Underground Construction for a Combined Sewer Overflow System in Providence, Rhode Island

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ABSTRACT

The underground construction for the Phase I Combined Sewer Overflow (CSO) Project is on an impressive scale by any measure, appearing more so in a midsize city like Providence, RI. The Narragansett Bay Commission recently completed $350 million in construction to improve the water quality in Narragansett Bay. This project included a number of serious challenges, delays, and disputes. Looking back on this complex project completed within budget, it is hard to consider the work as anything other than a success. Along the way, this outcome did not always appear certain.

INTRODUCTION

With a sewer system dating back to the 1870s and a treatment plant dating back to the early 1900s, Providence and the surrounding communities have a long history of keeping up with population growth and technological advances in handling the city's sewage and stormwater. The Narragansett Bay Commission (NBC), which has existed since 1982, provides sewerage treatment for 10 communities, with 360,000 residents and 8,000 businesses in the Providence area. Many upgrades were performed on NBC's system over the years. Since its inception, NBC has improved the function of the Fields Point Wastewater Treatment Facility from one of the nation's worst-rated treatment system in the 1980s to being designated by the EPA as the nation's best secondary treatment system in 1995. However, despite an effective secondary treatment system, Providence is burdened with combined sewer and stormwater overflows to the upper Narragansett Bay on an average of 71 times annually. The Bay is a water body of national estuarine significance, and the overflows have major impacts on shellfish harvesting and other commercial and recreational uses.

The Combined Sewer Overflow (CSO) system was mandated by the federal Clean Water Act to end overflows. A preliminary design was first developed in 1995 to address the overflows. Since then, the design has developed into a three-phase program that has spread construction out over 20 years.

When the $350 million plus Phase I program began, it was the largest civil works project in the history of Rhode Island. Although its scope is impressive in its own right, it is amplified when considering the project was constructed in the smallest state in the union by a midsize wastewater agency. Prior to Providence, deep tunnel storage projects for CSO abatements in the U.S. had been built predominantly in major metropolitan areas by larger agencies such as Milwaukee, Chicago, and Atlanta. The NBC managed to overcome difficult funding and public affairs hurdles to start Phase I. In terms of scale, this is a relatively small owner tackling a large project.

Figure 1 shows the Phase I layout. Starting in 2001, nine construction contracts were issued to build the Phase I CSO system. The subject of this paper is one of these contracts, Contract 6, which was the largest and covered the underground works. It was completed between 2002 and 2007 by Shank/Balfour Beatty (S/BB) with a final contract value of $173 million, 6% above the original bid. The scope of the contract is summarized in Table 1.

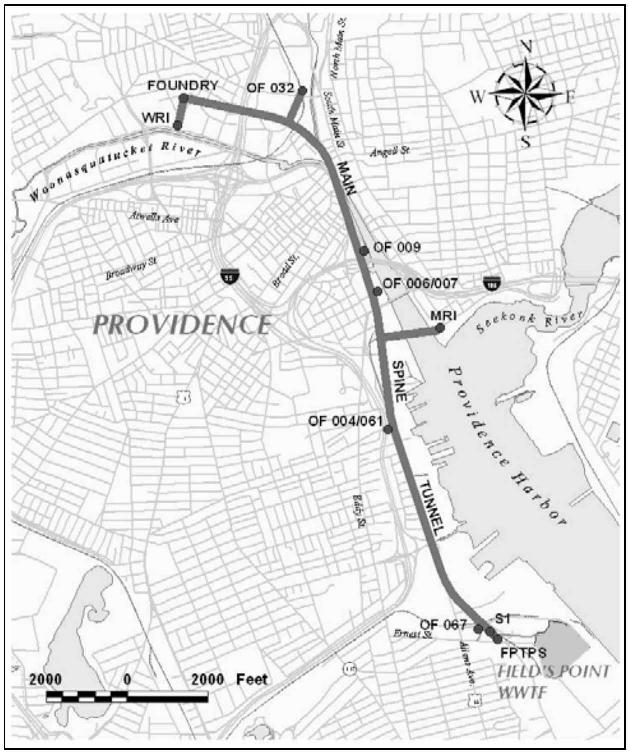

FIGURE 1. Location Plan.

TABLE 1. Contract 6 scope of work	
Work Element	*Description*
Shafts (six total)	• S-1: main work shaft servicing MST; screening shaft for permanent works. • Utility and Access: servicing the pump cavern. • Foundry: north terminus of MST; vent shaft for permanent works. • 067: drop and vent shafts for a CSO connection (six other locations constructed under other contracts; not in scope of this paper).
Main Spine Tunnel (MST)	4,963 m (16,284 ft) long, 7.9 m (26 ft) diameter, 70 m (230 ft) deep. This is the main CSO storage tunnel, which can hold 235 million liters (62 million gallons).
Adits (seven total)	2.7 to 4.8 m (8.75 to 15.75 ft) diameter deaeration chambers; 2.4 m (8 ft) diameter adits. Adits inclusive of chambers total 1,237 m (4,057 ft) in length. Convey CSO flows from drop shafts to MST.
Pump Cavern	A 189 ML/day (50 MGD) Field's Point Pump Station housed in a 92 m (300 ft) deep rock cavern with dimensions of 35.7 m × 18.6 m × 20.7 m (117 ft × 61 ft × 68 ft). S/BB constructed the cavern, floor, and roof; all other fit-out work was performed in subsequent contract. The Contract 6 pump cavern work is not covered in this paper (see Hughes et al. 2008).

Other Phase I contracts outside the bounds of this paper include six construction packages that tie the existing CSO system to the tunnel. Each of these packages consists of diversion structures, connecting conduits, gate and screening structures, and drop and vent shafts drilled down from the surface. The construction of these shafts is discussed in Castro et al. (2007). There also is a $54 million contract in the final stages of completion to fit-out an underground pump cavern with a 189 ML/day pump station, construct associated support buildings, and complete the mechanical, electrical, and instrumentation work. As of this writing, all of this work is sufficiently completed such that the bulk of Providence's storm overflows within the Phase I area is now collected in a tunnel rather than polluting the bay. Effluent is pumped from the tunnel after storms to the existing Fields Point Treatment Plant at the south end of Providence.

Three bids were received for Contract 6 on September 25, 2001, ranging from $163.5 million to $227.4 million. As low bidder, S/BB was awarded the contract in December 2001. S/BB mobilized to the site in early 2002.

SHAFTS

First, three shafts were sunk at the S-1 site at the southern terminus of the project, near the Field's Point Treatment Plant at the Port of Providence. The S-1 was sunk first, starting in 2002, followed by the Utility and Access shafts. Construction on the Foundry shaft at the northern end of the Main Spine Tunnel (MST) started later, in 2003, as it would not be needed until the MST hole-through. The 067 drop and vent shafts were frozen and raise-bored near the tail end of Contract 6. Table 2 summarizes pertinent facts about these six shafts.

TABLE 2. Shaft excavation data

	S-1 Shaft m (ft)	*Utility Shaft m (ft)*	*Access Shaft m (ft)*	*Foundry Shaft m (ft)*	*OF-067 Drop Shaft m (ft)*	*OF-067 Vent Shaft m (ft)*
Excavation diameter	Soil: 15.2 (50) circular Rock: 10.4 (34) square	11 (36)	4.3 (14)	11 (36)	3.7 (12)	1.8 (6)
Finish diameter	7.9 (26)	9.8 (32)	3.4 (11)	7.9 (26)	2.7 (9)	1.2 (3.9)
Depth to water table	4.6 (15)	6.4 (21)	6.4 (21)	7 (23)	4 (13)	4 (13)
Depth to top of rock	50 (165)	48 (157)	53 (174)	46 (152)	50 (164)	50 (164)
Depth of shaft	89 (292)	74 (242)	74 (242)	78 (257)	71 (233)	71 (233)
Soil support method	Ground freezing					
Initial and final shaft liners	Cast-in-place concrete, slip-lining method					

The S-1 is the main work shaft from which the MST was excavated. As noted in Table 2, the depth to bedrock is quite deep at 50 m (165 ft). The freezewall constructed to provide soil support for this shaft encountered a substantial delay, which impacted the critical path schedule of the project and the overall Phase I program. It was originally estimated that the freezewall would close in 38 days from the start of the freeze, and by 50 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. Further time was lost during the shaft excavation due to the increased ice growth inward during the initial freeze delay. This additional frozen ground had to be chipped by impact hammer, prolonging the excavation far beyond the time it would have taken had the bulk of the core remained unfrozen as planned. These events led to considerable efforts to determine cause and mitigate delays. The contractors worked aggressively to resolve the freezing problems with a variety of techniques, including additional freeze pipes, instrumentation, grouting, and pump tests.

After the freezewall closed and soil excavation was completed, the contractor and freezewall subcontractor submitted a claim that contended that three differing site conditions (DSCs) had interacted to cumulatively delay the freeze. They claimed that “free-phase” gasoline contamination delayed the freeze near the top of the shaft and was remedied by the installation of shallow freeze pipes. This problem masked two deeper “windows” at 37 m and 47 m (120 ft and 155 ft) depths caused by higher-than-anticipated permeability and groundwater flow up from bedrock through preconstruction grout holes that had been incompletely sealed by a previous NBC contractor.

The owner’s team was not in complete agreement with the contractors’ explanations as to the causes for the freezewall problems. For example, no “free-phase” gasoline was ever discovered. When freezing was initiated, the shaft core had already been excavated nearly to the groundwater table, which was then exposed to hot summer temperatures that apparently overwhelmed the desired effect of the freeze pipes on groundwater temperature. Thus, warm water, not gasoline, was the most likely culprit of the shallow freeze problems, and this conclusion was supported by direct temperature measurements. As for the deep windows, 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, a contractor’s own observation well, or grout holes as asserted. Soil permeability was found locally to be higher than anticipated; however, the gradient of the groundwater table was no steeper than described in the contract documents.

There also was some debate about contract provisions, specifically whether the geotechnical information included was sufficient to determine groundwater flow for the purposes of designing a freezewall, and whether the contractor needed to perform investigations. This was never done, with the freezewall contractor arguing that the information already included was adequate.

Sorting out responsibility for delays on a freezewall claim with multiple problems is certainly not clear cut. In the end, agreement was reached as part of a global settlement. The lessons learned are that closer attention must be paid when determining whether groundwater flow at a site is an issue for ground freezing, who will conduct those investigations, and how it is accounted for in the design of the freezewall. Secondly, a shaft collar installed and soil excavated to near the groundwater table within the core also must be properly accounted for in the design of the freezewall, especially during summer months when elevated air temperatures may persist very close to the ground that is to be frozen. The causes and mitigation measures to resolve this problem are described in greater detail in Schmall et al. (2007), which presents the perspective of the ground-freezing contractor.

The ground freezing at the other five shafts proceeded largely as planned. Cast-in-place (CIP) liners were used for longer-term soil support by means of the slip-lining method at all shafts, following which the ground-freeze systems were shut off. Excavation of rock in all the shafts was by drill and blast. The exception was the raiseboring of the 067 shafts. All shaft rock excavation proceeded without any noteworthy problems. However, there were some issues at the Foundry shaft that deserve mention, as briefly described below.

During the drilling for Foundry shaft pregrouting from the surface and the subsequent installation of freeze pipes, heavy water losses and grout takes were observed in the upper 6–9 m (20–30 ft) of bedrock. No clear cause of permeable ground was detected in the core borings, so there naturally was cause for concern with shaft sinking through this ground. A decision was made to increase the length of freezewall pipes to a depth from surface of 56 m (185 ft), well into rock. When the shaft was excavated through this zone, the occurrence of 12–50 mm (0.5–2.0 in.) wide fractures filled with a clean, fine-to-medium sand of probable glacial origin were discovered at a depth of 6 m (20 ft) below the top of rock. Grout had penetrated some of the fractures, but incompletely. These fractures would have been heavy water makers had the ground not been frozen and subsequently sealed by the cast-in-place liner. Even then, chemical grouting through the CIP liner was required to cut off silt laden inflows that occurred through shrinkage cracks.

These fractures also were observed to a lesser extent in the Utility shaft, where some remedial work was required. In fact, the authors are aware of other shafts in New England where this problem has been encountered. The common characteristics of these fractures are their occurrence in the upper 6 m (20 ft) of bedrock, an aperture of up to several inches, a filling of clean sand, and the potential to produce high inflows at depths below the water table. They often are undetected by the single boring typically used for shaft exploration and require grouting, time, and money so that shaft sinking can proceed. Perhaps these features are unique to shaft sinking at locations with a record of glacial activity.

MAIN SPINE TUNNEL EXCAVATION

It has been a practice of the M.L. Shank side of the S/BB team to design and build its own tunnel boring machines (TBMs), and the Contract 6 machine was a scaled-up version of their previous smaller TBMs. Other than the cutterhead, main shield, and tail shield being fabricated by Hitachi of Japan, the fabrication and assembly of the TBM and trailing gear were performed by S/BB forces.

Under a value-engineering proposal, the tunnel liner was a composite consisting of 25.4 cm (10 in.) thick precast segments and a cast-in-place liner 30.5 cm (12 in.) thick, placed after mining was completed. Installation of the precast segments was within the TBM shield. As the TBM advanced forward and the four-piece, 1.2 m (4 ft) wide precast ring emerged from the tail shield, hydraulic jacks were engaged in a 0.6 m (2 ft) wide open key gap at the crown to expand the segments against the rock. Struts filling the key gap were installed thereafter to hold the segments in place.

The cutterhead was 9.1 m (30 ft) in diameter and equipped with 65 43-cm (17-in.) diameter, back-loading cutters. The machine employed 20 jacks to propel and steer off precast segments, with a total available thrust of 28,024 kN (6,300,000 lb). The cutterhead was rotated at 3 to 5 rpm and was driven by 20 hydraulic motors rated at a total of 1,837 kW (2,465 hp). Power for the thrust, head rotation, conveyors, and segment erection was electric over hydraulic.

The TBM was launched from a starter tunnel at the S-1 shaft on March 8, 2004 (Figure 2). Within a month, production was in the 12.2 to 13.7 m/day (40 to 45 ft/day) range. Mining was performed on one long shift, from 6:30 a.m. to 6 p.m., with maintenance performed on the back shifts. The basic mining cycle consisted of mining through 1.2 m (4 ft) of rock in 25 to 40 min, separated by time to install segments and wait for trains. Haulage by rail was performed by three locomotives hauling five muck cars and one segment car. Two rail switches were used—a fixed one at the S-1 shaft and a second, forward one that was periodically advanced toward the heading to improve rail transit. A muck car roll-over dump, a vertical conveyor in the shaft, and a radial stacking conveyor at the surface were used to move tunnel muck to a surface pile, where it was trucked off site for commercial uses. As with the TBM, most of this equipment was designed and fabricated by S/BB forces.

FIGURE 2. TBM in starter tunnel.

The rock encountered in all the underground works was a weakly metamorphosed sequence of folded sedimentary rocks of the Rhode Island Formation that includes sandstone, shale, siltstone, graphitic shale, and conglomerate. With the exception of the graphitic shale, the rocks were stable with good stand-up time.

A ground classification system for tunneling was included in the Geotechnical Baseline Report (GBR) and made part of the contract documents, as a two-pass lining system was permitted in the bid documents. The classification system was based on criteria that characterized the rock behavior and was intended for use in the selection of ground support. Type I ground required rock bolts and shotcrete; Type II ground called for steel sets, lagging, and shotcrete. S/BB's precast segments installed immediately behind the TBM made the need to classify ground unnecessary. However, the classification system did become a source for a differing site condition (DSC) claim related to TBM penetration rate. S/BB 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. This was not an unreasonable approach, although certainly not the one intended by the authors of the report. Considerably more Type I ground was encountered in the MST than the 65% predicted by the GBR, and the actual TBM penetration rate was lower than bid. The claim was eventually settled as part of a global settlement.

None of this should overshadow the overall performance of the TBM, which operated with a high degree of reliability and made steady progress. The TBM holed through at the Foundry shaft in early December 2005 (Figure 3). Production on the single long shift, five days a week, yielded 274–427 m (900–1,400 ft) per month, with a daily production between 12.2 m and 18.3 m (40 ft and 60 ft) per mining day. Table 3 provides a summary of performance data for the TBM. One factor that affected monthly production was the number of days lost to mining to perform pre-excavation grouting, as discussed next.

TABLE 3. TBM Production Data		
Description	*Quantity*	*Unit*
Single-shift mining	10.5	h
Average TBM penetration	2.1 (6.8)	m/h (ft/h)
Average cycle time (mine 1.2 m [4 ft], erect segment ring; wait for next train at heading)	54	min
Mining	36	min
Segment erect	11	min
Wait for train at heading	7	min

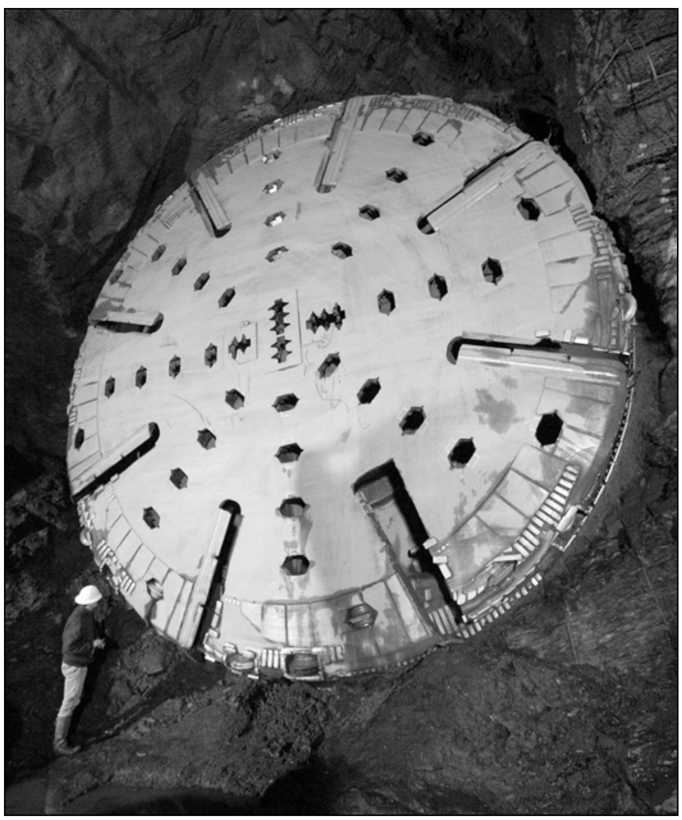

FIGURE 3. TBM cutterhead following hole-through at the Foundry shaft *(photo by Sue Bednarz)*.

PRE-EXCAVATION GROUTING

It was anticipated that pre-excavation grouting would be required to maintain a total steady-state groundwater inflow from the MST, adits, and shafts below 12,490 Lpm (3,300 gpm). All inflow except that from the pump cavern was pumped from a dewatering station at the bottom of the S-1 shaft to a settling pond and pretreatment system at the surface, then to the Field's Point Treatment

Plant for discharge. Contract specifications called for grouting if probe holes through the tunnel face, which had to provide continuous, overlapping coverage, produced greater than 190 Lpm (50 gpm). Once triggered, grouting would be performed using Type III or microfine cement grout.

During the early stages of mining, inflow into the tunnel increased at a greater rate than anticipated, despite the use of the pre-excavation grouting program. By the time 25% of the MST had been mined, the inflow already had reached 4,540 Lpm (1,200 gpm), or 36% of the total anticipated inflow. For this reason, the grouting program was modified a number of times, and it is worth noting the more significant changes to the program based on conditions encountered. A summary is shown in Table 4. Figure 4 is a graphic of the inflow and the grout-event history.

TABLE 4. Pre-excavation Grouting Program Summary

Criteria	*Original Program*	*Modified Program*	*Discussion*
Grout trigger	190 Lpm (50 gpm)	57 Lpm (15 gpm)	Trigger was changed several times: first down to 26 Lpm (7 gpm), then up to 57 Lpm (15 gpm), which was the predominant trigger during mining.
Grout pressure	3.5 MPa (500 psi)	2.4 MPa (350 psi)	At the higher pressures, grout takes were in the range of 142–170 m^3 (5,000–6,000 ft^3) per event. It was judged that a considerable portion of the grout was pushed through fractures outside the tunnel envelope, perhaps due to hydrojacking. After the pressure was reduced, grout takes were lowered by a factor of two to four, with no apparent loss in effectiveness, as is visible in Figure 4 from Station 80+00 and beyond.
Grout holes	two	four	Based on inspection of the face, it was judged that two holes were insufficient for a 9.1-m (30-ft) diameter face.
Equipment	Complete mobilization of all grouting equipment from surface to heading for each grout event, probing and grouting through one hole at a time.	Rolling gantry behind the TBM trailing gear was fabricated; included equipment to allow rapid mob/demob for grout events and ability to probe/grout two holes simultaneously. A second grouting/mixing plant was also fabricated to boost grouting capacity.	No mining could be accomplished when pre-excavation grouting was performed. Initially, mob/demob time resulted in two lost mining days per grout event. These equipment changes, which the owner paid for through a change order, reduced the lost mining days from two to one while increasing the number of holes that could be grouted at a time.
Grout	Microfine and Type III	Same. However, with probe flow of greater than 190 Lpm (50 gpm), Type III would be started with to reduce cost.	This was done to reduce cost by initially filling heavy water-making fracture systems that took large quantities of grout at low pressure with the less-expensive Type III cement.
Cost	$5.8 million paid using unit price items	$10 million using unit price items with quantities replenished through change order.	All parties universally agreed that inflow had to be reduced by pre-excavation grouting to a point where it was manageable for mining and final CIP lining, and that cut-off grouting (attempting to grout off inflows after exposed in the mined opening) was not a reasonable approach. A total of 3,364 m^3 (4,400 yd^3) of grout was pumped over 44 grouting events, or 59 working days.

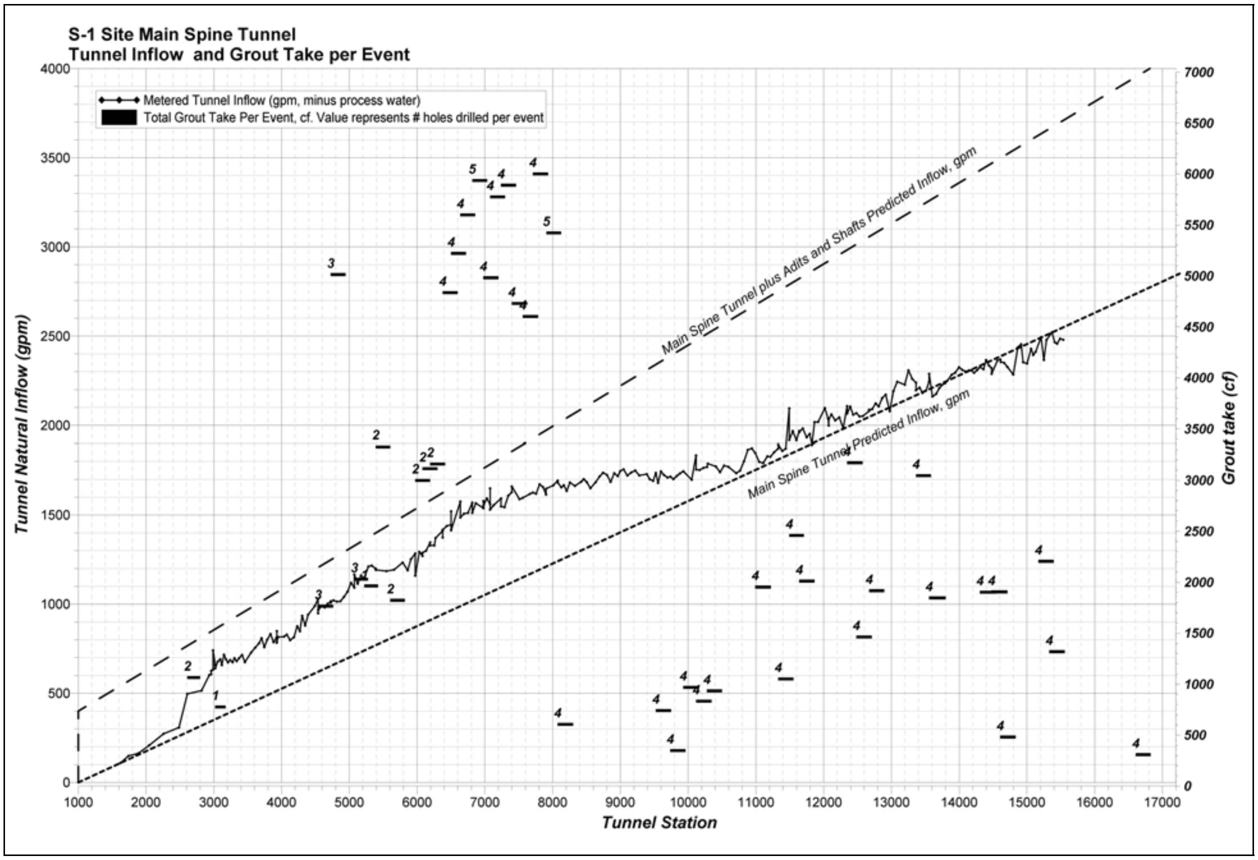

FIGURE 4. Water inflow and grout take per event in the Main Spine Tunnel.

There was relatively little decay observed in the inflow rates over time in the MST. This assessment was based on direct measurement of sustained inflows. The overall effectiveness of pre-excavation grouting can therefore be analyzed in rough terms by comparing probe inflow to tunnel inflow in grouted versus ungrouted portions of the tunnel. The results are shown in Table 5.

TABLE 5. Pre-excavation grouting performance summary.
Grouted versus ungrouted intervals of the Main Spine Tunnel

	Interval length, m (ft)	*Increased inflow in tunnel, Lpm (gpm)*	*Cumulative probe inflow for interval, Lpm (gpm)*	*Ratio of increased tunnel inflow/ probe inflow*
Grouted intervals	2,425 (7,957)	4,546 (1,201)	12,343 (3,261)	0.37
Ungrouted intervals	2,275 (7,465)	5,435 (1,436)	1,154 (305)	4.71

It is critical to understand the purpose of a grouting and water handling program in a rock tunnel. In this case, the intention was to keep inflow at a level manageable for tunneling equipment operation and successful placement of the CIP liner without washout. Following placement of the liner, subsequent contact grouting and a final round of chemical grouting at a few select locations, the long-term, steady-state inflow from the MST, all adits, and connecting shafts was approximately 1,140 Lpm (300 gpm).

ADIT CONSTRUCTION

Seven adits were excavated off the MST to connect the tunnel to the drop and vent shafts previously installed by other NBC contractors. To mitigate the schedule loss of a year's time that occurred prior to the start of MST mining, all adits were excavated and lined concurrently with the excavation of the MST. The exceptions were the 067 and WRI adits, which were driven from the base of the S-1 and Foundry shafts, respectively. Elevated decks were constructed in the MST at each of the adit portals to service the adit excavation and concrete lining. This allowed access to the adits, which intersected the MST at the spring line while allowing MST train traffic to pass unimpeded beneath the deck (Figure 5).

FIGURE 5. Elevated deck for construction of the adit from the Main Spine Tunnel.

Excavation was by drill and blast and proceeded on a 24-hour basis. Most of the blasting was performed on the second and third shifts so as to avoid interference with the daytime MST mining operation. Considerable outreach helped project neighbors understand the program and allayed common fears that come with blasting. Convincing authorities to permit 24-hour blasting was also simplified because few businesses above the adits operated at night and there were few residences.

One of the adit alignments is beneath the main power plant in the city, causing concern with the local hospital, which had previously experienced power outages unrelated to construction. Through meetings and test blasts, these concerns also were addressed, and adit excavation proceeded as planned.

Ground support consisted of swellex bolts and shotcrete in competent rock (Type I ground). Steel sets, lagging, and shotcrete were used where weak graphitic shale was encountered (Type II ground). Probing ahead of the excavation face was performed, yet pre-excavation grouting was triggered rarely and water inflow was generally not a problem in the adits. Table 6 provides a summary of pertinent adit/chamber data.

TABLE 6. Adit/Deaeration Chamber Summary									
				Adit		*Chamber*			
Adit	Adit length m (ft)	Chamber length m (ft)	Total length m (ft)	Adit Excavated Diameter m (ft)	Finished Diameter m (ft)	Chamber Excavated Diameter m (ft)	Chamber Finished Diameter m (ft)	Ratio Type I/II Ground %	Shifts to excavate
067	3.0 (9.7)	36 (117.5)	39 (127.2)	3.4 (11.3)	2.4 (8)	5.8 (19)	4.8 (15.75)	100/0	69
004/ 061	1.5 (5)	20 (65.5)	22 (70.5)	3.4 (11.3)	1.4 (4.5)	3.7 (12)	2.7 (8.75)	100/0	28
MRI	521 (1,710)	20 (65.5)	541 (1,776)	3.4 (11.3)	2.4 (8)	3.7 (12)	2.7 (8.75)	86/14	386
006/ 007	96 (314)	20 (65.5)	116 (380)	3.4 (11.3)	2.4 (8)	3.7 (12)	2.7 (8.75)	100/0	81
009/ 010	40 (132)	36 (117.5)	76 (250)	3.4 (11.3)	2.4 (8)	5.8 (19)	4.8 (15.75)	100/0	83
032	226 (740)	20 (65.5)	246 (806)	3.4 (11.3)	2.4 (8)	3.7 (12)	2.7 (8.75)	100/0	152
WRI	189 (619)	20 (65.5)	209 (685)	3.4 (11.3)	2.4 (8)	4.3 (14)	3.2 (10.5)	33/67	143

The deaeration chamber connections to the drop and vent shafts installed under other NBC contracts proceeded as planned with one exception. A failure of the shaft liner was experienced just above the intersection with the chamber at one adit where Embedded Cylinder Pipe (ECP) had been used for a shaft liner. In just a few hours, the mortar lining spalled off the bottom 6 m (20 ft) of the shaft in dramatic fashion, and the thin steel lining ballooned inwards under the groundwater pressure, pinching the shaft diameter to a few feet. It was apparent that water had leaked in between the inner layers of the pipe, causing it to burst. The failed pipe section was removed and repaired with mesh and shotcrete. In hindsight, a steel transition section for the bottom 3 m (10 ft) of each shaft where cylinder pipe was selected would have been far more robust and able to withstand the rigors of the chamber excavation and shaft installation. For three other locations that also had ECP pipes used for shaft liners, procedures were revised and all were completed without problems.

Concrete lining and contact grouting of the adits and deaeration chambers was accomplished by pumping from a surface pump fed by ready-mix trucks through slicklines run down the drop shafts.

MAIN SPINE TUNNEL—FINAL LINING

The final cast-in-place MST liner was constructed from the Foundry shaft to the S-1 shaft by pumping concrete from the surface at four locations. A fleet of ready-mix trucks delivered 27.6 MPa (4,000 psi) concrete with 10 mm (3/8 in.) stone from a batch plant constructed and operated by S/BB at the S-1 site. The maximum pumping distance was 1,006 m (3,300 ft) from the pump to the forms. Additives included new generation full-range and high-range water reducing admixtures, retarder, fly ash, and an air-entraining admixture. A total of 44,340 m^3 (58,000 yd^3) of cast-in-place concrete was placed in the MST for the final lining.

Once beyond the initial learning curve, 50 m/day (165 ft/day) placements were achieved. This required a sustained delivery of 443 m^3 (580 yd^3) over eight hours. A bulkhead was constructed at the downstream end (Figure 6) for each day's pour, and concrete was typically filled to a height of approximately 80% full or higher, leaving a sloping construction joint. Within the forms, a slickline at the crown was dragged along hanging rollers as the pour progressed.

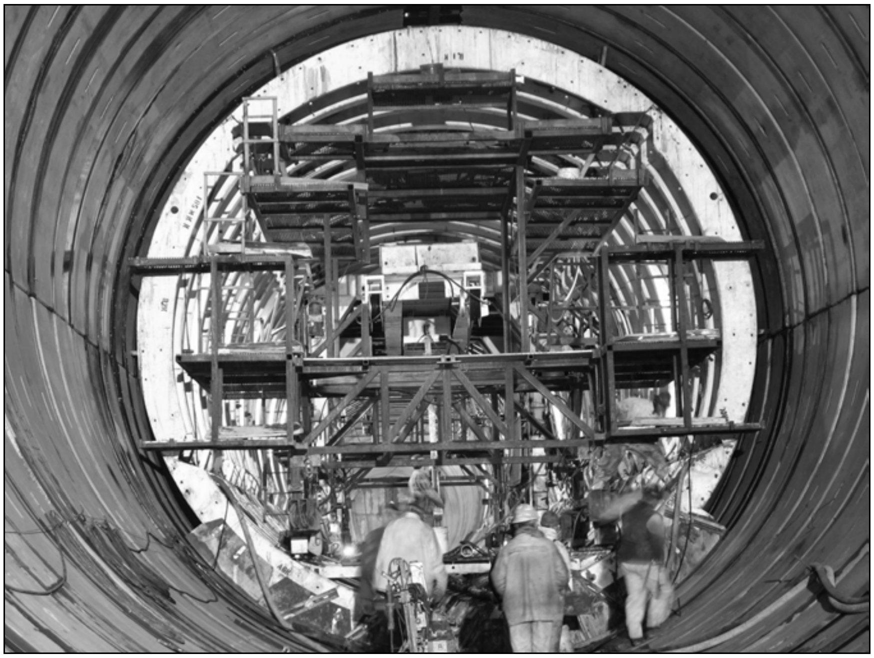

FIGURE 6. Gantry for bulkhead construction at downstream end of Main Spine Tunnel forms.

When the liner was nearly complete, displacement was observed to have occurred along this construction joint at about half of the 100 bulkhead locations. In the most severe case, a concrete block approximately 0.6 m × 1.2 m (2 ft × 4 ft) fell to the invert. An 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 and implemented to remove the displaced concrete, install dowels, and backfill with shotcrete. Pressure relief holes were added to the problem areas. Repairs were completed without impact to the project schedule, and costs to perform this work were split between the NBC and S/BB. An inspection performed a year after the work was completed revealed that the repairs were successful. In the future, the authors suggest that the use of sloping cold joints in large diameter cast-in-place liners should be avoided and full bulkhead pours stipulated as a requirement.

CLOSING

A year was lost on the schedule prior to the start of MST mining. Of this, six months is accounted for on the S-1 shaft freeze and excavation. A concurrent and continuing delay associated with TBM delivery, fabrication, and setup in the starter tunnel resulted in an additional four-month slippage. S/BB developed a recovery plan to mine and line six of the seven adits concurrently with MST mining. This plan was successful in preventing further schedule loss and was well managed by S/BB. Figure 7 depicts a comparison of the planned and actual project schedules.

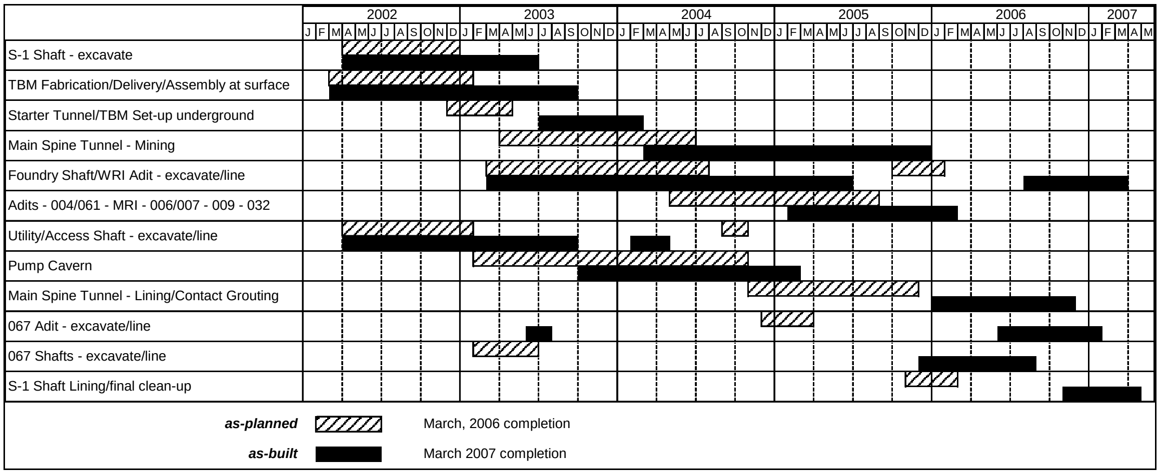

FIGURE 7. Comparison of planned and actual schedules.

Contract 6 was completed within budget and at a final contract price within 6% of the original bid price. Although some delays also occurred on the shaft/surface contracts, the overall Phase I program also was completed within budget and within 7% of the total original bid price.

Prior to the Phase I CSO, the last major tunneling project completed in Providence was over 50 years ago, and it was a relatively short bus tunnel. Phase I remained largely out of the local press, allowing the project to achieve its objectives without negative reviews. Multiple tunneling technologies were used to improve the environment, in this case Narragansett Bay. This project is an example of how a public agency in a midsize city like Providence, without significant tunneling experience, can tackle a large and complex underground project for the benefit of its citizens and do it within budget. Tackling problems early and head on is one of the keys to success, particularly with large projects.

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