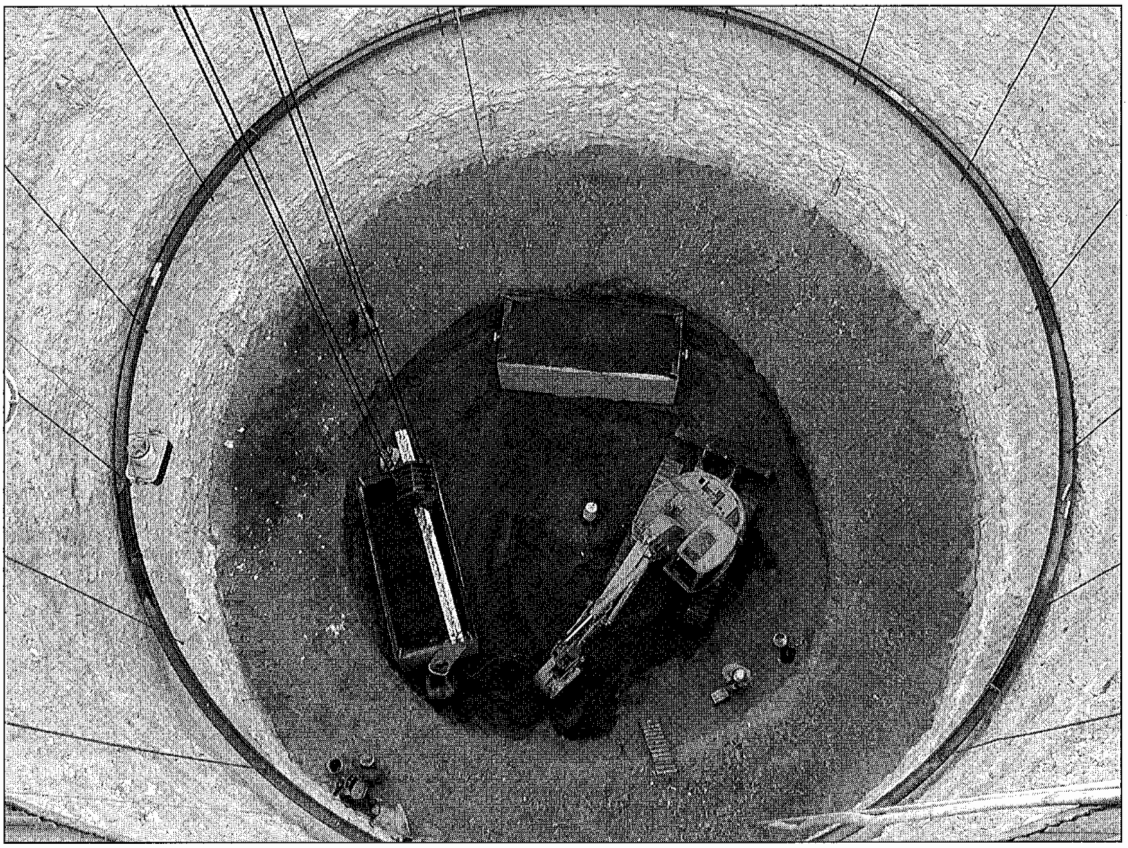

FIGURE 3. With depth, the width of the outer rim of frozen soil increased while the inner core of unfrozen soil decreased, slowing production.

ment with the causes asserted by the contractor. Direct evidence of free-phase gasoline was never discovered and soil contamination was found to be no worse than described in the bid documents. Testing showed that soil samples froze as expected. It was judged that warmer than expected groundwater likely delayed the freeze near the top of the shaft. The owner's team concurred that there likely was leakage up from rock into the shaft core, but found the flow could have come through bedrock fractures, the subcontractor's own observation well or the grout holes. Deposits of gravel were found that had a higher permeability than anticipated, but their continuity beyond the excavation could not be confirmed. Lastly, the gradient of the groundwater table was no steeper than described in the contract documents.

The contract also called for the contractor to perform an exploration at each site to determine groundwater flow and thermal properties sufficient to design a freeze system. The subcontractor took the position that the contract data were sufficient, and that currently available techniques for establishing groundwater flows were unreliable. In hindsight, it may be that more intensive exploration should have been performed by the NBC and/or the contractor, but this would still not have guaranteed the problems could have been avoided.

It should be apparent that sorting out responsibility for these delays was not an easy task. The contractor and subcontractor worked diligently to troubleshoot the problem and mitigate the delay. A discussion on this topic from the contractor's perspective can be found in Schmall *et al.*[3] The claim was further complicated by a concurrent delay associated with TBM delivery and fabrication. Ultimately, the claim was negotiated as part of a global

FIGURE 4. Sinking the S-1 shaft into rock below the concrete lining through soil.

settlement with the cost shared by the contractor and the NBC.

S-1 Shaft Rock Excavation

A proactive approach was taken with respect to blasting in an urban setting. During the design phase, a test blasting program had been performed to allay concerns from the owners of a very sensitive manufacturing facility located near to the site. Several meetings had also been held to advise and receive input from public safety officials. Immediately prior to shaft construction, an information package was distributed to surrounding neighborhoods and public meetings held to address the commonly held fears and misperceptions about blasting methods. The NBC paid for the city of Providence to retain an independent consultant to review the contractor's blasting plan and assist in writing the terms of the blasting permit. Despite these efforts, the project encountered a five-day delay in obtaining the blasting permits, which were ultimately obtained following agreement between the NBC and the city of Providence. No major incidents or substantive complaints occurred due to blasting operations throughout the project work.

Following the first blast in March 2003, the S-1 shaft was excavated to tunnel elevation as a 34-foot square centered within the 50-foot cylindrical soil liner, as depicted in Figure 4. From the bottom of the shaft, a 366-foot-long circular-shaped starter tunnel was driven north in two headings — an upper and a lower — starting in May 2003. A 180-foot-long,

10.5-foot-diameter horseshoe tail tunnel, in line with the MST, was excavated southward concurrently.

Foundry Shaft Construction

Events at the S-1 freeze prompted concern from the contractor and the NBC's project team regarding the feasibility of freezing at the other main shafts. The NBC proceeded with a program to recover continuously sampled borings using sonic drilling and install additional wells at the foundry shaft. The subsequent analysis confirmed that soil permeability and gradients were not sufficiently adverse to preclude freezing. The site investigation did show, at one boring location, that the top of rock was overlain by very coarse gravelly soils, raising concern that the absence of the impermeable basal till could complicate excavation at the soil/rock interface.

Potential difficulties at the soil/rock contact were further confirmed during the shaft pre-grouting. This program originally consisted of sixteen vertical holes drilled 276 feet deep with the goal of reducing the water-bearing characteristics of the bedrock. Significant water losses during drilling were noted at the soil/rock interface. The program was expanded to an additional sixteen grout holes targeting the upper 20 to 30 feet of bedrock where water losses were the greatest. Even with this additional grouting effort, further water loss was noted during subsequent freezepipe drilling.

It was agreed, through a change order, that freezepipes would be extended to 185 feet deep, or 25 feet below the top of rock, and additional groundwater monitoring would be performed. The freeze was completed by mid-November as planned. At that point, concern remained that a residual mass of unfrozen soil or rock still existed at the center of the shaft near the top of rock. This worry was an especially sensitive matter, considering the potential for rapid freezewall erosion if flowing water were to be induced during excavation. Probe holes were drilled once the excavation reached the top of rock to check for unfrozen water-bearing zones. Drill-and-blast excavation was used to sink the shaft 2 to 5 feet at a time to a depth where a completely frozen core well into rock was verified.

The top of bedrock was found to vary considerably between 145 and 162 feet deep within the confines of the excavated 30-foot-diameter shaft. The cause of the drill water losses was discovered as the shaft was sunk into rock. Fissures in the rock 0.5 to 2 inches thick were found to contain clean fine to medium sand as deep as 20 feet below the top of rock. The sand was clearly detrital in origin and undisturbed except for being frozen. In other locations, lenses of grout from the pre-grouting program were discovered sandwiched in the sandy fissures, but by no means had the grouting provided a complete sealing. Based on visual evidence alone, these seams would have had the capacity to permit large inflows into the shaft had they not been sealed off by freezing. The initial CIP concrete liner was taken down to an elevation to cover these sand seams in order to prevent inflow after the freeze was turned off. Even then, chemical grouting through the liner was required to cut off silt-laden inflows.

The remainder of the shaft sinking through rock proceeded uneventfully to 235 feet deep, a receiving chamber for the TBM was opened up at the base and the WRI adit was driven in.

Expanded Precast Segment Lining

On previous projects the contractor had successfully employed a two-pass excavation and lining system whereby expanded precast segments temporarily support the rock during tunnel mining and a permanent liner is placed thereafter. Shortly after the award of the contract, the contractor proposed that in lieu of the 18-inch CIP contract lining that would be placed within the contractor's 10-inch precast initial support for a total concrete thickness of 28 inches, a 22-inch-thick composite liner consisting of 10-inch-thick precast segments and 12-inch-thick CIP should be considered (see Figure 5). Ultimately, this value engineering cost proposal was accepted and the contract price reduced. The savings were realized predominantly from a reduction of nearly 28,000 cubic yards of concrete and a similar reduction in handling mined rock.

The 5,000 pounds per square inch (psi) UCS precast segments were fabricated by the contractor at a yard located adjacent to an existing

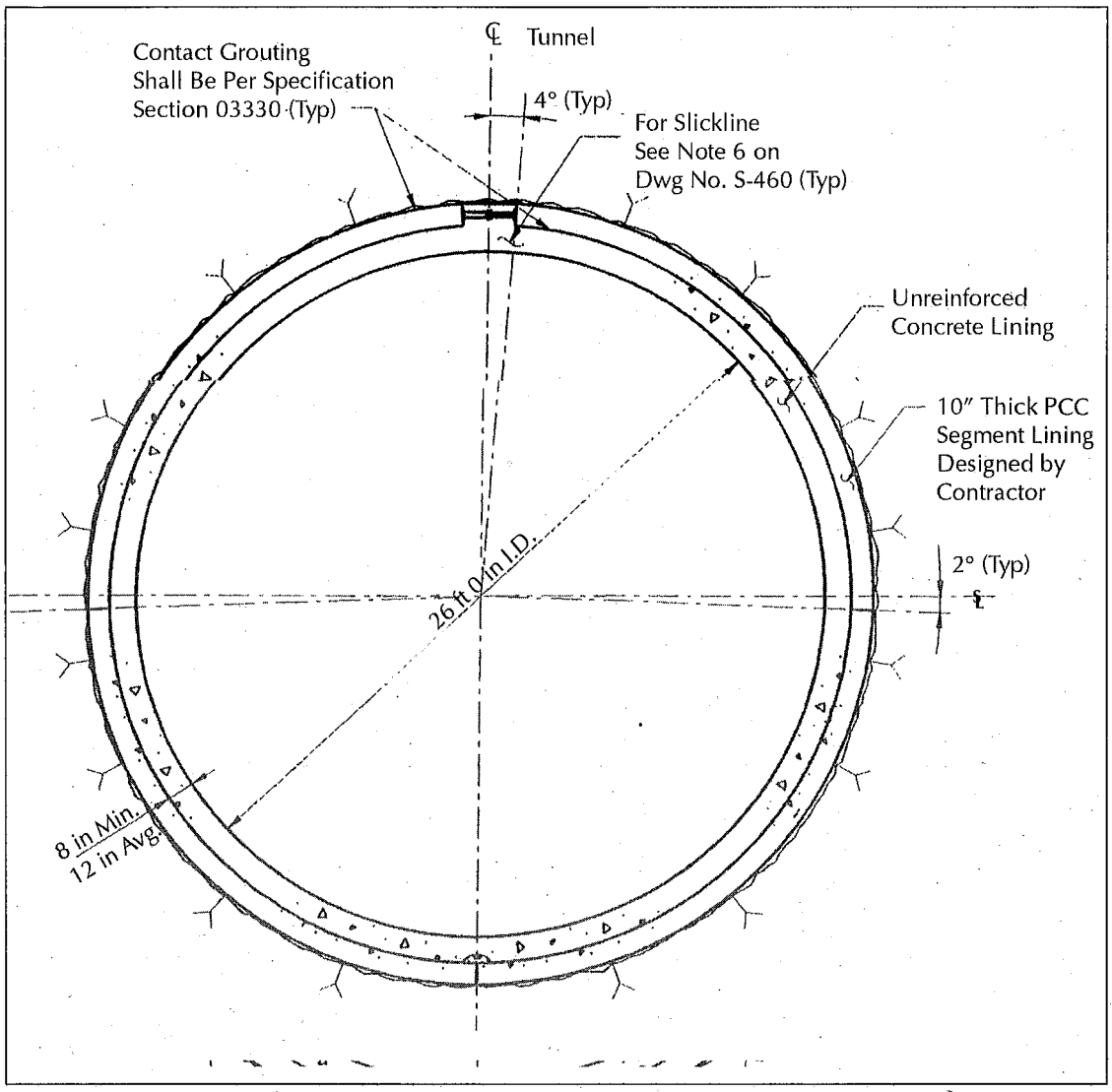

FIGURE 5. The composite MST lining system.

batch plant approximately 10 miles from the tunnel. The yard produced up to sixty-eight segments per day, starting nearly a year before mining commenced.

MST Excavation

The TBM was designed by the contractor in 2002. The cutterhead, main shield and tail shield were fabricated in Japan, arriving by ship in Providence in February 2003. The remaining electrical, hydraulic and mechanical components and the trailing gear were purchased individually or fabricated by the contractor and assembled at the S-1 shaft site (see Figure 6). Power for the thrust, head rotation, conveyors and segment erection was electric over hydraulic.

The cutterhead was 30 feet in diameter and was dressed with sixty-five 17-inch-diameter back-loading cutters. The machine employed twenty jacks to propel and steer off precast segments with a total available thrust of 6.3 million pounds. The cutterhead rotated at 3 to 5 revolutions per minute (rpm) and was driven by twenty hydraulic motors rated at a total of 2,465 horsepower.

The TBM allowed for lining installation within a full shield. The precast ring was com-

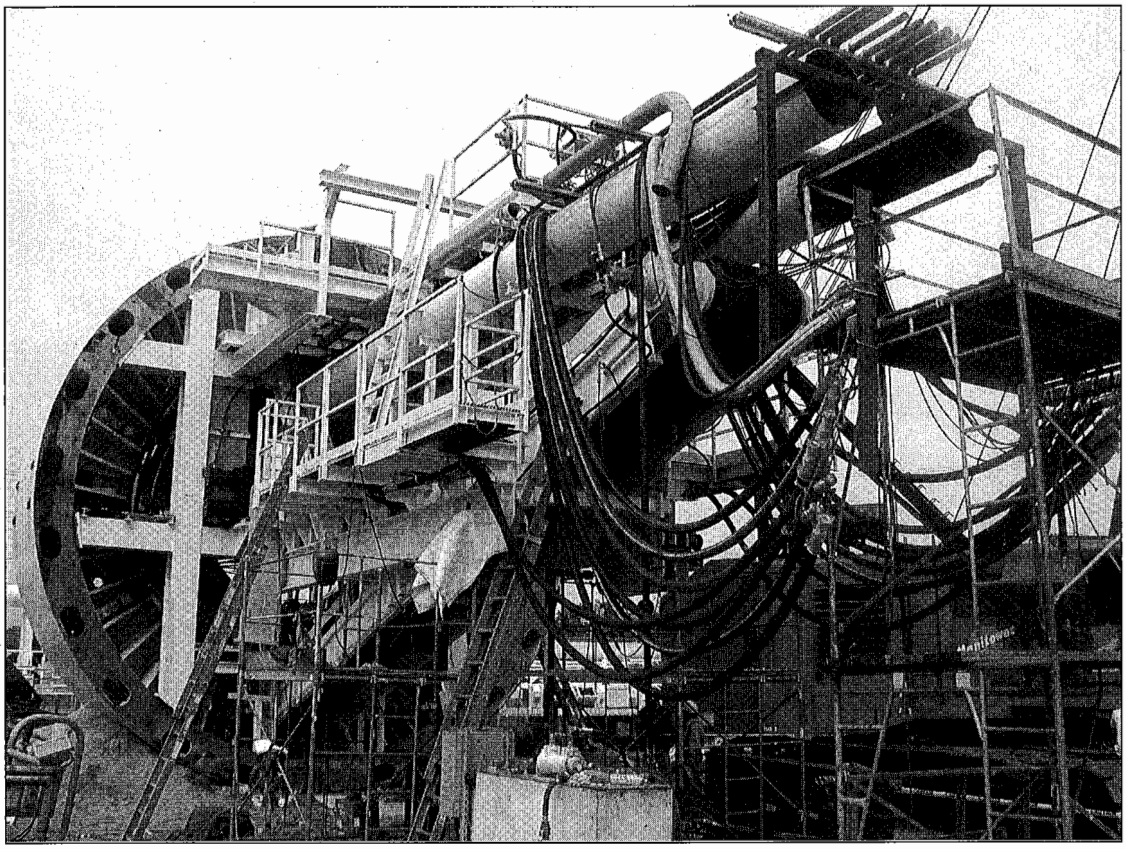

FIGURE 6. The TBM nearly fully assembled at the surface prior to disassembly and installation in the tunnel.

prised of four 48-inch-wide, 10-inch-thick precast segments each weighing 11,000 pounds. As the TBM advanced forward and the precast ring emerged from the tail shield, hydraulic jacks were engaged in a 2-foot-wide open keygap at the crown to expand the segments against the rock. Steel struts were then installed at the keygap to hold the segments in place.

The TBM and trailing gear were lowered and assembled underground beginning in October 2003. The focus then moved to the build-out of the mining support equipment in the shaft area (such as the main rail switch, a muck car roll-over dump system and a vertical belt conveyor). A radial stacking conveyor was installed at the surface. Other activities included the installation of a 4,000 gpm capacity dewatering system, the ventilation system and personnel hoist. The ventilation system consisted of two 125-horsepower fans at the S-1 shaft surface exhausting from the back end of the TBM trailing gear and through a 60-inch hardline mounted along the tunnel. Smaller fans and ductline were installed on the trailing gear to move air between the tunnel face and the rear of the trailing gear (see Figure 7).

Tunnel Mining

TBM mining of the MST commenced on March 8, 2004. Within a month, production was in the 40 to 45 feet per day range. Other than relatively minor issues with the mucking system and hydraulic power to the motors, the TBM and support systems operated smoothly.

A typical day on the TBM consisted of a mining shift from 6:30 A.M. to 6:00 P.M., a 6:00 P.M to 1:30 A.M swing shift to perform maintenance and a graveyard shift from 10:30 P.M to 6:00 A.M to change cutters and scrapers. The basic mining cycle consisted of mining

through 4 feet of rock in 25 to 40 minutes, separated by time to install segments and wait for trains. Haulage by rail was performed using three locomotives hauling five cars each — three muck cars and two segment cars (see Figure 8). To allow trains to pass side by side in the tunnel, a moveable "California" rail switch was employed at roughly the midpoint between S-1 shaft and the heading, which was relocated forward as the excavation advanced.

During design, bedrock lithology had been estimated in the GBR based on relative percentages of rock types logged in retrieved cores. The rock types that were encountered were largely as expected, as shown in Table 1.

The GBR also included a ground classification system established for tunnel construction. The classification system was based on a number of criteria including rock quality designation (RQD), unconfined compressive strength, rock type and anticipated tunneling conditions. The purpose was to provide an estimate of corresponding rock support type for a two-pass system where decisions had to be made as to the type of ground support to install. The contractor's precast segments made the selection of ground support unnecessary since the precast segments could support either ground type, essentially making the ground classification system moot.

However, the classification system did become a source for a DSC claim related to the TBM penetration rate. The contractor noted that in preparing its bid it had assumed that the TBM penetration rate would be 7.5 feet per hour in Type I ground and 14.8 feet per day in Type II ground. The contractor anticipated that the TBM would penetrate weaker, broken ground faster than less fractured, more competent ground, relying upon the ground classification quantities provided in the GBR for its estimate. Considering the information provided in the baseline report, this approach was not an unreasonable one to estimate penetration although certainly not the one intended by the authors of the baseline report. Considerably more Type I ground was encountered than the 65 percent predicted by the GBR. The claim was eventually settled as part of a global settlement. For future GBRs, it is recommended that baseline statements of ground support be clearly distinguished from those ground properties used to predict TBM performance, otherwise conservatism in the estimation of support could result in an overly aggressive penetration prediction.

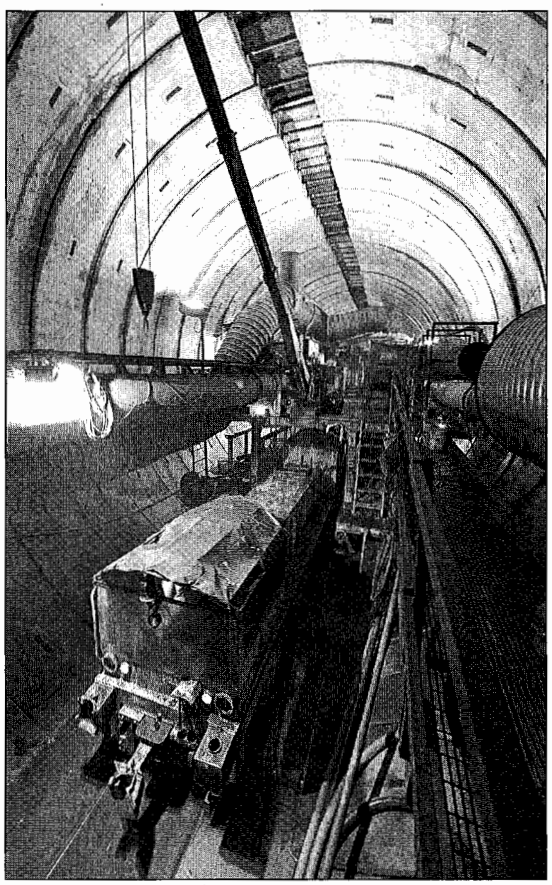

FIGURE 7. A view of the locomotive at the rear of the trailing gear. Note the ventilation intake on the left side and exhaust from the TBM face into the tunnel hard line at right.

Apart from the slower than anticipated advance rate, the contractor's mining operation was performed exceptionally well. The contractor's approach was atypical in that the contractor designed and assembled the mining equipment on its own, the general trend being for tunnel contractors to rely upon specialist suppliers. The TBM and support systems were operated at very high rates of availability with minimal unanticipated downtime and hole-through was achieved in December 2005. Mining production is displayed in

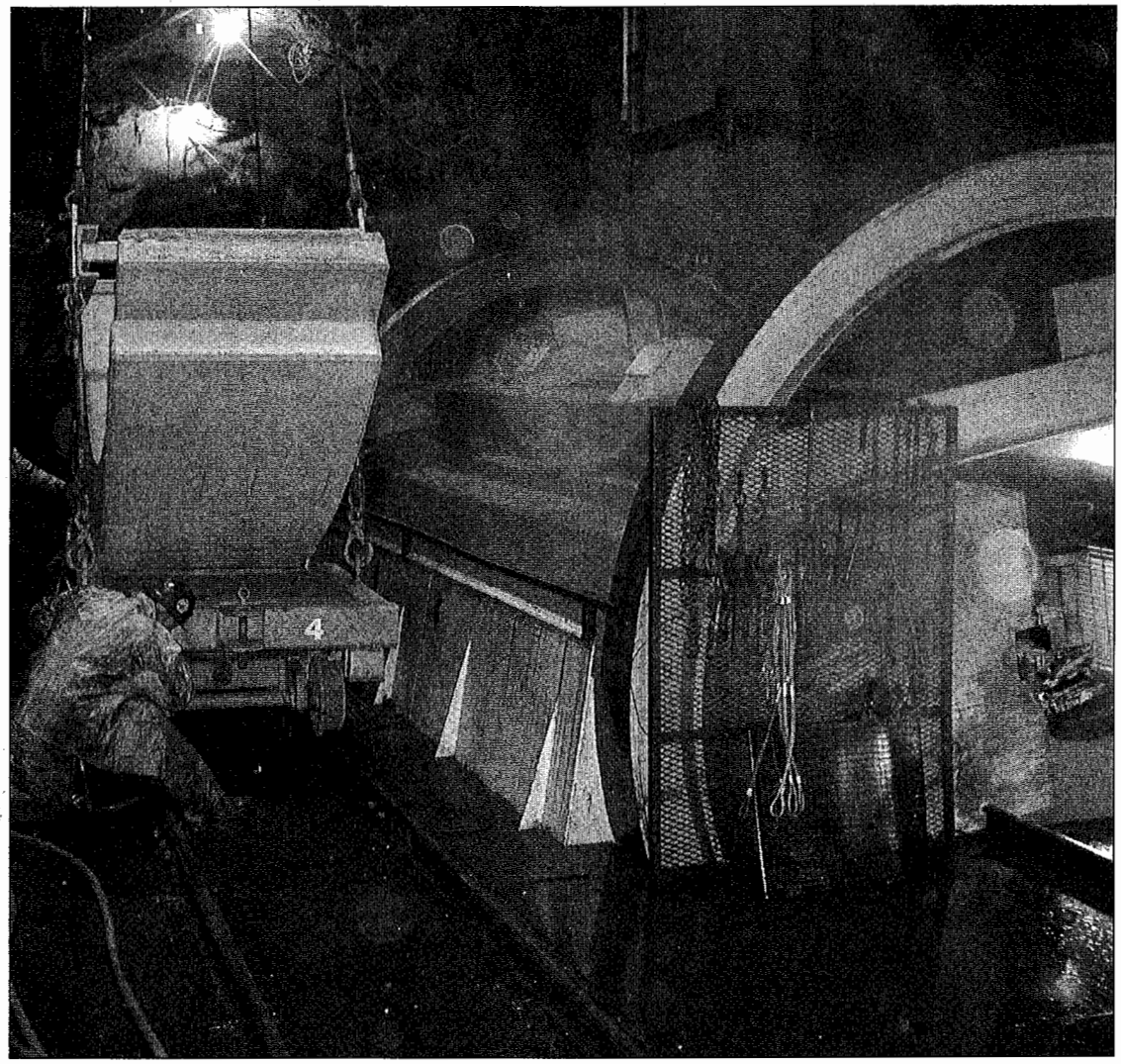

FIGURE 8. The bottom of the S-1 shaft was a busy place during TBM mining. Segment cars were lowered by crane to the track as seen at the left, while muck cars spilled their load onto a rollover dump that fed a vertical conveyor on the right.

Figure 9; the mining crew at hole-through are shown in Figure 10. Figure 9 also shows the impact of pre-excavation grouting on mining production.

Pre-Excavation Grouting

The GBR projected that a steady-state groundwater ingress of 3,300 gpm would be expected from the mined tunnel and adits. Invert water flowed downslope away from the face to the S-1 shaft where it was pumped from a dewatering station to a settlement and treatment system at the surface. Diverting this treated flow to NBC's FPWWTF avoided the need to obtain a project-specific discharge permit. It was predicted that there would be long stretches of the tunnel that would produce little water, with intervening geologic structures that could be significant water bearers. Contract specifications called for 100- to 170-foot-long horizontal probe holes to be drilled from the TBM ahead of the excavated face. If water flows from the probes exceeded 50 gpm, then these holes as well as additionally drilled holes would be grouted with Type III Portland or microfine cement mixes. Grouting was per-

formed through holes drilled through the slots in the cutterhead with a grout plant set-up situated in the tunnel behind the trailing gear. The TBM would then advance within the envelope of grouted ground to approximately 30 feet from the end of the grouted zone, at which point the next probe hole would be drilled.

This pre-excavation grouting program went through several stages of modification in response to the conditions that were encountered. First, water inflow into the tunnel was encountered at flow rates greater than the predicted average. By the time 25 percent of the tunnel had been mined, nearly 1,200 gpm (or 36 percent) of the total anticipated inflow had been encountered. Yet, probe holes were yielding less than 15 gpm, well below the grouting trigger. Second, grout takes at the 500 psi grout pressure being used were quite high, often 5,000 to 6,000 cubic feet. It was suspect-

TABLE 1.
A Comparison of Rock Types Encountered in the MST

Rock Type	Actual Encountered* (%)	GBR Encountered (%)
Shale	23	25
Sandstone	56	48
Siltstone	16	14
Graphitic Shale	5	9
Conglomerate	1	4
Coal	0	<1

Note: *Based on rock samples collected from keygap, conveyor, muck pile and core samples from the tunnel invert.

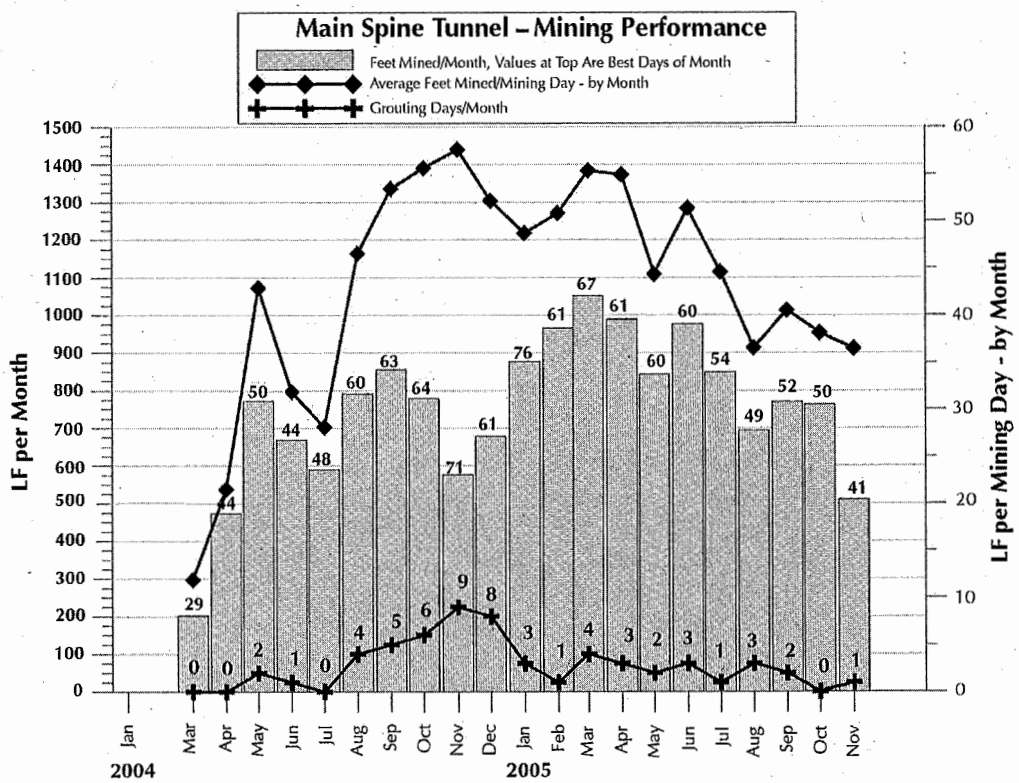

FIGURE 9. TBM advance rates.

FIGURE 10. TBM hole-through at the foundry shaft.

ed that at this pressure grout was being pushed through fractures far outside the tunnel envelope. Third, it was believed that two grout holes were insufficient to provide adequate grout coverage over the tunnel envelope.

The program was modified after several rounds of changes. The maximum grouting pressure was reduced to 350 psi. The grouting trigger was decreased to 15 gpm and the starting mix was changed to less expensive Type III cement when the probe inflow exceeded 50 gpm. Grouting was performed through four holes rather than just two.

Also at this point, both parties agreed that negative impacts to the schedule could be mitigated by reducing the duration of grouting events from two mining days to one. To enable quicker mobilization of grout equipment and faster grouting, the NBC paid for several equipment upgrades (such as a new rolling gantry to boost the air compressor capacity and a second mixing and grouting plant). The goal was to drill and grout multiple holes simultaneously. Furthermore, the pumping capacity was increased from 4,000 to 5,700 gpm in order to boost the margin of safety for dewatering.

The final inflow at the end of mining peaked at 2,500 gpm. There were a total of forty-four grouting events in which 4,400 cubic yards of grout was pumped over fifty-nine working days. The inflow increased rapidly over the first third of the tunnel. In the second two-thirds of the tunnel, the rate of increase of inflow diminished. The reduced grout takes at lower grouting pressures appeared to be no less effective at controlling water into the excavation than the higher pressure grouting. The equipment upgrade allowed grout events to be accomplished in one day versus two days in which no mining could be performed, thus reducing costs and aiding the schedule. Rather than a simple

TABLE 2.
Pre-Excavation Grouting Performance Summary for the MST (Cumulative Values)

	Tunnel Length (ft)	Tunnel Inflow (gpm)	Probe Inflow (gpm)	Ratio of Tunnel Inflow to Probe Inflow
Grouted Intervals	7,957	1,201	3,261	0.37
Un-grouted Intervals	7,465	1,436	305	4.71

reliance on empirical rules and sticking rigidly to the original plan, the MST grouting program succeeded by modifying the plan along the way based on frequent monitoring and feedback.

The MST grouting program cost $10 million (versus $5.8 million in the bid estimate) and effectively kept inflow within manageable limits. Without it, the inflow would have threatened the ability to successfully place the CIP liner and would have impacted the NBC's treatment plant operation. There was widespread agreement among project participants that using a less aggressive pre-excavation grouting program and trying to tackle heavy inflows with cut-off grouting would have been less effective and more costly.

The effectiveness of the grouting program can be estimated in gross terms by comparing water inflow in the grouted zones versus the ungrouted zones, as shown in Table 2. This table also shows the importance of probing because in nearly half the tunnel there was no grouting triggered at all.

Adit Construction

Seven adits were excavated off the MST to connect the tunnel to the previously installed drop and vent shafts installed by other NBC contractors. All adits were excavated and lined concurrently with the excavation and lining of the MST except for the 067 and WRI adits, which were driven from the base of the main shafts. A typical intersection of adit de-aeration chamber and vent shaft is shown in Figure 11.

The 067 adit was driven concurrently with the excavation of the starter tunnel since the adit portal was within 50 feet of the S-1 shaft. The adit at the far north end, the WRI adit (see Figure 12), was driven from the base of the foundry shaft since it too was within 50 feet of that shaft.

The 004/061 adit was the first one driven off the MST, commencing in February 2005, nearly a year after the start of TBM mining. Temporary work decks were installed in the MST at the adit portals so that adit operations would not block the passage of trains to the TBM heading (see Figure 13). Due largely to the demonstrated success of shaft blasting, both city and state authorities permitted 24-hour blasting within the tunnel. Blasting was performed on the second and third shifts so as to not disrupt the operation of the TBM. Materials were delivered to the adit deck by rail during the third shift. Load-haul-dump (LHD) vehicles were used to load and transport muck from the face. In the 1,800-foot-long MRI adit, a widened turn-out station was excavated at the mid-point to allow LHD vehicles to pass. The excavation rate for the adits averaged approximately 13.6 feet per workday (three shifts).

Vibration and noise monitoring was performed by the owner's representatives and measured against Rhode Island's legal limits; no complaints or exceedences occurred. Considerable outreach was performed to adjacent neighbors, but the task was simplified by the lack of residences in the project area. In addition, nearby businesses generally did not operate at night.

FIGURE 11. Intersection of the adit de-aeration chamber and the vent shaft in the 004/061 adit, with the MST visible in the background.

One potential problem did arise when authorities at a neighboring hospital expressed consternation regarding blasting vibration. Their sensitivity regarding this issue had been heightened the previous year when transmission relays located above the adit had tripped for reasons unrelated to construction, causing a loss of power to the hospital. Additional monitoring instruments were installed and blasting started with reduced charge weights per delay, enabling excavation to proceed with minimal delay.

Concrete placement in the adits was performed during the day, pumped from the surface of the drop shaft using ready-mix trucks and a high-capacity pump. A de-aeration chamber is shown in Figure 14. Following concrete placement, contact grouting was performed by injecting cement grout through holes drilled in the crown of the adits and chambers at 10-foot intervals using a mobile grout plant located at the top of the drop shafts. All of the adits were completed by February 2006.

One lesson learned from the adit work is related to the use of embedded cylinder pipe (ECP) to line four pairs of the drop and vent shafts. ECP consists of a thin steel cylinder inside a layer of concrete reinforced with prestressed wire and an interior lined with mortar. In October 2005, as the contractor was completing the concrete work for the connection between the de-aeration chamber and the drop shaft in the WRI adit, the bottom 20 feet of the shaft lining failed. It was determined that water pressure had leaked into the inner layers of the pipe, causing it to burst inward under the full

head of water pressure. The section was replaced with shotcrete and wire mesh.

Of the other ECP shaft locations, two had already been completed and were undamaged. At the 032 adit, reduced blast rounds and hand mining were performed as the chamber excavation approached the shafts. In hindsight, a steel pipe transition section for the bottom 10 feet of the shaft would have been a more robust lining material at the shaft/chamber intersection where ECP was used.

Water inflow was minimal in all of the adits and the MRI adit was also the only one to need pre-excavation grouting. While direct measurements were difficult to acquire, the highest flow recorded was from the MRI adit, which produced an estimated 200 gpm This flow was reduced to about 15 gpm following lining and grouting. There were no differing site conditions in any of the adits.

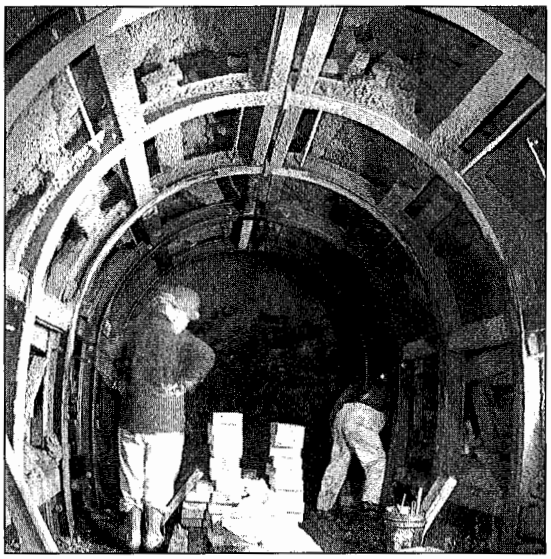

FIGURE 12. The installation of ribs and lagging in the WRI adit.

MST CIP Final Lining

Following TBM hole-through, the MST was lined starting at the foundry shaft. The contractor erected an on-site batch plant at the S-1 shaft site. A fleet of eight ready-mix trucks were used to deliver the concrete to pump sites at the foundry shaft, 009 and 004/061 adits, and S-1 shaft as the lining advanced. A slickline ran from the concrete pump located near the shaft collars to the forms (see Figure 15), with pipe added each day as the forms were advanced. The maximum pumping distance was 3,300 feet.

Panning materials were installed ahead of the concrete placement area to keep the freshly placed concrete separated from the 2,500 gpm of inflowing groundwater. A main drain was formed by placing corrugated steel sheets over a channel cast into the precast segment invert. One-foot-wide strips of thin steel sheets were tacked along the vertical joints of the precast segments, forming channels to transfer flow down to the drain. Sandbag dams were used to isolate the forms and water was diverted by pumping.

After the initial learning curve, production achieved 165 feet of advance per day. This rate required a sustained delivery of 580 cubic yards of concrete over eight hours. A 4,000 psi concrete design mix with 0.375-inch aggregate was used.

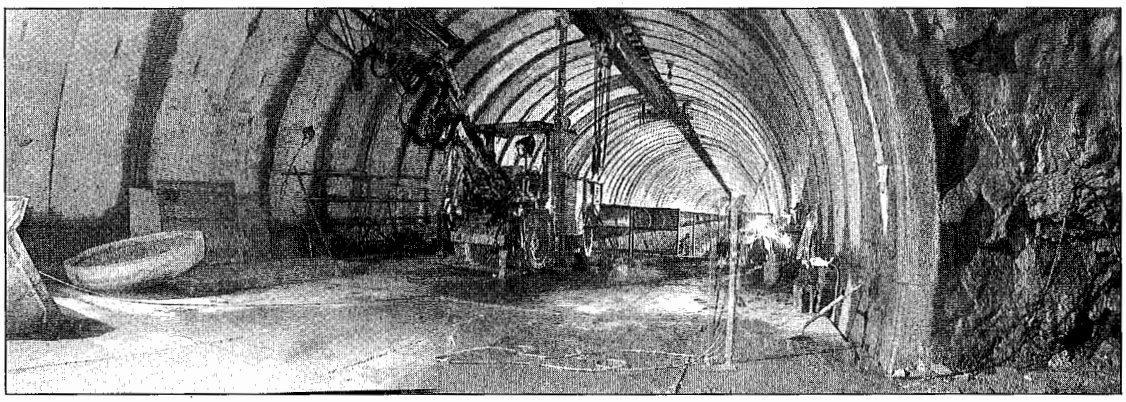

FIGURE 13. Typical work deck installation, with the adit portal visible at right.

FIGURE 14. Concrete work in a de-aeration chamber, looking out toward the MST. Adit forms are partially visible in the background.

Additives included new-generation full-range and high-range water reducing admixtures, retarder, fly ash and an air-entraining admixture. A total of 58,000 cubic yards of CIP concrete was placed in the MST for the final lining.

A typical placement day involved hauling, pumping and placing from 7:00 A.M. to 3:00 P.M. Preparation for the next pour, stripping of the bulkhead and movement of the forms occurred over the back two shifts. A single set of 35-foot-long invert forms was always left at the rear end of the pour, with 165 feet of forms leapfrogged past for the next pour (see Figure 16). At the downstream end of each form set-up, a vertical bulkhead enabled the concrete to be placed vertically to within a few feet of the crown. Above this, the end of each day's concrete placement would slope back for about 25 to 40 feet until it met the crown. The next day's concrete was placed against this geometry, creating a construction joint. The final lining was completed in September 2006.

Contact grouting was performed from a mobile grout mixing plant situated at the surface of the shafts. A 1:1 cement grout was pumped down the shafts through slicklines, then through packers installed in drill holes at the invert and crown. Grouting started at the foundry shaft end of the tunnel, and the grout plant was relocated southward to shaft sites as grouting progressed down the tunnel.

In September 2006, a block of concrete about 2 by 4 feet was discovered to have fallen onto the MST invert. The block had fallen out along the face of a construction joint. Distress was noted at a number of other locations, all confined to the sloping construction joint. The investigation revealed that concrete in the crown area was displacing away from the precast segments along the sloping joints, possibly as a result of increasing pressure from the recovering groundwater table. A repair plan was developed that involved removing a 2-foot-wide slot of CIP concrete along the con-

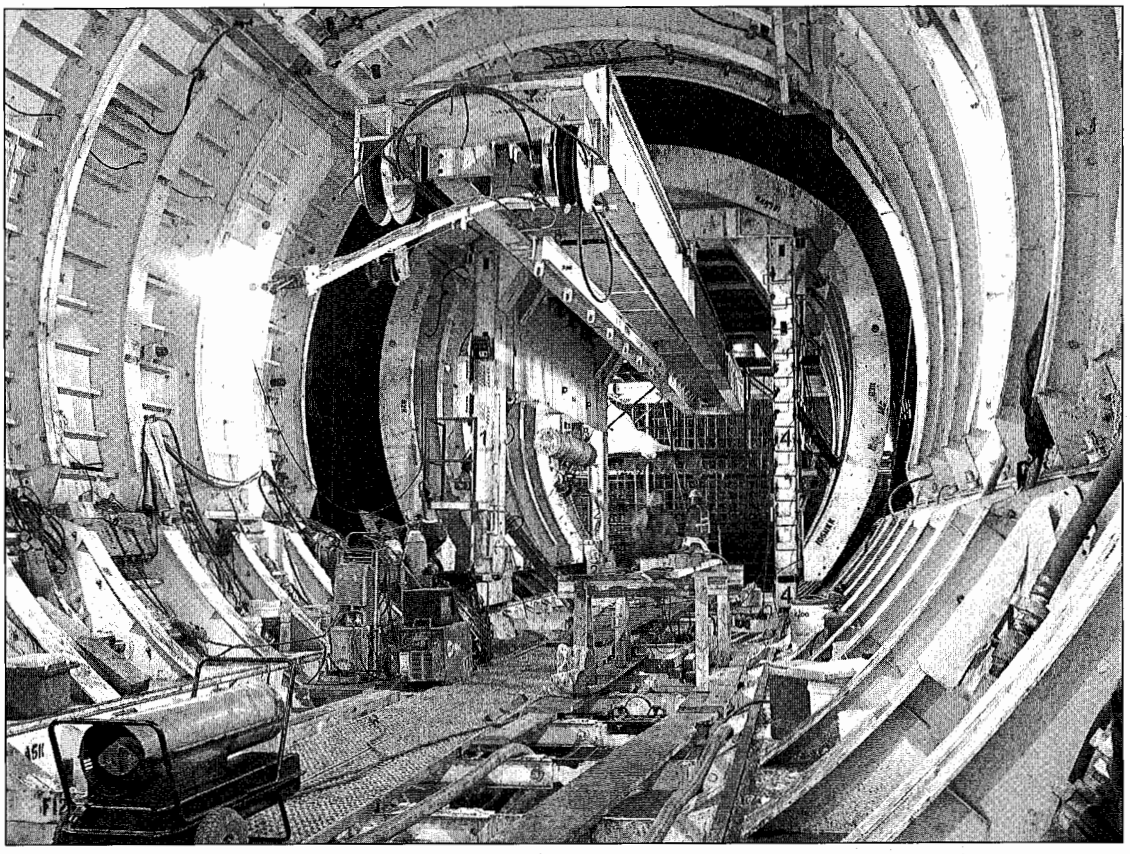

FIGURE 15. Forms and form carrier assembled in the MST just downstream from the foundry shaft.

struction joint using hydro-cutting until the underlying segment was fully exposed. Steel dowels were installed in the slot and embedded in the precast segments and adjacent CIP liner as reinforcement. This slot was then backfilled with shotcrete and trowelled to a smooth finished surface. Repairs were completed by February 2007 without impact to the project schedule. Costs to perform this work were split between the NBC and the contractor. An inspection performed a year after the work was completed revealed that the repairs were successful. Sloping cold joints in large-diameter CIP liners should be avoided and full bulkhead pours should be a requirement.

Following tunnel lining, both the foundry and S-1 shafts were lined with CIP concrete using the slipform technique. The 067 adit drop and vent shafts were excavated using ground-freezing and raise boring, then lined with CIP concrete by slipforming. Inflow as measured at the base of the S-1 shaft at the completion of the underground structures was approximately 700 gpm, but had attenuated to 250 gpm one year later, a dramatic reduction from the 2,500 gpm measured at the end of TBM mining.

FPTPS Cavern Construction

Shortly after notice to proceed in early 2002, the contractor communicated to the NBC its lack of comfort in building the FPTPS cavern as designed with regard to rock stability and support (see Figure 17). The contractor requested additional exploration of rock conditions, further modeling and analyses, and the involvement of a third-party rock mechanics expert to review the design.

The NBC agreed to perform additional investigations. The schedule allowed time for this work without negatively impacting the critical path. Two additional borings were drilled and downhole geophysics were con-

FIGURE 16. Looking downstream at the finished concrete lining and the forms in the MST.

ducted within the footprint of the cavern to obtain more data on rock conditions in the roof. Next, the NBC hired a third-party reviewer who concluded that rock conditions could be expected to be predominantly as described by the contract, but recommended the following modifications:

- The addition of mechanical end anchorages on roof dowels to provide immediate support;
- The addition of dowels at the utility shaft/cavern intersection as spiling (installed prior to excavating adjacent openings); and,
- The addition of dowels angled out from the intersection of the arch roof and walls (haunch).

Further discussion on the design of the pump cavern is contained in Hughes *et al.*[4]

Work on the shafts to reach the cavern started in 2002. The procedure for shaft construction for the utility and access shafts was similar to the S-1 shaft procedure. Following pre-excavation grouting from the surface, freeze-pipes were installed, a collar to just above the water table was installed, the freeze was completed without delays, a slip-lined CIP liner was placed and rock excavated to the cavern level by the drill-and-blast method.

Two notable issues were encountered during the excavation of the utility shaft. The first was the occurrence of a highly uneven contact between soil and rock, with nearly 10 feet of relief across the 35-foot shaft footprint. In order to create an even surface from which to start the overlying slip-formed liner, the deep soil pocket was excavated and backfilled with concrete. A second issue was the presence of soil-filled lenses and seams encountered within bedrock below the soil-rock contact. Remedial work consisted of over-excavating the areas where soil was discovered, installing weep pipes, then shotcreting over wire mesh held to the rock with dowels and following up with chemical

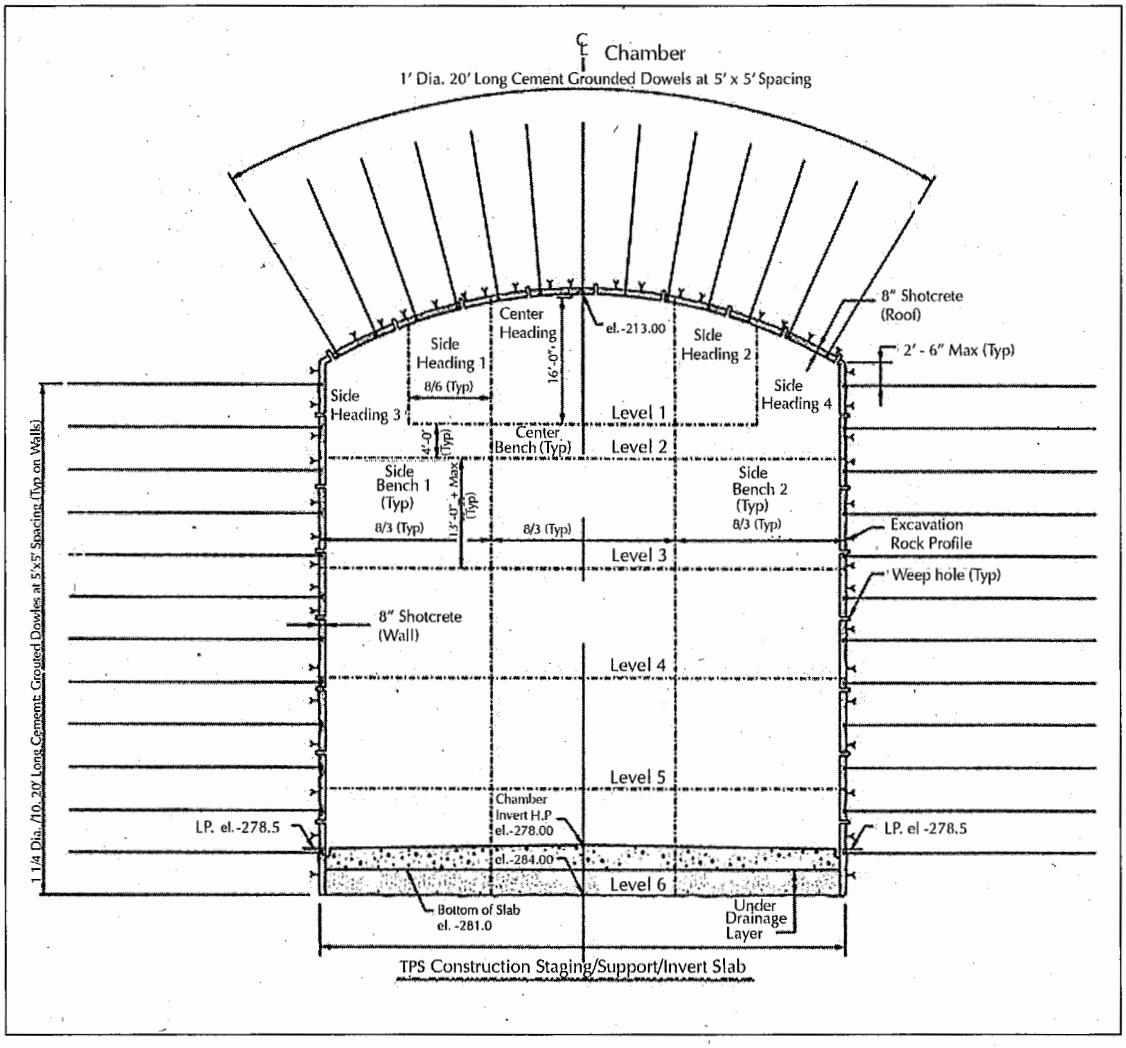

FIGURE 17. The FPTPS design — lining and excavation sequence.

grouting where inflows were detected. The material filling the rock fractures and making water was a fine, clean detrital sand.

FPTPS cavern construction commenced in October 2003 with the bell-out of the utility shaft into the top heading of the center drive of the cavern. The contractor designed a transition from a circular section to a rectangle to square up with the cavern geometry. The cheeks were shot out so that by the time the horizontal drive of the cavern top bench started in December 2003, the full width and endwall of the cavern were opened to a level 24 feet below the cavern crown.

A probe hole drilling and pre-excavation grouting program was undertaken from the utility shaft end of the cavern. Holes were fanned out along the cavern crown, extending to the mid-point of the cavern. Approximately 2,500 gallons of ultrafine cement grout were pumped into five holes. A second phase of probing in four holes was undertaken near the mid-point of the cavern to the access shaft endwall that did not trigger grouting.

The bedrock geology of the cavern consisted of sloping layers, from top down, of conglomerate, sandstone, graphitic shale and shale (see Figure 18). Actual conditions encountered were reasonably close to those that were predicted. The sharply dipping contact between the conglomerate and sandstone at the utility shaft/cavern intersection proved

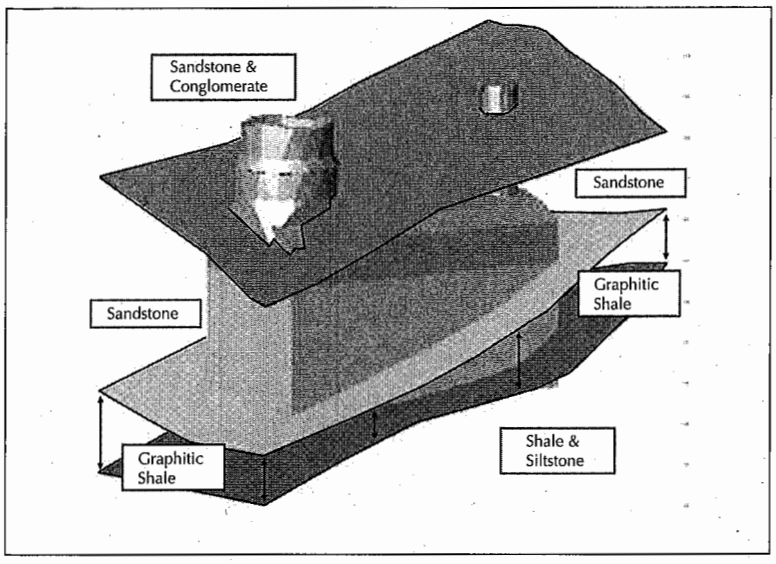

FIGURE 18. A three-dimensional view of the pump cavern with general geologic rock units predicted by site investigation.

troublesome with respect to ground behavior and water inflow. Sandstone blocks fell out below the contact and additional initial support bolting was required to prevent further loosening. Secondly the 30-foot permanent wall dowel holes drilled through this contact produced up to 100 gpm of water inflow from two holes. It was decided initially to leave these holes to drain the formation rather than grout the flow off and potentially fight inflows as successive dowel holes were drilled. In general, this approach worked since inflow from these two holes remained high with light or no inflows from following holes.

The top heading was driven horizontally at a height of 24 feet at the center of the roof arch and 15 feet at the sides. This height was necessary to install the 20-foot-long roof bolts. The heading progressed forward in roughly 8-foot-deep, 19- to 25-foot-wide drifts, so that the width of the cavern was advanced three drifts wide. The center drift was advanced several rounds before the side drifts were pushed forward. A two-boom drill jumbo was used to drill the heading blast holes and holes for the roof and wall dowels.

The 20-foot-long galvanized roof dowels were 1-inch-diameter, all-thread bar and installed on a 5- by 5-foot pattern with a spin lock anchor assembly used for anchorage at the end of the hole. Once the collar was set and hardware installed and torqued, a 9,000 psi UCS thick cement grout mix was pumped into a short grout tube until return was observed from a vent line that reached the end of the hole. The grout mix included an additive to provide shrinkage control, improve bonding and improve flowability characteristics. In the walls, 30-foot-long galvanized 1.25-inch-diameter dowels were angled downward at 3 degrees on a 5- by 5-foot spacing and installed similar to the roof dowels except that mechanical anchors were unnecessary and that the grout tube was run to the end of the dowel. A 5,000 psi wet-mix shotcrete was applied to a minimum thickness of 8 inches over the walls and roof. Synthetic fibers at a dosage rate of 17 pounds per cubic yard were approved as a substitute for the specified steel fibers.

Water inflow created difficulties grouting both the roof and wall bolts. As little as 0.5 gpm of water inflow from the bolt hole could wash through the freshly placed grout. Partial encapsulation was unacceptable considering the need for a high degree of corrosion protection. Several techniques were developed to completely cut off the water. Where the inflow exceeded 5 gpm from a single hole, it might be abandoned and a new one drilled adjacent to the water maker. The dowel would be installed in the hole making the least amount of water, the other one cut-off grouted with a microfine cement mix after the dowel was successfully grouted. Where water flowed from the grout tube or from around the plate following set-up of the grout, the tube would be re-grouted using a microfine cement mix. Dowels were not accepted until all flow was cut off. Where efforts to control water in the dowel holes was exceptional, the parties developed a means to change the work and pay items without dispute.

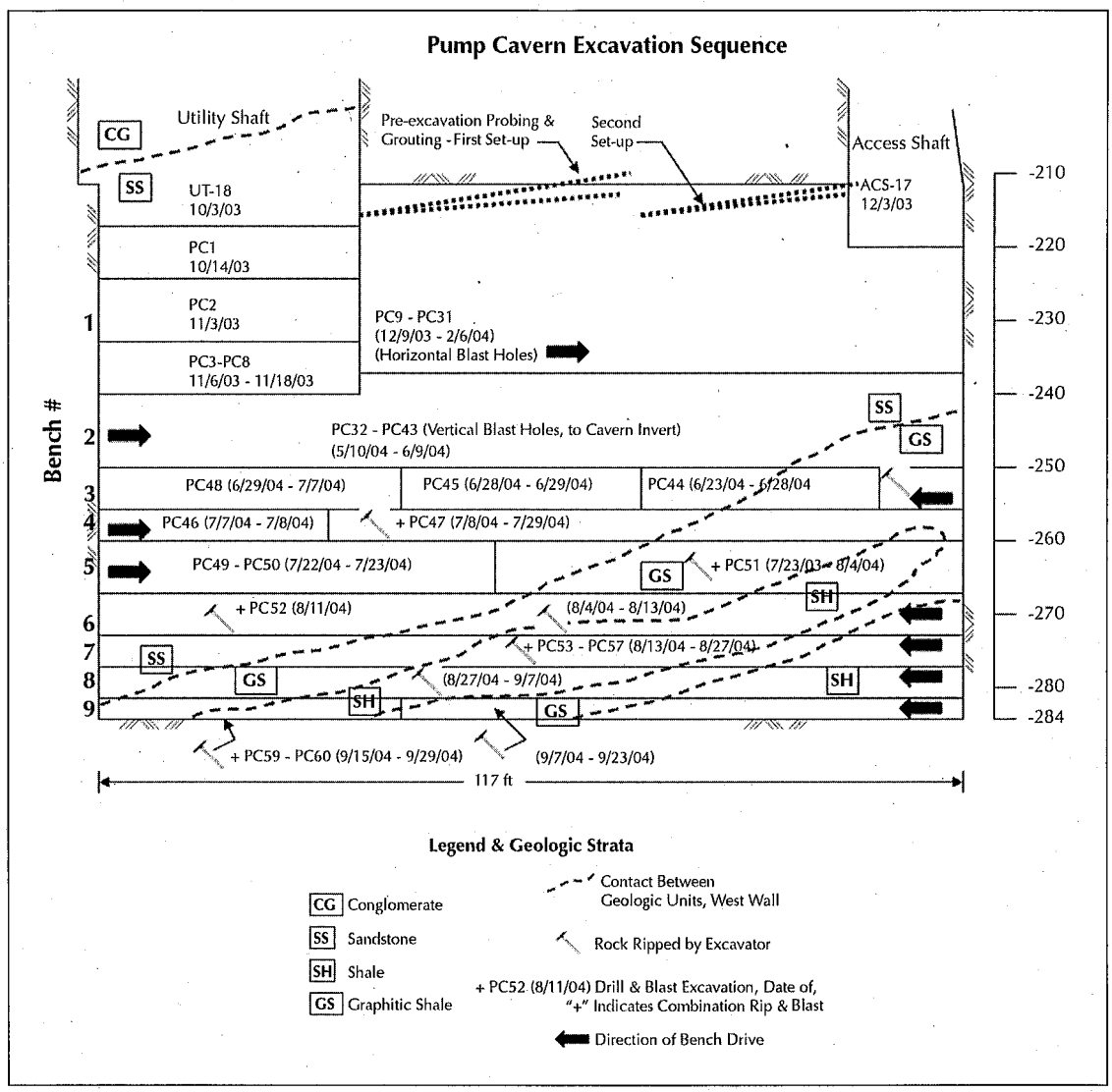

FIGURE 19. The FPTPS cavern excavation sequence.

Four multiple point borehole extensometers (MPBXs) were installed 50 feet deep into the roof to monitor rock movement. With a threshold of acceptable movement of less than 0.36 inch, actual displacement in all roof MPBXs never exceeded 0.10 inch. MPBXs were also installed in the wall, where the allowable displacement was 1.2 inches and a maximum of less than 0.5 inch was actually observed.

Following completion of the cavern top heading, work proceeded on the final slip lining of the access and utility shafts. This work involved setting up a shoring tower on the invert of the top heading beneath the two shafts. Cavern excavation resumed in May 2004 and proceeded downward in a total of nine benches until completed in September 2004. The cavern excavation sequence is shown in Figure 19, and the fully excavated cavern is shown in Figure 20.

A sloping layer of graphitic shale was encountered on the second bench at the access shaft end of the cavern. The graphitic shale could be ripped with an excavator making blasting unnecessary. Control of the wall excavation line proved difficult in that scaling the graphitic shale for safety often resulted in

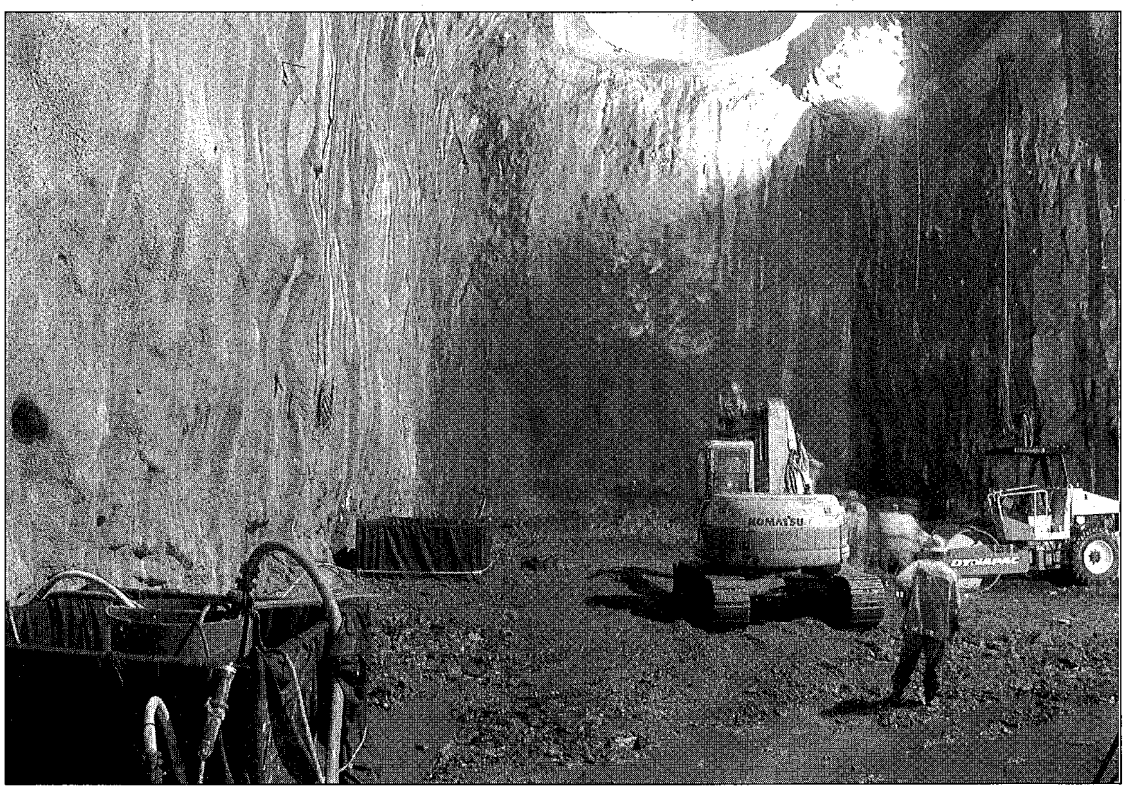

FIGURE 20. The pump cavern viewed from the portal of suction tunnel, fully excavated to invert, with the utility shaft visible in the upper background.

development of a belly in the wall, where overlying loose rock slumped off well beyond the excavation line. This condition resulted in the need to build the wall back out with shotcrete. It was noticed that while shotcrete would stick to a fresh surface of graphitic shale, the outer surface of the thin platy layers would often delaminate, sending chunks of rock and shotcrete to the invert shortly after shotcrete application. These problems were overcome by adapting the application technique to apply an initial thin coat of shotcrete to minimize fall-out. Beyond these observations, the graphitic shale proved no worse to deal with than expected.

Following completion of the excavation, a drainage system of gravel, filter fabric, slotted drain pipe and sumps was installed in the invert. A 230-foot-long, 9-foot-wide, 9-foot-high horseshoe tunnel was then excavated by drill and blast to connect the pump cavern to the S-1 shaft. This suction tunnel was lined with 54-inch-diameter precast cylinder pipe and backfill grouted in October 2004. A rolling scaffold system was then erected on the cavern invert and used to install the galvanized steel roof beams and decking during December 2004. The beams were attached to wall brackets bolted into rock just below the arch. The wall had to be built out at this elevation with shotcrete so the wall brackets could be located at the proper position since blasting overbreak in the wall was as great as 2 feet.

A gutter/downspout system consisting of hypalon hung as a gutter lined the edge of the roof draining to stainless steel downspouts and floor drains leading to a main sump. While this system did not create a bone-dry cavern perimeter, it did work as intended and eliminated the need for a watertight cavern lining designed to withstand 140 psi of hydrostatic groundwater pressure.

In early February 2005, the contractor turned over the access and utility shafts and pump cavern to the NBC so that the subsequent fit-out contract could start. The contrac-

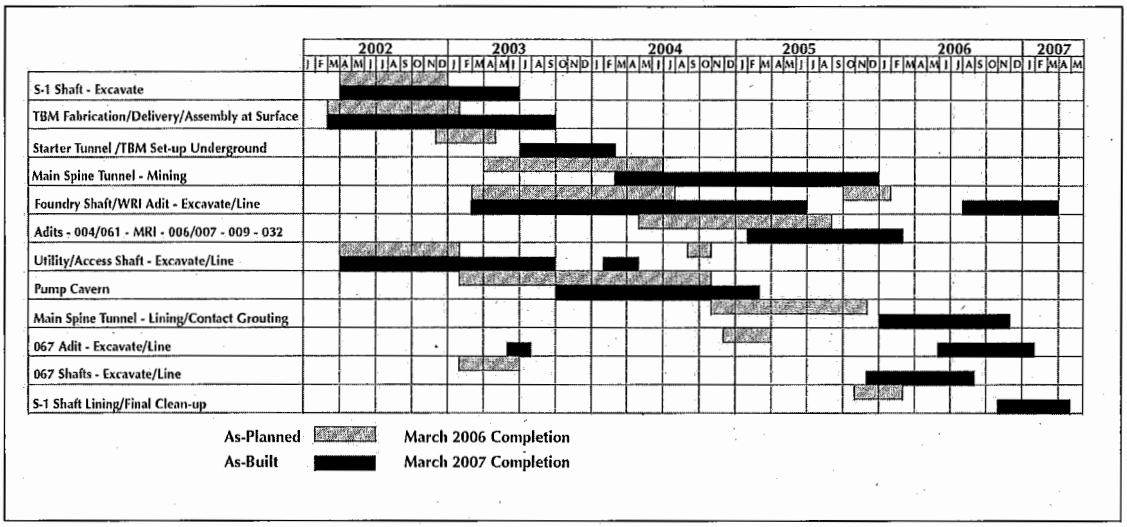

FIGURE 21. Project schedule summary.

tor's expertise in cavern construction was demonstrated by completing the pump cavern on schedule, safely and with minimal change orders associated with additional rock support and shotcrete build-out. The final water inflow into the pump cavern was approximately 35 gpm.

In the final analysis of the cavern construction, several points are worth noting. First, the third-party review performed after the contract was awarded is somewhat unusual in the fact that the contractor demanded it and the owner agreed to it. However, the cost was not excessive nor did it delay delivery of the cavern, and overall it is considered to have been a beneficial process. The importance of addressing the impact of water inflow on the installation of the designed ground support cannot be overemphasized. Techniques also had to be adapted to achieve an 8-inch thickness of shotcrete where flowing water occurred, by channeling or controlling the inflows. Finally, it would have been beneficial for the design documents to have more clearly defined the blasting methods and specified the tolerances of finished wall and roof surfaces.

Schedule & Project Cost

Substantial completion was achieved in March 2007, approximately one year later than planned. A global settlement was reached for all outstanding change-order requests and claims, and a time extension was awarded. The contract was completed for $173 million, 6 percent above the bid price but still under the project budget.

The schedule in Figure 21 shows that, in essence, a year was lost prior to the start of MST mining. Of this year, eight months had been lost on the S-1 shaft freeze and excavation. A concurrent and continuing delay associated with TBM delivery, fabrication and set-up in the starter tunnel caused a further four-month slippage. As a recovery measure, the contractor modified its work plan so that the six adits were mined concurrent to MST mining. The complication to daily operations was well managed by the contractor and greatly mitigated earlier delays. From the start of MST mining until project completion, there was no further erosion of the schedule.

Contractual Features

The contract included three elements typical to major U.S. tunneling projects: a GBR, a dispute review board (DRB) and escrow bid documents. The purpose of the GBR was to establish a contractual baseline of anticipated ground conditions and assist bidders in evaluating requirements for supporting ground and controlling groundwater. The DRB was a three-person panel jointly selected by the contractor and the NBC. There were fifteen board meetings in total, held approximately at a

quarterly frequency. No hearings on specific claims or disputes were ever held. In hindsight, the regularly scheduled DRB meetings were productive and provided a positive influence on the outcome of the project whereby issues were brought into the open.

As with most publicly bid lump-sum contracts, payment was made through monthly requisitions from a schedule of values negotiated after contract award. Approximately 22 percent of the contract price was paid out in allowances or unit price items that were bid and included for those activities where the scope of work could not be determined with a reasonable degree of certainty, such as grouting, discharge water treatment rock support and panning.

Closing

On November 1, 2008, the Phase I CSO system was made operational. The tunnel is now accepting flows from storm events that otherwise would have overflowed into Upper Narragansett Bay. The project is a fine example of a mid-size city making cost-effective use of underground space to improve the environment, in this case dramatically reducing water pollution to an estuary of national significance. The system is operating as planned, and the NBC has embarked on the design of Phase II of its CSO system.

ACKNOWLEDGEMENTS — *The Louis Berger Group, Inc., provides program management services to the NBC for the CSO Abatement Program. The Phase I tunnel, shafts and near-surface facilities were designed by Jacobs Engineering Group, Inc., geotechnical services were provided by Haley & Aldrich, Inc., and construction management was performed by a joint venture of Gilbane Building Company and Jacobs Associates. Construction was performed by Shank/Balfour Beatty (S/BB), a joint venture between the U.S. tunneling firm M.L. Shank, the managing partner and firm that provided all on-site management personnel, and Balfour Beatty, a large British company. S/BB retained Moretrench American Corporation to install freezewalls at all six shafts in the S-1 shaft construction. The cutterhead, main shield and tail shield of the TBM were fabricated by Hitachi of Japan. Photo credits: Figure 8 – Susan Bednarz; Figure 11 – Peter Goldberg; all others – John Kaplin.*

JOHN KAPLIN *is employed by Gilbane Building Company, having served as a resident engineer and construction manager on Phase I of the NBC CSO Abatement Program. Previously, he worked for the Massachusetts Water Resources Authority, Stone & Webster Engineering Corp., and EBASCO Services on heavy civil and underground projects. He has B.S. and M.S. degrees in geology and engineering geology from Colorado State University.*

GEOFFREY HUGHES *is employed by The Louis Berger Group, Inc., as the Principal Tunnel Engineer, supervising the design and construction of the NBC CSO Abatement Program. His twenty years of work experience includes several tunnel projects in the United States and the United Kingdom, including the Massachusetts Water Resources Authority's Outfall and Metrowest Tunnels. He holds a B.Sc. (Hons.) degree in Minerals Estate Management from Sheffield Hallam University, United Kingdom.*

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