The Bypass Tunnel: Excavation, Interliner, and Lining

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ABSTRACT

The Rondout-West Branch Bypass Tunnel is being built to bypass a substantial leak in New York City's primary water supply tunnel. The 13,500 foot long tunnel travels 600 feet under the Hudson River through the same troubled geology and high pressure water inflows that the original tunnel did in the 1940s. A Robbins TBM traversed this geology and built gasketed segments. Nine thousand two hundred feet of 16 foot diameter interliner was installed and backfilled within the segments. Finally, reinforced cast in place lining was placed. This paper addresses the challenges and the multi-tasking approach the contractor utilized.

INTRODUCTION AND BACKGROUND

The New York City Department of Environmental Protection (DEP) has a significant leak in a primary water tunnel that delivers 50 percent of the City's water from upstate reservoirs to the City. The Rondout-West Branch Tunnel (RWBT), a portion of the Delaware Aqueduct, leaks between 15 and 35 million gallons per day primarily on the west side of the Hudson River in Roseton, NY. This portion of the 45-mile-long RWBT where most of the leaking occurs is 600 feet under the Hudson River, and it is also the low point of the tunnel. Safety issues associated with working at the low point of a tunnel, while millions of gallons of groundwater are entering the tunnel, rendered a traditional in-tunnel repair impossible.

After much study, the solution to the leak is to build a Bypass Tunnel to circumvent the leaking portion. The Bypass Tunnel is offset 1,750 feet to the north of the existing tunnel. See Figure 1. The connection points at the ends of the Bypass Tunnel where it will be connected to the RWBT are located in competent rock formations. The Bypass Tunnel includes 9,200 feet of steel interliner that will contain the internal water pressure as the tunnel passes through the same problematic geology as the original tunnel. The new tunnel is 13,543 feet long, 14 feet finished diameter, and connects to the RWBT at the same elevation, about 600 feet below sea level.

Prior to the connection, the Bypass Tunnel will be built in its entirety, but not connected to the RWBT. The Bypass tunnel excavation ends 100 feet from the RWBT at each end, leaving a rock barrier.

The length of the Bypass Tunnel (13,543 linear feet) is such that it could be excavated economically with a tunnel boring machine (TBM) or by drill and blast methods. The DEP decided to let the market/tunneling industry decide whether to excavate the tunnel by drill and blast methods or by using a TBM and a segmental lining. The bid documents allowed for each method. The contract was awarded to the low bidder, a joint venture Kiewit-Shea Constructors Inc (KSC).

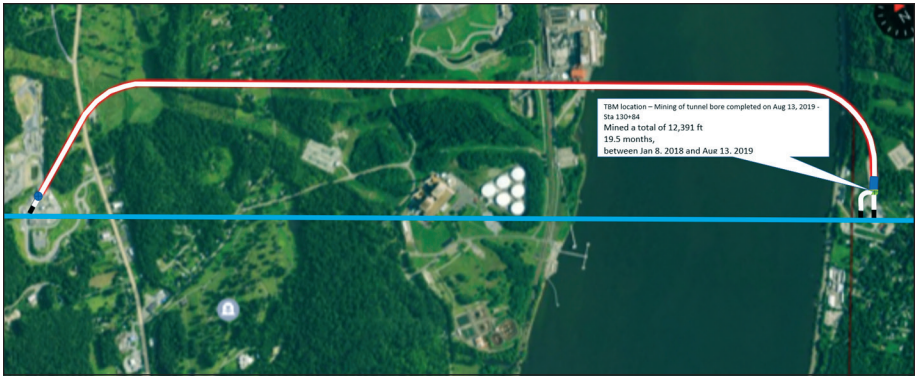


Figure 1. Rondout West Branch Bypass Tunnel—TBM Progress and Hole-Through Location (blue dot)

GEOLOGY AND GROUNDWATER

The RWBT is located in the physiographic province of New York State's Lower Hudson Valley within the Hudson Valley fold-thrust belt. The geology consists of very thick sequences of Ordovician-Age sedimentary rocks deposited roughly 450 million years ago, during the Ordovician Period of the Paleozoic Era. The bedrock is overlain by glacial soils deposited approximately 17,000 years ago during the most recent regression of the (Late) Wisconsinan Ice sheet.

During Paleozoic and Mesozoic Eras the rocks were deformed and slightly metamorphosed during episodes of crustal tectonic plate collision and extension during long periods of Super Continent development and break-up. Subsequent crustal extension reactivated pre-existing normal faults in the region. More recently, erosional remnants of taller mountains and plateaus were then dissected by water and glacial erosion.

The RWBT was excavated through three different sedimentary rock formations with distinct distinguishing characteristics. The rock formations include the foliated Normanskill Shale Formation on the west, the Wappinger Group consisting of limestone and dolomitic rocks in the middle, and the Indian River/Mount Merino Shale

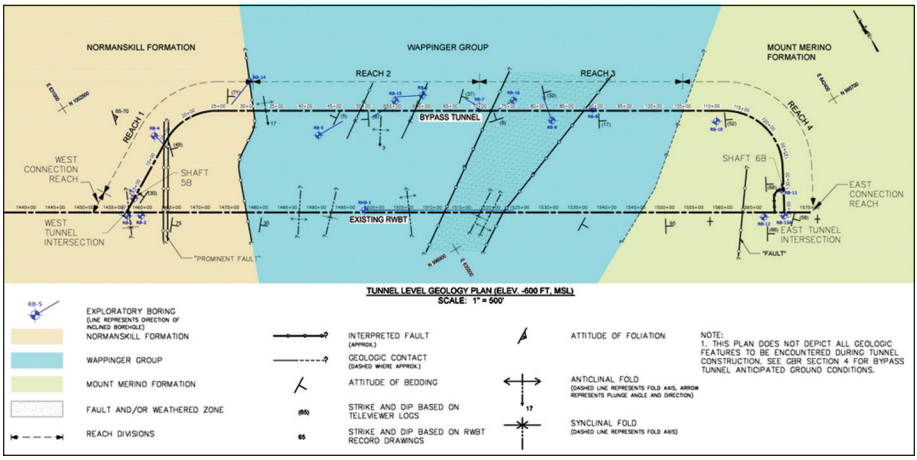


Figure 2. Geologic Formations of the Bypass Tunnel

Formation on the east. See Figure 2 below for illustration of the project tunnel alignment through the three project rock formations.

The rock formations can be highly jointed, folded and faulted. The jointed and faulted rock can be highly permeable in places. The major faults trend generally Northeast/Southwest. Secondary sub-faults trend somewhat perpendicular, west-northwest/east-southeast to the major faults. The rock formation boundaries are typically fault contacts. The strike of the formation contacts are sub-parallel to the trend of major fault and/or axis of regional folds. Local variations in bedrock joint patterns are present in places as a result of the deformation associated with the historical folding and thrust faulting.

RWBT EXCAVATION METHODS

The RWBT was excavated by both Drill & Blast (D&B) methods and a Robbins manufactured single shield hard rock tunnel boring machine (TBM) with bolted and gasketed precast segments. D&B methods were used for the lower 20 feet of both the Shaft 5B launch and Shaft 6B receiving shafts, the Shaft 5B Bellout/TBM starter tunnel, and at a lower level, to advance the drainage tunnel. The construction shafts (Shafts 5B and 6B) were constructed under a previous contract.

DRILL AND BLAST

The Bypass Tunnel's D & B headings consisted of cumulative, 1,300 feet of tunneling and are as follows:

- Shaft 5B Bellout constructed at the west end of the tunnel is 43 feet wide by 35 feet tall, and 130 feet long; and a contiguous, 350 feet long, 25 feet diameter conventional starter tunnel for TBM assembly;
- Each end of the tunnel contains a short tunnel between the construction shafts and the existing RWBT, a 150 feet long west connection tunnel and the 265 feet east connection tunnel; and
- At a lower elevation and connected to Shaft 6B is a roughly 400 feet long 20 feet diameter drainage tunnel that will drain the existing RWBT during construction of the substantial inflows (reverse leaks).

The Shaft 5B Bellout presented the most challenging heading to excavate in terms of shape and initial rock support across the arch. The contractor and their designer, Brierley Associates designed a sequential excavation, top heading and bottom bench approach to excavate the 5B Bellout. The top heading was 18 feet (5.5 m) tall and divided into three faces—a 22-foot wide center and two 11-foot wide, side faces. The center face was advanced ahead of the side faces with a lag of approximately two rounds of blasting, or about 16 to 20 feet. The bench was 14 feet tall and was also excavated by center and slash sequence. Each round was mapped by both the designer and construction manager's geologists to determine the rock class and corresponding initial support design. Initial rock support included a passive 16 feet long solid number 8-bar spaced on a 5-foot by 5-foot staggered pattern.

Blasting restrictions limited vibrations and air pressure/noise. The blast restrictions were 0.5 inches per second and an air overpressure of 130db. Blast designs were developed by KSC's blasting consultant, John MacGregor.

All geotechnical mapping data was tabulated in a database to evaluate results, compare to design assumptions, and modeled to assess secondary rock support. The

results were reviewed with the contractor and Engineer during monthly geotechnical meetings. See Figure 3.

The project includes four rock classes, Class I to Class IV, that are based on rock mass characteristics and the associated behaviors. Where variable ground was encountered, the dominant rock class drove round length and initial support decisions. The description and behaviors of each rock class were summarized in the project GBR, which were primarily based on the Geological Strength Index.

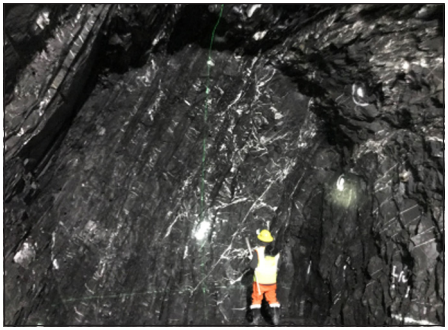


Figure 3. Mt. Merino Shale of the East Connection Tunnel

The employed geological mapping methods included both the Rock Mass Rating System and the Geological Strength Index method to determine the support type. Results were reviewed daily with project personal and summarized on a monthly basis during monthly geotechnical meetings.

TUNNEL BORING MACHINE TUNNEL

The Robbins Company was selected by KSC to provide the TBM for the project. See Figure 4 and 5. Given the project challenges of water inflows and high water pressures with fault zones, the TBM was designed with probe drilling and grouting as key features. The machine was not designed to operate in closed mode, instead the plan was to cutoff water features ahead of the machine through probing and grouting. The machine was designed to probe ahead to identify water and then grout those features ahead of the excavation.

The cutterhead was a closed face and equipped with 41 cutters and eight buckets with replaceable scrapers. The cutterhead was designed with 2 unique positions which then aligned with 16 individual drilling locations for probing and grouting holes. See Figure 6. The cutterhead could set up on position “A” and drill 8 holes and then rotate to position “B” and drill an additional 8 holes if necessary. This was an essential design parameter to permit multiple opportunities to grout off the water bearing zones.

The main bearing and sealing system were designed to withstand 35 bar of hydrostatic pressure, with five outer seals and five inner seals. The main drive system had nine 330 kw electric motors which were controlled by a high torque limit. The TBM thrust system was comprised of 32 thrust cylinders and 16 articulation cylinders. The thrust cylinders were sized to react to 30 bar of water pressure and could provide ‘exceptional’ thrust should the machine encounter squeezing ground.

The shield was comprised of three components, the forward, rear and tail. The shield was required be designed to withstand water pressures up to 30 bar, a turn radius of 1,000 ft and multiple drilling ports. The forward shield was equipped with ports to drill at 0, 4, and 7.5 degrees from the tunnel centerline. There were five dedicated dewatering ports and eight ports for bentonite injection. The rear shield housed the thrust, gripper and skew cylinders and the articulation joint with a sealing system with grease and water flushing capabilities for seal protection.

The machine was equipped with a conveyor to move the muck from the cutterhead on an open belt. The TBM conveyor moved the muck through the trailing gear to muck

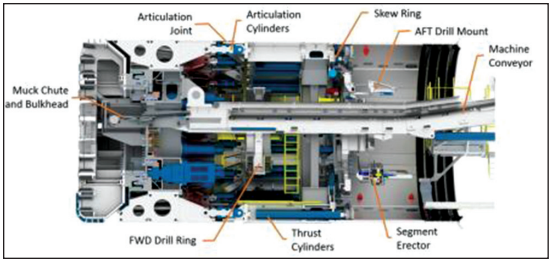
cars that transported the muck to the shaft for disposal. A unique design feature for the conveyor was its ability to be retracted. If the TBM encountered high water inflows the TBM could be sealed against the water inflow to begin grouting. The muck chute door would be retracted, the conveyor tail frame retracted, the cutterhead access doors closed, the stabilizer access doors closed, and a bulkhead sealing plate retracted. After that primary and secondary inflatable seals would then be utilized to keep water from entering the TBM. This functionality was tested underground however was never needed to be deployed against high inflows.

The dewatering system was designed to nominally control up to 800 gpm and if necessary, additional pumps could be deployed to handle up to 2,500 gpm. Water was collected on the machine in the cutterhead chamber, in the invert of the machine, at the ring build area and conveyor transfer point. The water was collected in two 10 cubic meter tanks on the trailing gear and then batch pumped to the shaft through an eight inch discharge line to a frac tank at the shaft. The purpose of the batch discharging was to minimize to collection and buildup of sediment in the discharge line. An additional 10 inch discharge line was run behind the TBM for discharging large inflows if needed. The last car of the trailing gear was equipped with a hose reel for extending the discharge line.

For probing and grouting the TBM was equipped with two drills capable of drilling through the cutterhead concurrently. Additionally, the TBM was equipped with a single aft drill deck to drill through the shield ports if needed. The forward drills were attached to a drill ring which allowed the drills to index to the 16 ports in the cutterhead. The drills utilized on the machine were Wassara down the hole (DTH) water hammers with Ripamonti drill feeds, blow out preventers and a Kamat pump. The Wassara DTH water hammers were capable of drilling longer straighter holes than traditional top hammers drills that typically provide energy into the drill string at the feed which may cause deviation at longer distances. The DTH water hammers supply high pressure water down the drill string to a piston/hammer near the point of drilling, reducing deviation. The Wassara hammers were tested at a nearby quarry with similar geology prior to equipping the TBM and demonstrated the ability to drill holes greater than 200 feet with little deviation from the drill string.

In order to provide pre-excavation grout the TBM trailing gear was equipped with two mixing/pumping stations that could be run concurrently. Each station had two independent double piston Hany plants capable of pumping up to 50 gpm of grout or grout up to 1,000 psi. Dry cement was transported to the machine using five cubic yard hoppers that were stored on the trailing gear. The cement was augered to the Hany grout plants and mixed at varying water/cement ratios by computer-automated control. The bins were designed for varying types of cement and rapid change out.

To fill the annular space between the machine overcut and the segments the project specified a two component (A+B) grout with a compressive strength of 500psi at 28-days. The project batched the A component on the surface using a Team Mixing batch plant. Both the A and B components were pumped from the surface down the shaft and thru a series of energy dissipaters at the shaft bottom into the tunnel via 2.5 inch and two inch lines to holding tanks on the TBM. The accelerator (B) was mixed with the grout in the tail shield. The tail shield grouting system was equipped with a water flushing system to remove the any gelled grout out of the lines when not in use. Thru a series of approximately 40 mix design trials, the project developed a grout that remained pumpable for approximately five days without set that when mixed with the accelerator still achieved rapid initial set and 28-day compressive strengths.



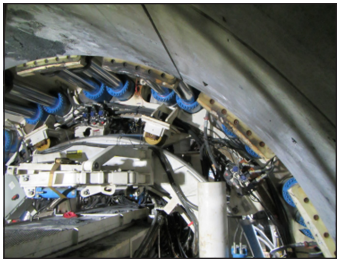


Figure 4. The Robbins TBM Section View and Looking Forward from the Tail Shield

Underground assembly of the TBM occurred at the bottom of Shaft 5B and in the Bellout. The TBM shields were assembled underground while resting on a steel cradle with bogey wheels riding on a wide-gauge rail system (Cradle). See Figure 7. The Cradle structure slid back and forth through the shaft bottom to facilitate assembly of the machine. The Cradle was designed to roll with the full weight of the TBM. Once the machine was assembled at the shaft, the Cradle carried the TBM to the end of the starter tunnel. At the end of the starter tunnel the TBM and Cradle rolled on a bridge through a launch pit. During drill and blast development, KSC enlarged the final 50 feet of excavation in order to launch create the launch pit. Once the TBM was positioned onto the bridge in the launch pit the TBM cradle was jacked up so that the bridge could be removed and then the TBM was lowered into the final position for mining.

The TBM launched from a steel jacking frame and nine false rings were erected on launch. The TBM jacking frame consisted of rectangular wide flange members with supporting rackers and a steel ring that transferred the TBM thrust cylinders to the frame. The jacking frame was designed to withstand 1,840 kips of force from the 32 thrust cylinders during launch and ring erection.

TBM EXCAVATION PROGRESS

The Robbins TBM mined approximately 12,400 feet of 20.5 feet diameter

Figure 5. Robbins Tunnel Boring Machine

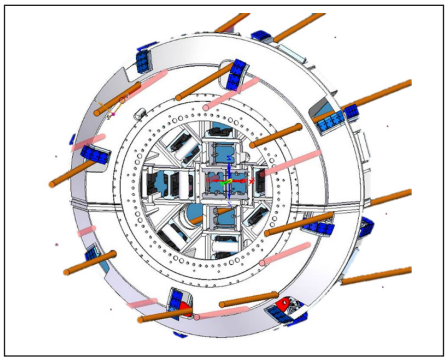


Figure 6. Cutterhead Drilling Locations

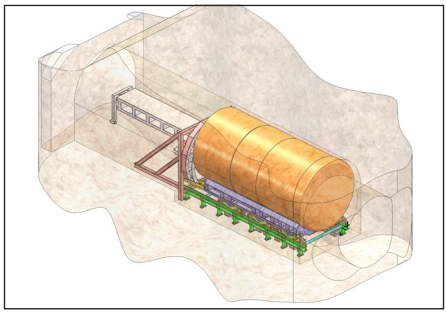


Figure 7. TBM Launch, Cradle and Jacking Frame

tunnel through the atypical shales and the dolomitic limestones (with a wide range of strengths) and occasionally faulted ground.

Rock average compressive strengths ranged from 10,000 psi in the shale formations, to 35,000 psi (with outliers close to 54,000 psi) in the dolomitic rock formations.

The project alignment was broken up into four Reaches, dependent on rock type and quality. A breakdown of each Reach by tunnel length, rock type and the anticipated ground conditions is as follows:

1. Reach 1 was 2,400 feet long in the Normanskill Shale Formation
2. Reach 2 was 3,900 feet long in the Wappinger Group Dolomite, with minor faulted ground
3. Reach 3 was 3,500 feet long in the Wappinger Group Dolomite with major water bearing, faulted ground
4. Reach 4 was 2,500 feet long in the Mt. Merino Shale Formation

The best daily footage at 90 feet per day helped the TBM achieve the best week of mining at 355 feet. The best monthly advance rate was 940 feet that occurred twice; once in the hard dolomitic rocks of Reach 2 and the other within the eastern shale unit in Reach 4 toward the end of the tunnel.

TBM cutter wear checks were routinely completed during scheduled maintenance. The TBM cutterhead included 41 cutter positions with eight 17-inch center (quad cutters), thirty-three 19-inch center face cutters, two transition and eight gauge cutters. A total of 337 new cutters were installed to replace worn ones for approximately 12,400 feet of mining a 20.5 feet diameter tunnel. The cutter change rate equates to approximately 37 feet of mining per cutter or about 500 cubic yards per cutter change.

PROBE DRILLING

The contract required two probe holes in the shale (low probability of encountered high inflow rates) and mandated four probes, one in each quadrant of the Wappinger Group (high probability of high groundwater inflow). The probe hole alignment stayed within 50 feet of center line and remained inside the easement for the tunnel. KSC employed use of downhole survey tools to confirm positions of the probe holes. The contractor and construction manager logged each probe hole. Rock core samples were collected every 500 feet through the cutterhead by contract.

As mining progressed KSC drilled probe holes as long as 400 feet. Probe drill cycles were shortened when pre-excavation grouting (PEG) was triggered due to exceedance of the threshold for groundwater inflow rate. A total of 41 probe drill cycles were completed for the 12,400 long alignment.

PRE-EXCAVATION GROUTING

The contract pre-excavation grouting (PEG) trigger rates were as follows

- 0.3 gpm/ft or 30 gpm per feature
- 0.2 gpm/ft or 20 gpm per feature

PEG materials were Type III cement and ultrafine cement, as needed. The TBM was fitted with two computer -automated grout plants made up of Hany, double double-acting piston pumps capable of producing pressure up to 1,000 psi with maximum pumping capacity of 400 cubic feet per hour (50 gpm) at a pressure of 350 psi. Two plants were available to treat two probe holes simultaneously. PEG refusal criteria was less than

0.5 cubic foot of grout injected over a two minute period (flow less than 1.87 gpm for 2 minutes) at the specified maximum injection pressure and grout consistency. Maximum volumes were mandated at 30,000 pounds per 100 feet length of grout hole.

The PEG started at 50 psi above hydrostatic pressure. The mix was thickened every 15 minutes unless pressure built up by 10 percent over 15 minutes or pumped at the maximum pressure (50 above hydrostatic) and grout flow rate decreased by 10 percent. PEG mix water /cement ratios (by weight) started at 4:1 and were reduced to 0.5:1 as needed. The neat cement admixtures included, Super P retarder. Accelerants were not allowed for PEG.

Grout packers include a pneumatic, 3.5-inch drill though packer that was used at the hole collar. A 2.5-inch diameter, disposable steel pipe bladder packer made by Ground Machinery Applications was designed with a checkvalve to prevent backflow.

Probe drilling did not encounter groundwater at PEG trigger rates for the first 26 probe drill cycles or 7,500 feet of mining. PEG trigger rates were encountered at the 27th probe dill cycle, which was 1,350 feet into Reach 3.

Five water bearing zones triggering PEG were encountered over approximately 4,500 feet. In Reach 3 and into the beginning of Reach 4. High pressure conditions were associated with three water bearing zones covering over a continuous 600 feet of tunnel in Reach 3. The initial PEG event required 10 rounds of probe drilling and PEG before the inflow rates were reduced below trigger rates. Others were accomplished in three rounds. The highest reported hydrostatic pressure was 250 psi. Subsequent measurements in the segments were 220 psi approximately 2,200 feet into Reach 3, on the eastern aspect of the high pressure zone.

A total of 162,000 pounds of Type III cement and 6.500 pounds of ultrafine was consumed for the five events of PEG.

FAULT ZONES

Monitoring TBM performance parameters, probe drilling, and observations of the rock face when probe drilling indicated the TBM encountered at least two major water bearing fault zones, a dry prominent fault in the first 1,500 of mining and several minor faults (in terms of rock quality and groundwater inflow). The major water bearing faults were encountered roughly 8,000 feet along the alignment, below the western shoreline of the Hudson River. The ground exhibited characteristics of Type II rock class but produced relatively high groundwater inflow under moderate to high hydrostatic pressures.

During probe drill events, monitoring drill cuttings, abrupt changes in rock color, and strength and texture indicated possible faulted ground. The changes in rock characteristics were immediately followed by groundwater flush flows in the range of 100 gpm. Probe drill depths in each quadrant were recorded to develop three points that would indicate the orientation of fault planes. The alignment of the tunnel could be used in relation to the fault strike and dip for modeling of wedge failure. There were no reports or observations of overbreak at the major faults. The faults are thought to be mostly healed fault shear zones, possible reactivated with normal displacement during extensional tectonic events millions of years ago. The quality of the ground did not impact TBM production but groundwater did. Overall, the forecasted pre-excavation grouting events were less than expected, but 17 rounds of PEG events were necessary over

approximately 600 feet to reduce groundwater flow from two major fault and one possible minor, water bearing faults.

GROUNDWATER DRAINAGE

Despite thorough probing requirements in front of the TBM, no groundwater was encountered at pre-excavation (PEG) grout trigger rates for the initial 7,500 feet of TBM excavation. Ultimately groundwater was encountered. The specifications included direction to handle the groundwater infiltration through the drain holes installed through precast segmental lining. KSC elected to install several drains in the precast segments to reduce hydrostatic at rock face. Measured hydrostatic pressures at the face did not exceed 300 psi, the contract trigger to initiate pressure relief drains. No relief drains were drilled throughout the excavation of the tunnel.

When groundwater inflows were encountered by the TBM, segment backfill grout washout became a challenge (shunt flows). KSC installed multiple water relief drains in the segments behind the tail shield to relieve water pressure and flow behind the segments.

The contract called for five segment rings that would address the shunt flow in the backfill grout. One was used at the starter tunnel. A mockup on the surface exposed deficiencies in the grout bags in their ability to close the gaps. Ultimately none were used. In lieu of segment rings the contractor used chemical grout rings behind the segments once the tunnel was fully excavated.

In order to address backfill grout washout behind the segments, Stage II backfill grout was pumped through the segments on the TBM to fill the voids. The stage II backfill grout was to have been triggered by two criteria, either 10% reduction below theoretical, rock weight on the conveyor belt indicating possible overbreak, and/or higher backfill grout placement pressures.

STAGE II GROUTING

Stage II grout was placed off the trailing gear. The contractor employed several methods to place Stage II grout depending on location relative to the TBM. There was a mobile plant to carry out Stage II grouting away from the TBM. The contractor set up the pre-excavation grouting plants located on the TBM to pump grout from the TBM to locations in close proximity to the back of the trailing gear. A total of eight stage II grout events were performed.

To further backfill voids and reduce tunnel seepage, chemical grout rings were placed in lieu of the segment rings. Sub-technical from Allegheny County, Pennsylvania provided technical assistance for the chemical grouting. Chemical grout was placed six times behind the segments to create three groundwater barriers and reduce groundwater inflow below contract limits, which was 0.5 gpm per foot of tunnel or approximately 60 gpm for 12,400 feet of TBM mined tunnel.

HIGH PRESSURE WATER

Final inspection of the segments before the installation of the interliner indicated a high water pressure situation existed over a portion of the tunnel. Water pressure gauges were installed on the segments and the pressures monitored. Monitoring indicated the contract baseline for maximum groundwater head of 675 feet or 292 psi was exceeded. Surface elevations capable of producing pressure in excess of 300 psi were determined to be too far away. Ground surface elevations directly above

the area of interest equated to 600 feet of groundwater head or 260 psi. The most logical source of the pressure was the existing RWBT tunnel, located approximately 1,750 feet away.

A high pressure monitoring and drainage program was initiated by the contractor with direction from the Engineer. The high pressure water was monitored over 3,500 feet of tunnel, drained through segments over a distance of 1,000 feet, and then conveyed by four inch steel pipe 9,000 feet out to Shaft 5B. The high pressure zone (HPZ) pipe was independent of the primary water utility, discharge line. HPZ inflow rates were consistent at around 150 gpm.

After several weeks of monitoring there was an increase in pressure at certain locations and a reduction in flow rate from the HPZ discharge line, indicating there was obstruction developed. Upon inspection of the HPZ piping systems revealed excessive efflorescence and calcification. The 4-inch steel pipe had to be replaced with plastic, PVC, 'yelomine' pipe. Efflorescence occurred in the Yelomine pipe also, but it accumulated at a slower rate. Ultimately the calcification in the HPZ drain network required maintenance every three to six months. See Figures 8 and 9.

The designer determined the high pressure zone required continuous drainage until either the final cast in place lining was complete or the existing RWBT tunnel could be unwatered during the future connection. The present plan is to grout off the HPZ inflows after the placement of the cast in place lining.

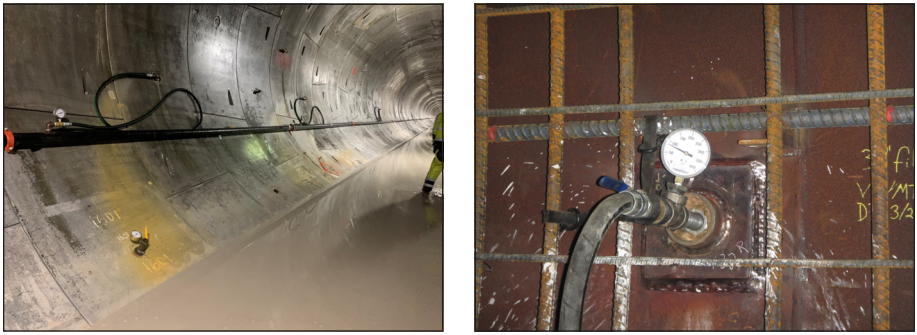


Figure 8. High Pressure Zone Drains in the Segmental and Interliner

Figure 9. High Pressure Zone Steel Discharge Line Efflorescence and Calcification

In order to allow the HPZ water to drain while installing both the interliner and the final cast in place lining the team designed a network of 23 HPZ drains over about 1,000 feet spaced approximately 40 feet apart. These penetrations through the interliner were fitted with drain/grout ports. A sequence was developed to allow for constant drainage while the interliner was installed through the HPZ. A second sequence was developed to bring the cast in place final lining through the HPZ. The monitoring program continued on a weekly to monthly basis.

SEGMENTS

Within the TBM tail shield a precast concrete segmental lining was built and installed as the initial tunnel lining and ground support. The precast concrete segmental lining is a 6-piece tapered universal ring, with rebar reinforcement that is bolted and gasketed. The precast concrete segments are 13 inches thick with 9,000 psi compressive strength concrete with an outside diameter of 20 feet 4 inch and 5 foot nominal length. The segments were fabricated by CSI Tunnel Systems in Londonderry, NH using a carousel system. See Figure 10. The segments utilize 12 steel radial bolts supplied by Sofrasar Anixter and 16 polyamide dowels supplied by Optimas. The segments also included a five inch thread diameter by 7.5 inch long lifting socket by Optimas. The lifting socket could be drilled out to be used for grouting or as a drain. A unique feature of the segments was the gasket; the one inch thick, EPDM gasket provided by Datwyler was tested to withstand pressures of up to 35 bar for a period of 72 hours and in fact withheld over 50 bar.

The segments were manufactured in New Hampshire and transported to the site by tractor-trailer. The segments were pushed onto the shaft cover using a rail mounted dolly and picked up by a mechanical clamp hanging from the headframe hoist. The segments were lowered down the shaft in stacks of three and were transported to the TBM using rail mounted segment cars. See Figure 11. The segments were unloaded by hydraulic cylinders which raised the segments from the rail mounted dollies and set them onto the trailing gear. Individual segments were transported to the ring building area using an Acimex segment trolley where the erector built the segment ring. Due to concerns of high volumes of water in the ring build area the TBM was equipped with a mechanical type erector rather than a vacuum erector. The erector had a screw type connection into the segment insert and hydraulic stabilizers and was operated by wireless remote operation. The segment erector had dual functionality where it could also position the aft drill on the TBM for drilling through the shield ports. See Figure 12.

Figure 10. Segment Fabrication

Figure 11. Segment Transportation, Dollies, Segment Trolley and Ring Build

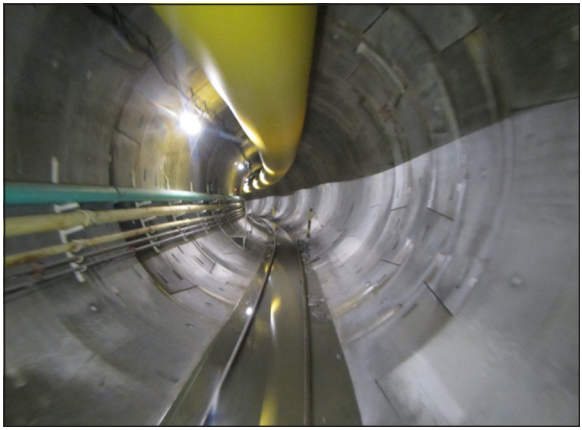


Figure 12. Segment Lined Tunnel

INTERLINER INSTALLATION

The interliner was fabricated from scratch from Russian steel ingots. The ingots were rolled into plates in Denmark at DanSteel NLMK. The plates were transported across the Atlantic Ocean in a ship to the fabrication yards in Louisiana. The plates were trimmed into 10 foot wide by 50-foot-long plates. Each 10 by 50-foot plate was rolled and then welded into a 16-foot diameter circle. Four of these rolled plates were welded together to form one section of pipe 40 feet long and 16 feet in diameter. Two hundred and thirty sections of pipe were fabricated. The pipe was braced internally with two rectangular braces (versus the more typical diagonal stulling) made of angle iron, each ten feet from the end. Each section of pipe weighed 84,000 pounds each.

Per the contract three penetrations were made in each pipe, one in the invert and two in the arch to aid the low-density cellular concrete backfill to be placement between the interliner and the tunnel segments. Each penetration had an external backing plate. The contractor added six additional penetrations, four in the arch for float pins and two centered on springline for lifting and pivoting the pipe while lowering into the tunnel. The end of each section of pipe was beveled to provide the correct geometry for the internal welds that create one continuous pipe. Twenty-two pipes were mitered such that these straight pieces of pipe could be installed in the curved section of tunnel.

Precision fabrication is the key to a successful pipe assembly in the tunnel. A fit up in Louisiana, where two sections of pipe were staged next to each other, simulated the installation. Dean's Welding, the subcontractor that would weld the pipe together, attended the fit up to assess the geometry and fit-up challenges ahead in the tunnel.

Figure 13. Interliner Transport

The pipe was then loaded onto a barge and transported from Louisiana around the Florida Keys, north up the east coast to the Hudson River and up the Hudson River to Newburgh NY where the interliner was temporarily stored near the offloading dock. Subsequently each section of pipe was transported from the temporary storage site seven miles to site, three to four per night. See Figure 13. Due to the height of the load, specialty Faymonville trailers were used to transport the sections which provided a ground clearance of just a couple of inches. Even with the low trailer to accommodate the over-height loads there were approximately 83 utilities that were raised or relocated in the seven miles between the temporary storage yard and the Shaft 5B site.

Once the interliner sections arrived on site they were fit out with reinforcement bars and an elevated rail system. This was done on the surface prior to being lowered down the shaft to the tunnel. The rebar consists of number five longitudinal reinforcement at ten-inch centers and number nine hoop reinforcement at eight-inch centers. In order to secure the reinforcement bar to the interliner for transportation vertically down the shaft, there were additional setup number ten bars welded to angle iron which were in turn welded to the interliner. The setup bars were welded to the gusset plates that served as the stulling to provide a positive connection to the steel interliner.

Additionally, on the surface an elevated rail system was installed in each section of interliner pipe. The purpose of the elevated rail in the interliner was to provide access for welding, splicing, and grouting gantries in the tunnel. The elevated rail system consisted of 40-pound rail on wooden ties. The ties were elevated above the rebar in the invert with box tubing. This permitted the rebar to be tied in a full 360 degrees prior to lowering into the tunnel.

The interliner sections were fabricated in 40 foot lengths and were to be lowered down a 30 foot diameter shaft so they must be rotated to hang vertically to fit down the shaft. With the reinforcing bars and the rail, each section weighed approximately 110,000 pounds when lowered 850 feet down to the tunnel.

To hoist each section of interliner the two hoist drums were engaged to run simultaneously. Each drum's wire rope was rigged to a spreader beam. The spreader beam was rigged to a hole at springline of the interliner to allow rotating the interliner section from horizontal to vertical at the top of the shaft and from vertical to horizontal at the bottom of the shaft. The holes at springline were placed eight inches closer to one end of the pipe to make it to hang vertically. The connection through each springline hole was made using a swiveling Crosby handling hoist ring which acted as a trunnion.

To control the rotation of the interliner sections there were 2 winches mounted to the spreader beam. The winches were connected to each end of the interliner pipe and were run in tandem to rotate the interliner vertically or horizontally. See Figure 14.

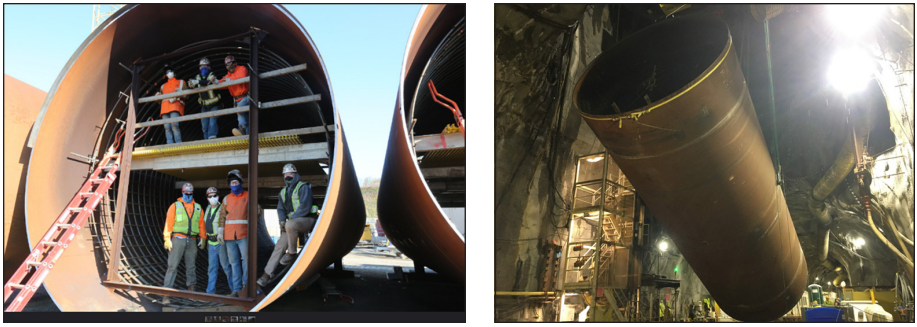


Figure 14. Interliner Section Fit Out and Lowering into the Tunnel

Once at the shaft bottom the pipe was trammed into the tunnel using a system of four dollies, pushed by a 35-ton locomotive. The interliner transport dollies ran on rail (previously used for running muck trains) using specially designed low profile high-capacity rail wheels. The line and grade of each interliner section was adjusted using air bags that were built into the dollies and then tack welded to the previously placed interliner section. The interliner sections were then fully welded from the gantries travelling on the elevated rail.

LOW DENSITY CELLULAR CONCRETE

The interstitial space between the first segmental lining and the second interliner pipe lining was filled with low density cellular concrete backfill (LDCC). By specification, the cellular concrete had a density greater than that of water (68 pcf) and a compressive strength of 750 psi at 56-days. The placement of the cellular concrete backfill was performed by Pacific International Grout in conjunction with KSC. Initially the LDCC was placed from Shaft 6B concurrent with the installation of the interliner from Shaft 5B. Upon completion of the interliner installation the LDCC placement was switched to Shaft 5B. Three full tunnel height bulkheads were constructed along the full alignment of the interliner: one at each end of the interliner and one at the mid-point. At Shaft 6B grout was batched on the surface and pumped to a secondary plant at the bottom of the shaft where the foaming agent was introduced and the grout was then pumped to a maximum distance of about 4,900 feet. At Shaft 5B, the grout was batched on the surface, pumped to the bottom of the shaft, re-pumped to the edge of the interliner (approximately 2,600 ft) where the foam was introduced and then the LDCC was pumped another 5,320 feet.

The LDCC was not placed fully around the interliner; a cap space was left in the crown of the interliner to facilitate draining water the non-high-pressure water. The cap space void was required because the interliner cannot withstand external hydrostatic pressure without the cast in place lining.

CAST IN PLACE LINING

Concurrent with the placing of the LDCC from Shaft 5B, KSC started the third and final tunnel lining from Shaft 6B. The third lining is a 14 foot diameter reinforced cast-in-place concrete. The concrete has a minimum compressive strength of 4,500 psi and the rebar configuration changes along the alignment of the tunnel. The cast in place lining thickness within the interliner is 12 inches and beyond the extents of the interliner is 24 inches. Beyond the extents of the interliner, rebar cages were prefabricated on the surface, lowered to the bottom of the shaft and transported into the tunnel on a

rail dolly pushed by a 6 ton or 4 ton loci. The rebar cages were 20 feet long and were spliced in place. Prefabricating the rebar cages allowed for much of the rebar tying work to be done concurrent with other tunnel lining operations.

After the installation of the rebar cages a 14 foot diameter steel form fabricated by Everest was assembled and the formwork was erected. The formwork is 180 feet of full round forms that is comprised of five foot long panels. The panels are moved in sections that are 30 feet long by a hydraulic needle beam carrier. The forms are split between arch and invert and supported by invert, side and crown float pins.

Approximately half of the concrete is being pumped from Shaft 6B and half from Shaft 5B. The concrete is being dropped (700 feet at Shaft 6B and 850 feet at Shaft 5B) thru an eight inch steel pipe At the bottom of the shaft after the energy dissipater the concrete is conveyed to a 12 cubic yard Maxcrete remix. The Maxcrete remix is supported on a steel platform over the Putzmeister 14000. The Putzmeister 14000 pumped to the point of placement. After pumping approximately 5,400 linear feet from Shaft 6B a second pump Putzmeister 1409 was added in the tunnel to complete the concrete lining to the midway point. A trial pump mockup was previously performed on the surface to ensure pumping these distances was feasible.

CONCLUSION

The Bypass Tunnel has been successfully excavated, segments erected, the interliner installed and welded together, a cast in place lining completed and high-pressure water has been channeled to allow the completion of the tunnel portion of this contract. Work on the shafts remains as does the connection of the Bypass Tunnel to the RWBT.