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Ground Freezing Experience on the East Side Access Northern Boulevard Crossing, New York

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Abstract

A brief review is given of ground freezing technology as a means of providing groundwater cut-off and temporary structural support in weak ground for transportation tunnelling and shaft-sinking operations. Then detailed coverage of a particular case history is reported in variable soils in Queens, New York. As part of the upgrading of rail access from Long Island into Manhattan, a tunnel was required that necessitated a frozen arch structure under a 'live' roadway and rail lines. The installation, development and performance of the frozen zone are described. At two small locations groundwater continued to flow through the line of the frozen arch, the heat in the newly arrived groundwater being greater than the capacity of the ground freezing technique to remove that heat, thereby preventing final 'closure' by frozen soil at those points. The remedial works to overcome these difficulties are described. Some conclusions are drawn concerning the reasons when and why these localized unfrozen 'windows' can be anticipated in freeze zones and about the requirements for remedial solutions.

Keywords: *Tunnels & tunnelling, Groundwater, Temporary Works*

1 GROUND FREEZING BACKGROUND

1.2 Application to tunnel and shaft construction

Ground freezing is a well-established technique to aid the installation, amongst other construction projects, of transportation tunnels. Mussche and Waddington (1946) reference five early projects undertaken between 1892 and 1935 that required ground freezing, and three of these were metro or vehicular tunnels in Paris, Antwerp, and Moscow. Although there have been many practical advances since those early ground freezing projects were undertaken, the principles remain the same today as when performed over 100 years ago. Holes are drilled on the line of a circular (or maybe ellipsoid) pattern surrounding the shaft or tunnel alignment. Each is equipped with closed-end steel pipes through which a chilled brine is circulated (Figure 1).

In this manner, a frozen cylinder is formed when the ground freezes between adjacent pipes. Figure 2 illustrates this concept for a vertical shaft, but the principle is the same for a horizontal tunnel. Excavation can then take place inside a self-supporting, groundwater-excluding structure in mostly unfrozen soil. Ground freezing in such peripheral freeze applications performs the dual functions of water cut-off and soil support, eliminating sheeting and internal bracing.

Figure 1 A typical brine re-circulation set-up (Powers et al., 2007).

Figure 2 A typical shaft freezing set-up (Powers et al., 2007).

Typically, a row of freeze pipes is placed in the soil on approximately 0.75 to 1.5 m centres (Jones, 1980) and heat energy is removed through them in a process remarkably analogous to pumping groundwater from wells (Powers, et al., 2007). The zone of freezing (as represented by isotherms) moves out from the freeze pipes with time, similar to groundwater contours around a pumping well. When the soil temperature reaches 0°C, assuming fresh groundwater, temperature lowering pauses while the latent heat of fusion is removed and water in the soil pores turns into ice. Then further cooling proceeds.

The frozen soil first forms in the shape of cylinders surrounding the freeze pipes. As the cylinders of frozen soil gradually enlarge, they intersect and form a continuous wall (Figure 3). "Closure" is said to occur upon complete interlocking of the frozen cylinders – which may, typically, take 6 weeks (as shown in Figure 3) – and, with further heat extraction, the thickness of the frozen wall will increase so as to achieve structural thickness (as shown at 12 weeks in Figure 3).

Figure 3 Typical and idealized development of a peripheral freeze (Powers et al., 2007).

1.2 Moving Groundwater and Freeze Closure

The body of knowledge concerning the interaction between ground freezing and moving groundwater is small. Where published, few details are provided. Moving groundwater is generally recognized as the most adverse condition for ground freezing, resulting in freeze formation (closure) difficulties and, if undetected, freeze failures. Failure of frozen shafts and excavations has occurred due to "windows" in frozen walls which did not close due to excessive groundwater velocities. Several examples have been documented (Shuster, 2000; Sopko and Braun, 2000; Sopko and Jatczak, 1999; Schmall, et al., 2007).

Movement of groundwater during freeze formation puts an extra heat load on the freeze pipes and the refrigeration plant, preventing or requiring more time to achieve "closure" and a continuous frozen wall. Where the groundwater velocity is high, groundwater flowing past a single freeze pipe transfers the cooling effect downstream which, in plan view, results in an egg-shaped formation of frozen soil around the pipe, growing more slowly on the upstream side (Hashemi and Sliepcevich, 1968). Where the groundwater velocity is excessive, groundwater flowing past a single freeze pipe introduces such a large amount of heat energy that freezing is impossible.

The critical pre-freezing groundwater velocity for typical brine freezes is generally recognized to be between 1 and 2 m per day (Corwin et al., 1999; Sanger, 1968; Andersland and Ladanyi, 2004; Grant and Iskander, 1997; Schultz et al., 2008). This threshold value of groundwater velocity, as stated by these authorities, depends on several factors, namely the freeze pipe spacing, freeze pipe radius, coolant temperature, groundwater temperature, and the thermal conductivity of the soil.

Even in an environment where groundwater is moving, the egg-shaped columns can grow and merge. However, as the columns grow, the cross-sectional area available for groundwater flow decreases. Thus, as closure approaches, the flow becomes concentrated through the remaining openings, with consequent increase in flow velocity as the cross-sectional area available for flow decreases due to ice formation. The fewer gaps (or "windows") that remain, and the greater the concentration of flow, the higher the velocity will be through these windows, potentially encouraging loss of fines thereby increasing the permeability, and requiring an increasing freezing effort to achieve closure due to the greater incoming heat energy carried in the flowing water. With a larger diameter shaft, the concentration of flow through a window will be greater, since a larger diameter shaft will provide a greater "damming effect" to the natural flow of groundwater. Thus problems of closure can be expected to be greatest for large shafts.

Ultimately, in the common application of a peripheral freeze around the location of a shaft sinking, perhaps only one "entrance" gap will remain on the upstream side of the shaft, with a corresponding "exit" gap at the downstream side of the shaft site, and the freezing effort applied through the array of freeze pipes may not be sufficient to overcome the additional heat loading and finally close the gaps.

2 EAST SIDE ACCESS PROJECT

The East Side Access project will connect the Long Island Rail Road's (LIRR) main and Port Washington lines in the New York borough of Queens to a new LIRR terminal beneath Grand Central Terminal on the east side of Manhattan Island. As early as 1969, New York's Metropolitan Transportation Authority commenced works to link rail lines in the eastern suburbs of the city (on Long Island) with the Grand Central Terminal. These early works incorporated tubes, earmarked for the future LIRR extension, into a submerged crossing under New York's East River. These tubes were installed at the time of construction of the 63rd Street Subway line. By 2001, tunnelling for the future link had extended eastwards to the west side of the Northern Boulevard in the borough of Queens (Figure 4). At the time of drafting this paper, tunnelling works commenced in 2006 (shown dotted in Figure 4) in Queens are complete with structural and track work still underway so that completion of the whole scheme is not expected until at least 2021.

Please note: much of the data presented in this paper has been drawn from project records in which, in accordance with customary US practice, lengths are dimensioned in feet, rather than in metres. These are not changed herein but can be converted to metres by multiplying by a factor of 0.3048.

Figure 4 Route of Long Island Rail Extension from Queens into Manhattan

2.1 Northern Boulevard Crossing (NBX)

As part of the scheme in Queens, tunnelling was required that would cross under both the Northern Boulevard road, which forms a major thoroughfare in for road and rail Queens (Figure 4). At the point where the new route must cross under the Northern Boulevard the three tracks of the Astoria BMT subway line are carried above the road on an elevated structure supported on piled columns that will be cut by the roof of the new tunnel (thus requiring temporary underpinning until they can be supported from the structural lining of the tunnel) whilst the five tracks of the Northern Boulevard subway line run in their own tunnel between the new tunnel's roof and the road surface (Figure 5).

To achieve this construction, a short length of connector tunnel, 38m long, was planned using the Sequential Excavation Method (SEM) with sprayed concrete-supported tunnelling between two, 25-m deep shafts on either side of the boulevard itself (Figures 5 & 6) (Rice, 2012; Clark et al., 2013). This tunnel would be excavated inside an arch of frozen soil (formed by horizontal freeze pipes over the tunnel alignment) (Figures 7 & 8). The project location is surrounded by several recent deep subway excavations, all of which were constructed in watertight excavations, with perimeter cut-off walls consisting mostly of slurry walls with some jet grouting.

2.2 Geological and Hydrogeological Conditions at NBX

Based on preceding site investigations, the solid geology at the site was understood to comprise Ordovician/Cambrian age metamorphic grey fine-coarse-grained, unweathered to moderately weathered, strong to very strong Gneiss (Rice, 2012; Clark et al., 2013). This layer provides the springing points of the frozen arch, being of low permeability. The bedrock was known to be covered by glacial, interglacial and post glacial deposits of Pleistocene age and it is this layer that required freezing.

Figure 5 Long section of finished tunnel (adapted from Rice, 2012)

Figure 6 Elements of NBX

Notes: 1 = Viaduct structure carrying Astoria BMT Subway Line; 2 = Northern Boulevard; 3 = Northern Boulevard Subway line; 4 = Tunnel cut in sections within frozen arch on top of rock.

Figure 7 Cross section of tunnel and frozen arch (adapted from Rice, 2012)

Figure 8 Image of installation of freeze tubes, from south side access pit, to form frozen arch

The tunnel invert was to be situated approximately 17 m below the water table, such that tunnel alignment would be passing through the glacial deposits comprising silts, sands, boulders, till and the bedrock. The most porous stratum at the depth of the intended arch was a sand/gravel glacial till for which grading curves are provided in Figure 9. In many of the soil samples obtained during ground investigation it was found to be both clean of fines and of a coarse overall grading. Rice (2012) describes it as consisting of a heterogeneous mixture of brown, grey to reddish grey/brown, medium dense to very dense sands with silts (generally 5–36%), gravel, and boulders (USCS classification SM, SW-SM and SM/GW).

Figure 9 Grain size distribution curves for the glacial till stratum at the NBX site

The project geotechnical report indicated that "intermixed with the till are materials that were deposited in melt-water channels, as outwash within and under the glacier as a result of re-working of glacial till." The highest hydraulic conductivity indicated in the contract documents was 4 x 10^{-4} m/s, based on aquifer pumping tests. Typically this technique is the most representative of in situ hydraulic conductivity, although generally representative of an average hydraulic conductivity (Powers, et al., 2007). The highest calculated hydraulic conductivity from grain size distribution curves (Figure 9) was 1.5 x 10^{-3} m/s using the method of Kozeny-Carman, after Carrier (2003), considering a porosity of 0.4.

3 GROUND FREEZING SCHEME & BEHAVIOUR

The freeze could be compared to a shaft lying horizontally, but with one open side at the bottom; i.e. the bedrock surface. The freeze tubes extended from the installation pit under the road and rail lines into an existing slurry wall on the far side (Figures 5 & 6). It was previously anticipated that closure (i.e. full freezing) of the arch might be difficult to verify.

In a vertical shaft, the enclosed water level rises when the annular ring becomes fully frozen as the expansion of the water (which is now fully enclosed) between 4° and 0°C can no longer be dealt with by drainage as it was when holes still existed in the freeze wall. In the horizontal NBX project, a rise in head inside the arch might be difficult to distinguish if there was leakage into or out of the rock. Also, given that the installation pit would be flooded once freezing started, the expansion might not be noticed given the volume of connected water in the pit.

The design called for 43 freeze pipes, but a total of 52 freeze pipes were installed. Seven additional pipes were installed to close excessively wide gaps resulting from borehole deviation. Two additional pipes were installed due to poor rock conditions encountered at the south side of the arch. The final spacing between pipes averaged 2.9m. Once installed, brine was pumped around these tubes; its temperature dropping slowly as the heat was extracted from the arch. Figure 10 gives a summary of the timeline events. A finite element thermal model of the as-built freeze pipe configuration indicated closure should be expected at Day 37. Although temperature monitor pipes indicated that there was sufficient freeze growth and overlap of the freeze columns, there was no pressure increase observed in the piezometers installed inside the arch.

Figure 10 Timeline of events at NBX freeze project.

On Day 37, a marked difference between interior and exterior water levels was discernible; the water levels on the inside of the arch dropped to approximately 2m lower than exterior water levels (Figure 11). Prior to Day 36, water levels inside and outside of the arch were approximately equal (Figure 11).

At Day 62, after 25 days of this sustained water level difference, and 25 days beyond the anticipated time to closure, the interior of the arch was pumped down, with several horizontally installed wells (within the zone eventually to be excavated) used as observation wells to verify if closure had been achieved.

The Day 62 pump-down test (Figure 12) revealed several significant facts. At the end of the pump-down test, approximately 91 m^3 of water was pumped and the water level was only lowered to the top of the (inside of the) arch. No drainage of the soil inside of the arch had occurred. If the arch was closed, and the specific yield of the ground was 15%, a pumped volume of 91 m^3 should have resulted in approximately 2.3 m of groundwater lowering below the top of the (inside of the) arch. It was therefore concluded that one or more unfrozen 'windows' were present in the freeze arch allowing the unfrozen centre to be supplied with ground water from outside the frozen arch.

Figure 11 NBX site water level history.

With a constant pumping rate, water levels within the arch were observed to rise over a period of one to two days. This suggested that the pumping of water aggravated the condition, resulting in a gradual opening of the window(s) in the freeze due to erosion of the frozen ground by the continued flow through the window. When the pumping from within the arch ceased, an almost instantaneous rebound of water levels was observed (Figure 12). The pressure rise (groundwater level recovery) occurred at a parabolic rate, confirming that the interior area was being recharged from an outside source.

Figure 12 The Day 62 pump-down test at the NBX site.

For these reasons, the pump-down test positively indicated that the freeze was not closed. All of the horizontal freeze pipes were subsequently profiled by inserting thermocouples into specially drilled holes and into the freeze tubes, revealing two very warm spots along the south side of the arch that spanned between several freeze pipes (Figures 13 and 14). Past experience of such, relatively warm spots indicated that there must be windows in the freeze. It should be noted that one of the windows indicated temperatures above 0°C. The brine was cooled down to a relatively cold -32°C by Day 21, precluding lack of system cooling capacity as a potential explanation.

Based on the groundwater pressure differential between the inside of the arch and outside of the arch, the windows were both “entrance” windows. Groundwater leakage into the rock pit excavated for the installation of the deep freeze pipes on the south side of the arch (Figures 8 and 15) was the only discernible exit point. Variation in the leakage flow rate to the south pit based on different flow and pressure conditions supported that this was the exit window. This leakage occurred due to a poor slurry wall / rock key-in constructed under a previous contract. Prior to any remedial action, this leakage was approximately 12 l/min and was measured colder than the ambient groundwater temperature, an indication of contact with or proximity to the frozen ground.

Figure 13 Locations of the warm spots revealed by temperature profiling.

Note: the freeze tubes are shown schematically; their equally-spaced perfect parallelism shows the intended arrangement, not that actually achieved (c.f. Fig. 17)

Figure 14 Isotherms of windows and position of grout pipes.

Figure 15 Rock pit excavated at the south side of the arch to permit drilling of the lowermost freeze pipes.

Precise groundwater level data inside and outside of the frozen arch immediately adjacent to the location of the window(s) was, for the most part, unobtainable because of the location of the windows and the complications associated with installing new piezometers beneath the subway. Piezometers were generally located at the corners of the frozen arch where they could be installed from outside of the busy roadway, the subway structure, and the arch. After the windows were located on the south side of the arch, it was apparent that additional water level data was needed in the vicinity. As soon as possible, an additional piezometer was installed in the till so that there were till piezometers on the south-east and south-west corners. The differential between the two piezometers was measured to be 0.45 m, presumably due to leakage beneath the slurry walls. The new piezometer was very quickly grouted up during the remedial grouting work that followed.

4 REMEDIAL ACTION

4.1 Potential Remedial Measures Reviewed to Overcome Window Conditions

There are two potential measures that, typically, can be implemented to correct the “window” condition should formation and closure difficulties be observed. These measures include:

- the reduction of ground hydraulic conductivity by various kinds of grouting, or
- increasing the freezing effort by adding freeze pipes and/or the use of an alternate freezing agent such as liquid nitrogen.

An example of the importance of this problem is given by the Providence Combined Sewer Overflow tunnel project in Providence, Rhode Island (Kaplin, et al, 2009). There the successful implementation of corrective measures required months of additional work, with a total direct cost of several million US dollars, and resulted in a delay to the overall project (Schmall, et al, 2007). Similarly, on an environmental freeze application for a contaminated soil excavation and removal project in upstate New York, approximately one million US dollars was spent on liquid nitrogen and grout injection (Schmall, et al, 2007).

Where an excessive groundwater velocity is suspected, periodic temperature profiling of the freeze pipes is performed by measuring the static and stabilized brine temperature approximately every 0.5 m within each of the freeze pipes. Anomalous warm spots may be an indication of a window in the freeze wall. With typical freeze pipe spacing (on the order of one meter), temperature profiles can provide a very detailed “snapshot” view of the perimeter ground temperatures and the progress of the freeze formation. In some cases, the warm spots are well defined and the location of the window can be precisely located. In most cases, however, the location of the window is not obvious. Where the window is well defined, the grouting or additional freezing effort can be applied with some degree of precision. Where the window is not well defined, the grouting or additional freezing effort must be applied to a broader area with a “shotgun” type of approach.

The temperature profiling data is obtained from within the freeze pipes themselves. At the time a potential closure problem becomes evident, the frozen wall will typically be relatively thick, on the order of 2 to 3 m, and access to the location of the window (e.g. by boreholes for purposes of grouting) is limited by the presence of frozen ground. So as not to encounter frozen ground in the drilling process, which creates problems for the advancement of the drill tools and the return of cuttings, new boreholes must be drilled at least 3 m away from the alignment of the freeze pipes (Figure 16). Consequently, the new boreholes will be drilled off the alignment of the freeze pipes where the data was acquired and thus there is no truly precise application of additional measures. It is partially for this reason that the use of alternate freezing agents, i.e. liquid nitrogen, for closing windows has proven to be very costly and not always guaranteed to fix the problem. Although liquid nitrogen has been cited to be effective under groundwater velocities of up to 6 m per day (Harris, 1995), or even higher (Shuster, 1972), the liquid nitrogen must be applied over a large enough area to encompass the high velocity groundwater flow regime (Figure 16). Although significantly colder than brine, the liquid nitrogen freeze must still propagate itself thermally through the formation, a process that is highly time-dependent and, albeit to a lesser degree, still susceptible to the adverse effects of high groundwater velocities.

Permeation grouting has, in many field cases, been the most effective technique to control groundwater flow and facilitate closure. The soils in which naturally occurring excessive groundwater velocities occur are typically formed of coarse sand and gravel, and amenable to permeation grouting with low cost conventional viscous grouts. In such a subsurface environment, the grout will find the “path of least resistance”, which often corresponds to the zone of high groundwater flow (Figure 16).

Figure 16 Altering ground conditions over a large area outside a window to encompass the high velocity groundwater flow regime.

The alignment of freeze pipes is critical to satisfactory performance of the ground freezing system. The formation time required is directly related to the spacing between pipes (Sanger, 1968). If the boreholes deviate too much, excessively large gaps can occur. Additional freeze pipes may need to be installed to close an excessively wide gap. The accuracy of the pipe installation is determined by the tolerances of the design with respect to uniformity of the ice wall growth and the time to complete the freeze formation (Harris, 1995). Powers et al. (2007) indicate that a 1% borehole deviation is considered very good for even the best drilling techniques in the best ground conditions.

4.2 Day 86-93 Action

On the basis of the review, on Day 86, tube-a-manchette (TAM) grout pipes were installed outside of the arch (Figure 17) to provide grouting access to the two windows. Clean gravelly soil was observed immediately above the rock on the first grout pipe drilled and 4,350 l of bentonite-cement grout was required to fill the annular space of the tube-a-manchette pipe due to loss of grout into the formation. A loss of this magnitude is indicative of clean, coarse, highly permeable ground.

One hundred thousand litres of bentonite-cement grout was injected through the exterior grout pipes on Days 91-93 but this grouting had no discernible effect on the windows, as determined with subsequent temperature profiling of the relevant pipes. This grouting had taken place under the prevailing water level conditions – i.e. with lower levels inside the arch than outside. This appeared to show that the rate of flow of water through the arch wall was sufficient, even in conditions modified by the grout, to bring sufficient heat to the window at a rate that prevented freezing.

4.3 Day 97-120 Action

To address this, starting on Day 97, grouting was repeated but with water levels inside and outside of the arch as near as possible equalized by actively recharging the interior of the arch by three horizontal wells (DW2 on Figure 17). The first week of equalization was approached cautiously with only partial pressure equalization. The pressure in two of the three interior horizontal wells (DW-1 and DW-3) was monitored closely so that the recharge injection could be periodically tuned to maintain the water pressure under the arch within 0.5 m of the exterior water pressure. This was as precise as the pressure regulators and valves available at the time would permit. The closest window to the excavation closed shortly following equalization of water levels, as determined from subsequent temperature profiling.

Concurrent with the equalization of the water levels, a total of 105,000 l of bentonite-cement grout was pumped until heave of the overlying subway structure was observed. At that time, grouting continued with several lower-viscosity sodium silicate-based grouts. A total of 125,000 l of sodium silicate grout was injected over a one week period. The only indications of the whereabouts of the grout were the heave measurements from within the overlying subway structure, which were greatest immediately overlying the window.

Figure 17 Sectional view of the NBX frozen arch showing freeze pipes, their surveys and interior wells.

Note: Each enumerated dot represents a freeze tube that is intended to run parallel to the tunnel. The as-built hole survey is represented by the coloured line. Freeze tubes are represented in red, interior wells are dark blue, combination heat / compensation grout pipes are yellow, compensation grout pipes are orange, and void pre-grouting holes are green. The orange and turquoise lines on the left represent the grout injection holes installed during attempts to close the windows in the freeze arch.

During sodium silicate grouting, greater efforts were made to equalize water levels more precisely between the inside and outside of the arch. The target equalization pressure was the average pressure observed between the south-east and south-west till piezometers (before the south-east piezometer was grouted up). Complicating this endeavour, it was repeatedly observed that water pressures within the arch were influenced by overhead subway activity. Pressures would rise typically on the order of 10 cm when a train would pass. With the peak morning and evening commuter traffic, with more trains active within the overhead five-track structure, water levels could rise as much as half a meter. Several mechanical devices were installed to adjust the recharge injection pressure to compensate for the fluctuations created by the train traffic. Further profiling data suggested that the freeze closed following the week of sodium silicate grouting and a weekend of more carefully controlled equalization and less commuter train traffic (i.e. less train-induced pressure fluctuations). There was no way to distinguish whether closure occurred due to grouting, more precise equalization of water levels, or a combination of the two.

Temperature profiling at Day 118 showed a very pronounced 4°C temperature drop at the previously warmest window, indicating closure. This was further confirmed on Day 119 when the water supply feed to the recharging equalization system was inadvertently shut off and a rapid water level drop was observed in all the observation points beneath the arch (Figure 18). The water level response upon ceasing the injection was significantly different than during the pump-down test of Day 62.

Figure 18 Rapid water level drop beneath the arch upon inadvertent shut-off of the recharge.

Although the rapid water level drops beneath the arch suggested that closure had been achieved, the recharging was continued so as not to subject the newly closed window to the groundwater pressure differential. Additionally, at the request of the project construction manager, grout pipes were installed inside of the arch and grouting was continued. Upon grouting of the first interior grout pipe, an almost immediate return of undiluted grout was observed at the south pit, providing further confirmation that the leakage into the south pit was from beneath the arch.

All freeze pipes were profiled again several days later (Days 132 to 136), temperatures at the window locations continued to drop, and the ground beneath the arch was subsequently drained and tunnelling proceeded (Figure 19). Temperature profiling of freeze pipes 39 to 42 at various dates is presented in Figure 20. The location of these pipes are shown in Figure 17.

Figure 19 Completed tunnel.

Note the frozen ground around the external periphery

Figure 20 Temperature profiling of the Freeze tubes 39-42 in the windows at various dates.

5 ANALYSIS AND DISCUSSION

Excavation of the tunnel did not occur in the vicinity of the windows, so physical examination of the soils was not possible. The greatest indication of the permeability of the soil at the problem location was the fact that 4,350 l of grout was required to fill the annular space of the TAM pipe due to loss of grout into the formation. This suggested clean, coarse, highly permeable ground existed in the window areas with a hydraulic conductivity greater than 1 x 10^{-3} m/s, as per Herndon and Lenahan (1976), Caron (1982), and Xanthakos et al. (1994).

Flow measurements and groundwater level differentials at various stages of the work provide insight into the nature of the window at the Northern Boulevard Crossing. These water levels and flow measurements are shown in Figure 21. The relevant data from several different phases of the work is as follows:

1. On Day 61, prior to the drain-down test performed on Day 62, leakage into the south side rock pit was measured at 11.4 l/min, and the groundwater pressure was 3.9m lower on the inside relative to the outside of the arch. It was deduced that the lower water pressure inside the arch was linked to some amount of leakage away from inside the arch. It was suspected, but not proven, that the leakage into the rock pit was for the most part from within the arch. On this basis, the rock pit inflow is assumed to equal the window flow.
2. The pump-down test, which commenced on Day 62, resulted in a maximum groundwater differential of 11.9 m, with a pumping plus pit leakage total flow rate of 50.4 l/min. As the pump-down test proceeded, and moving groundwater (presumably) eroded the window further, the water level rose inside the arch to a pressure differential of 9.9 m.
3. Upon recharging the water within the arch at approximately 19 l/min, the flow into the south side rock pit increased to 17.0 l/min. In theory, if the water levels truly were equalized, the recharge flow rate should equal the south pit flow rate. The south pit flow rate did vary with site conditions, precipitation, loss of construction water, etc. and should be considered an inexact indicator.

Figure 21 Flow measurements and groundwater level differentials at various stages of the work.

Based on these three events, and the hydraulic conductivity from the coarsest soil sample in the till (1.5×10^{-3} m/s), a window between 12 and 20 cm in diameter could be anticipated if the frozen wall thickness varies between 1 and 3 m. The basis for this computed window size is given in Figure 22 based on Darcy's Law.

Figure 22 Calculated window diameter based on estimated flow rates and pressure differentials.

Note: Window diameter was calculated based on the hydraulic conductivity estimated from the coarsest grain size curve from the geotechnical investigation. Window diameter was also calculated for hydraulic conductivities as much as three times lower and three times higher.

In this particular case history, with leakage from the slurry wall/rock interface in excess of 10 l/min, the groundwater velocities were very high. Using the calculated hydraulic conductivity of 1.5×10^{-3} m/s, the velocity through the freeze prior to any remedial work (phase 1 detailed above) could have been greater than 150 m/day. The soil hydraulic conductivity could be significantly lower and there would still have been a groundwater velocity issue.

6 CONCLUSIONS

The Northern Boulevard Crossing (NBX) project suffered high velocity groundwater movement due to leakage into the "bathtub" excavation at one end of the tunnel. Even though the amount of leakage into the excavation was modest, it was still significant enough to result in a very high (greater than 150 m/day) groundwater velocity.

The NBX project consumed approximately 230,000 l of grout. These quantities of grout were pumped in close proximity to the windows and would have undoubtedly penetrated directly to the edge of the frozen wall and probably into the windows. There is no other explanation for their ineffectiveness other than washout or dilution of the grout. The grout was, most likely, partially effective by altering the permeability of the surrounding ground, but not by directly plugging the window.

Experience from the Northern Boulevard Crossing shows that:

1. Clean coarse gravelly soils were the problematic soil type.
2. The windows were most likely 12 to 20 cm in diameter, based on estimated hydraulic conductivity and hydraulic gradient.
3. The soil permeabilities tended to be greater than as estimated from pre-excavation soil samples, perhaps by a factor of four.
4. Equalizing water levels, if possible, is an effective but difficult remedial window closure methodology. Reducing the hydraulic gradient across the partially frozen wall leads to a reduction in water flow and, hence, a reduction in heat arrival at the window allowing the freezing action to progress. Where water levels inside and outside of the freeze differ

greatly (such that excessive velocities could occur even in moderate or low permeability soils), this is most likely a mandatory measure.

5. Based on the quantity of grout injected, it appears that the grout was most likely partially effective by altering the permeability of the ground and hence the flow, but not directly by plugging the window. Grout injection adjacent to a window in a freeze wall may not be sufficient to promote wall closure – equalising water levels across the freeze wall may also be needed.
6. Care should be taken when freezing in and around pre-existing structures. Piezometer levels and groundwater gradients may be difficult to decipher when amongst a number of pre-existing structures which may act as dams or sources of leakage or recharge. In such cases, precisely equalizing water levels may be very difficult.

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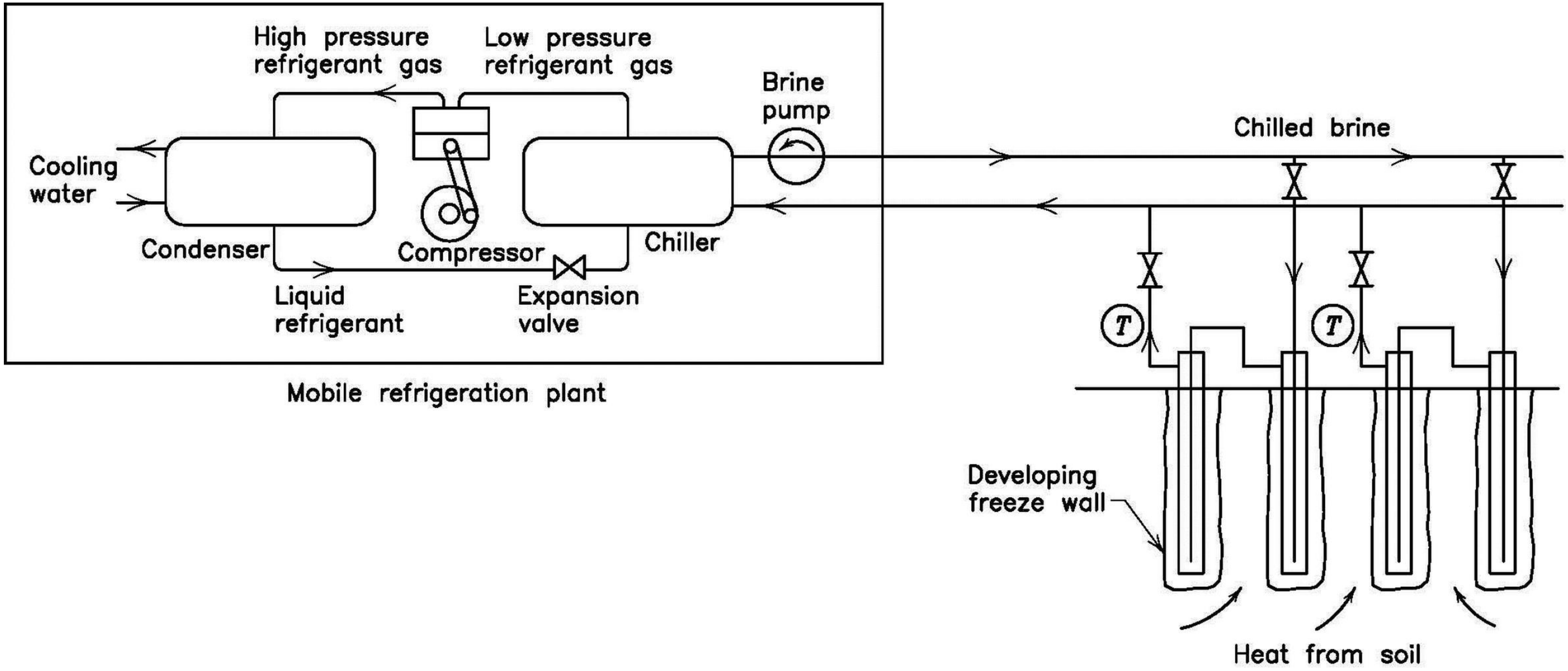

Figure 1 A typical brine re-circulation set-up (Powers et al., 2007).

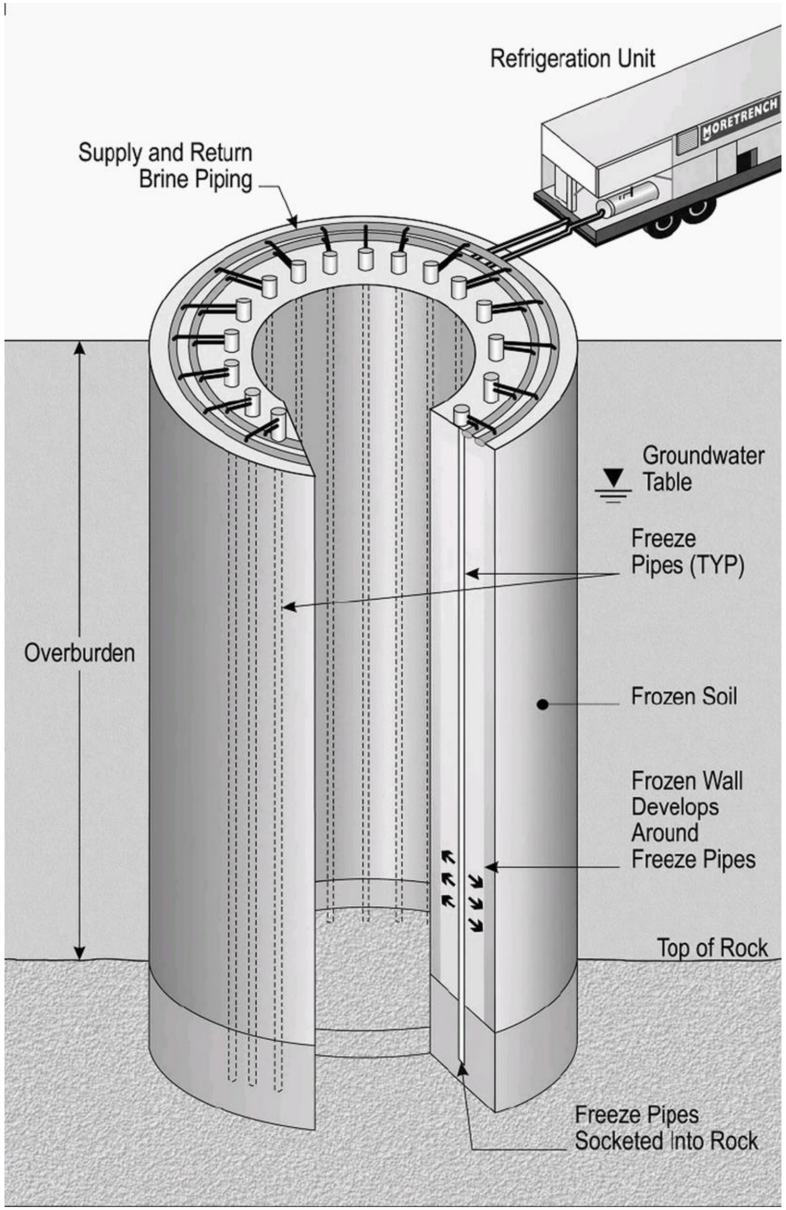

Figure 2 A typical shaft freezing set-up (Powers et al., 2007).

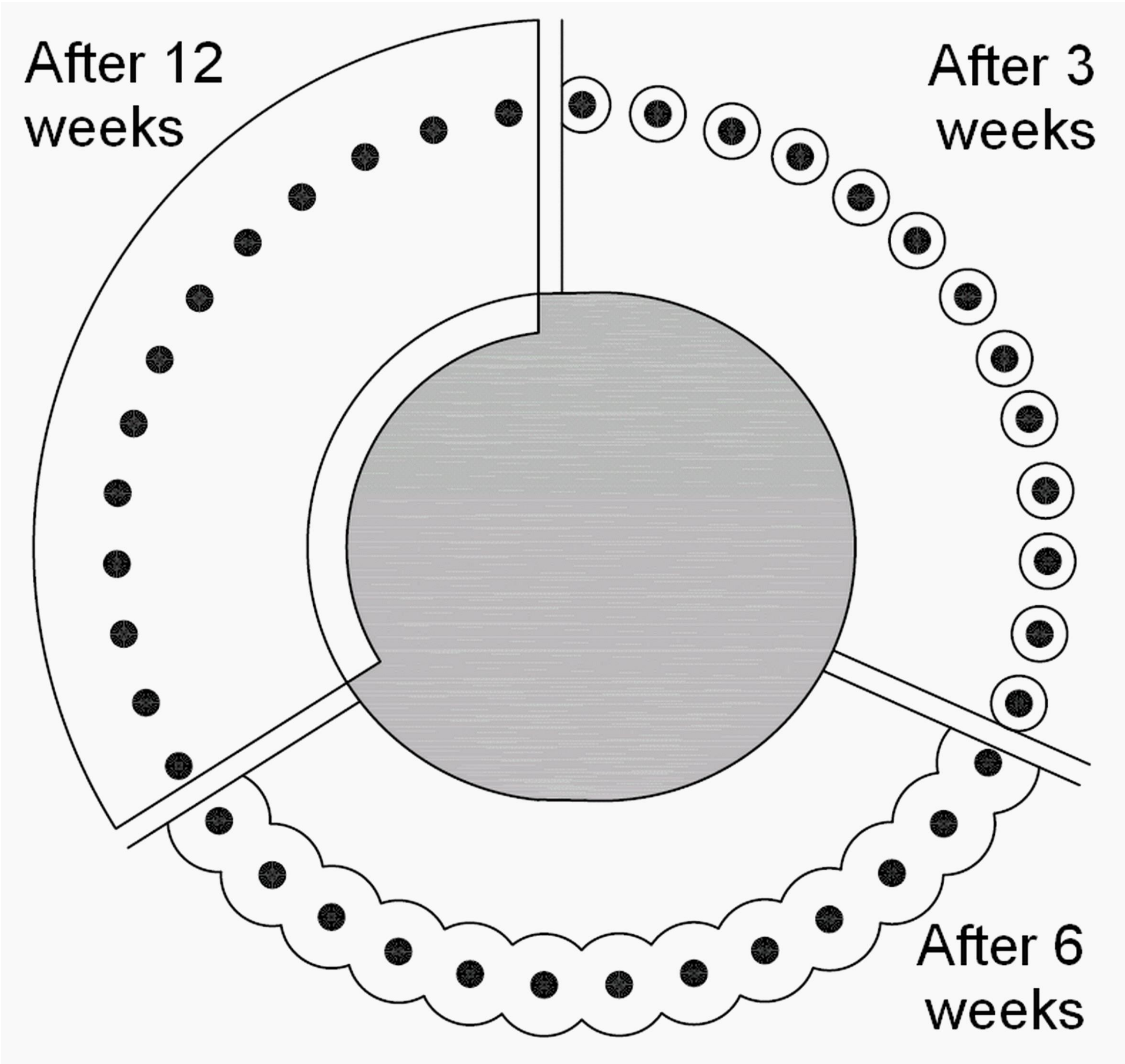

Figure 3 Typical and idealized development of a peripheral freeze (Powers et al., 2007).

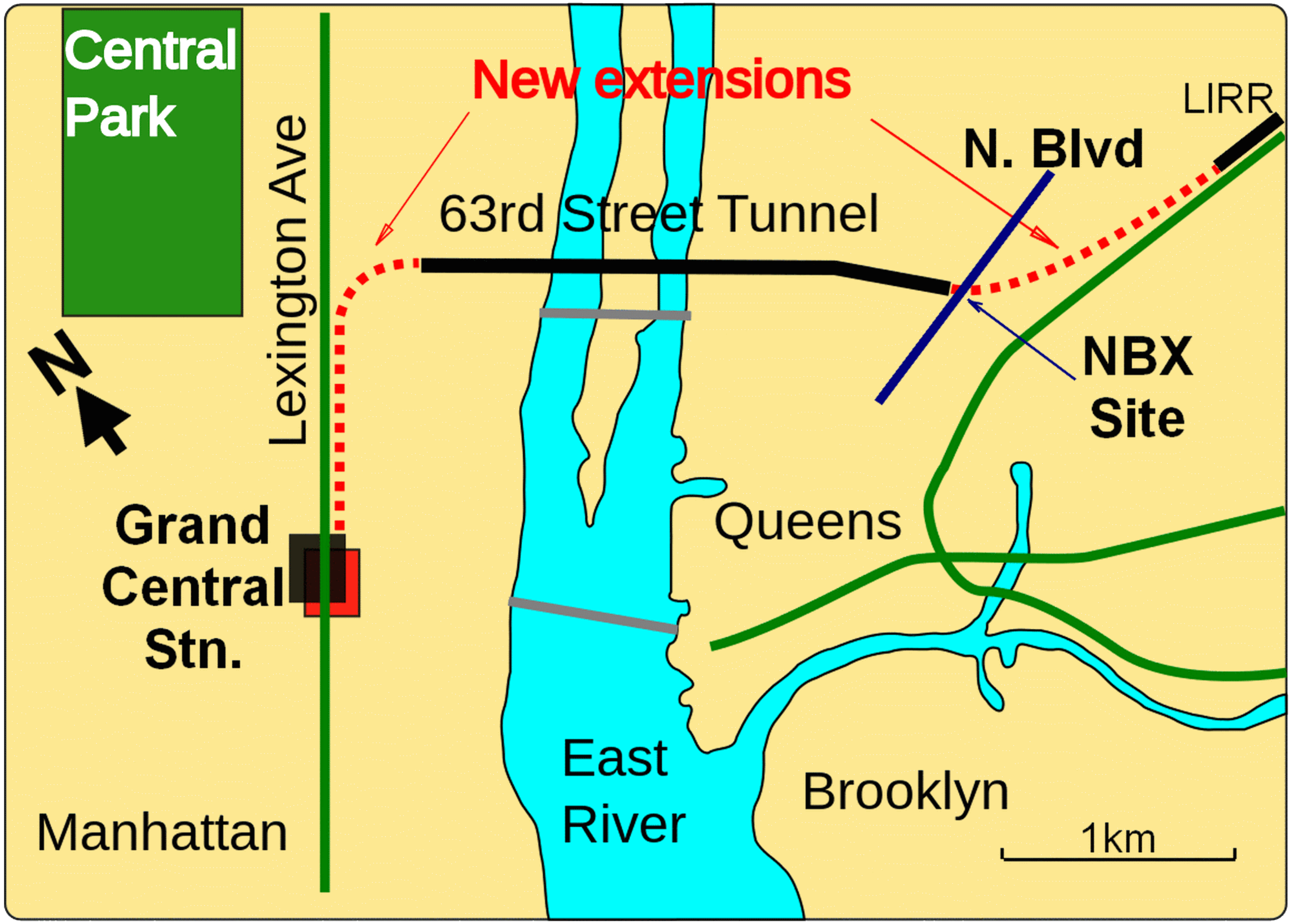

Figure 4 Route of Long Island Rail Extension from Queens into Manhattan.

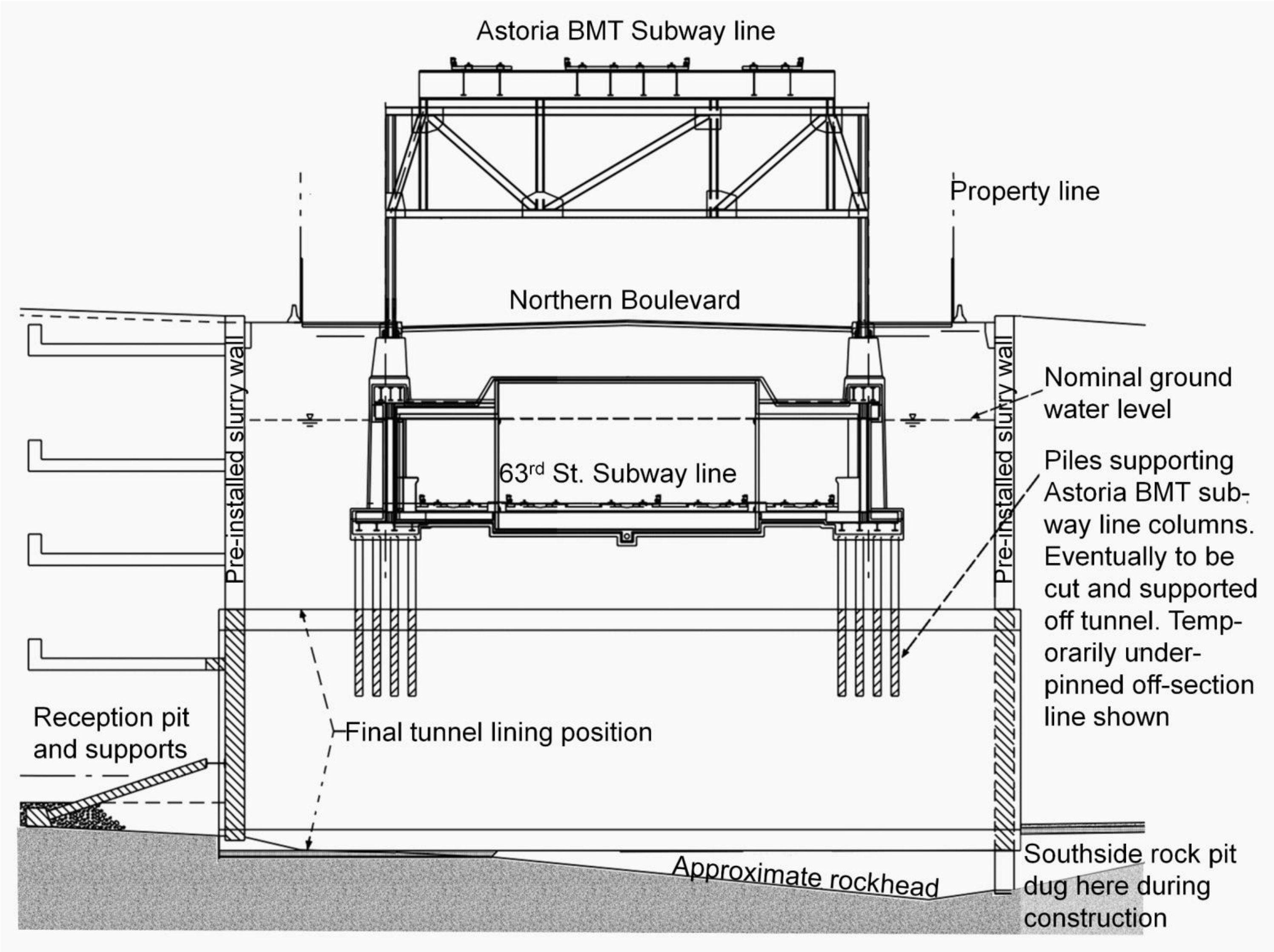

Figure 5 Long section of finished tunnel (adapted from Rice, 2012).

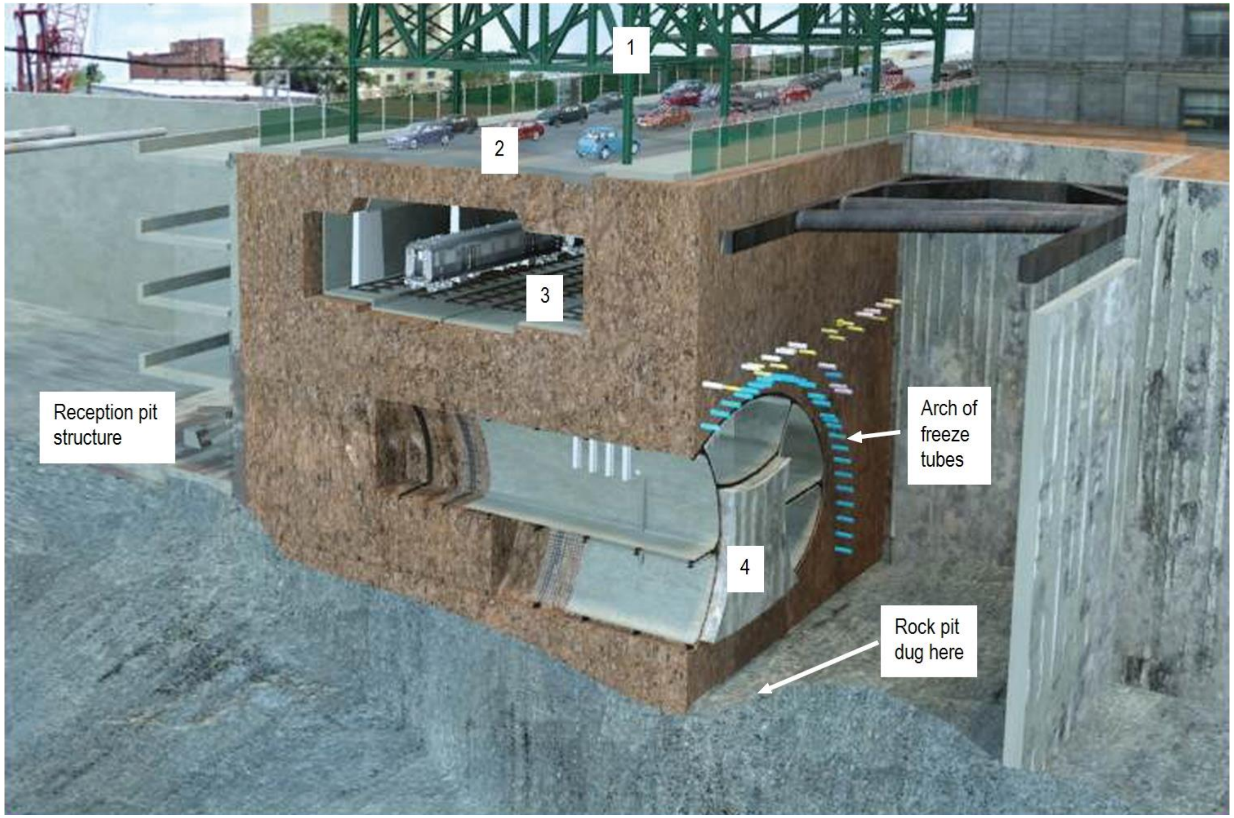

Figure 6 Elements of NBX.

Notes: 1 = Viaduct structure carrying Astoria BMT Subway Line; 2 = Northern Boulevard; 3 = Northern Boulevard Subway line; 4 = Tunnel cut in sections within frozen arch on top of rock.

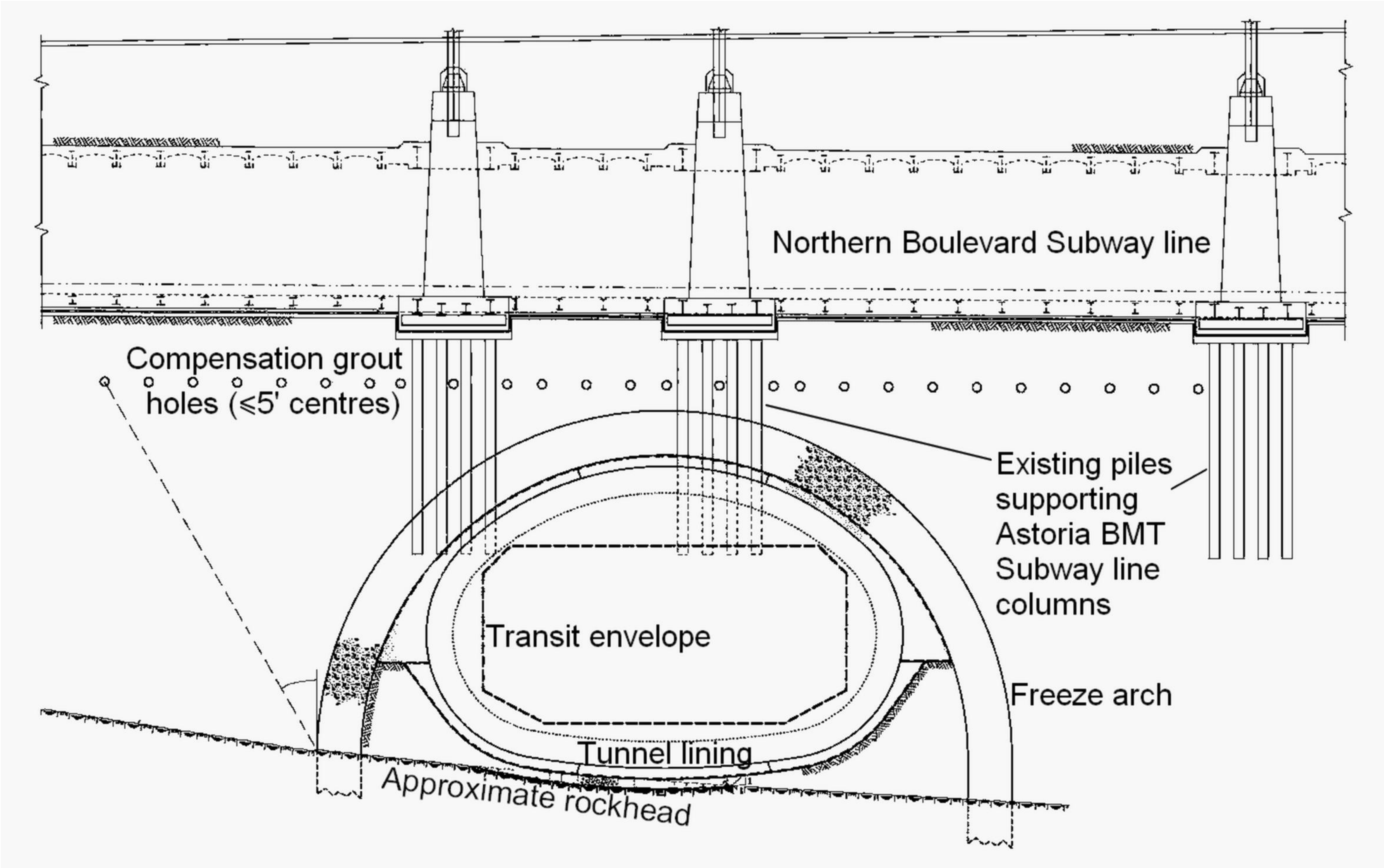

Figure 7 Cross section of tunnel and frozen arch (adapted from Rice, 2012).

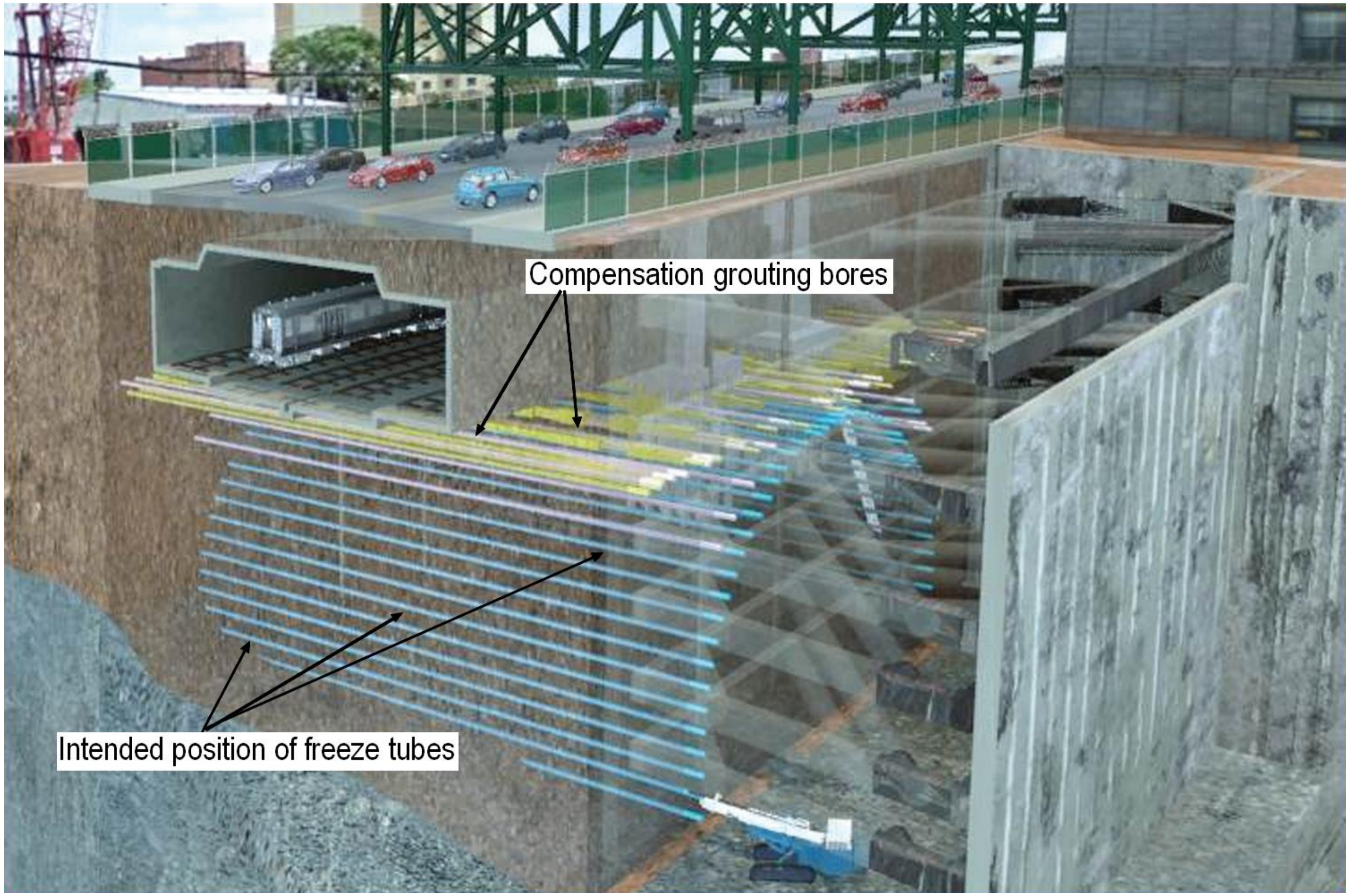

Figure 8 Image of installation of freeze tubes, from south side access pit, to form frozen arch.

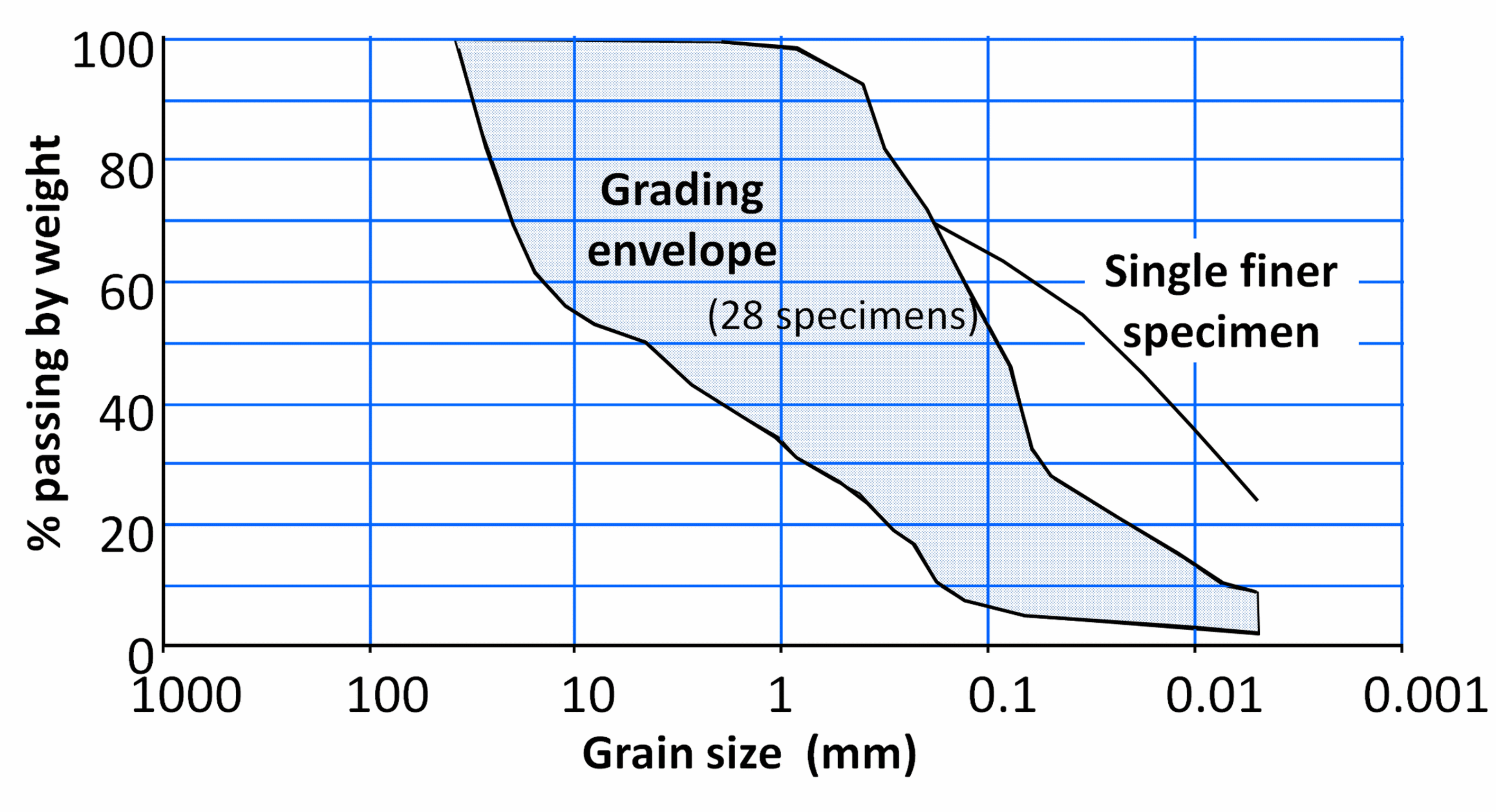

Figure 9 Grain size distribution curves for the glacial till stratum at the NBX site.

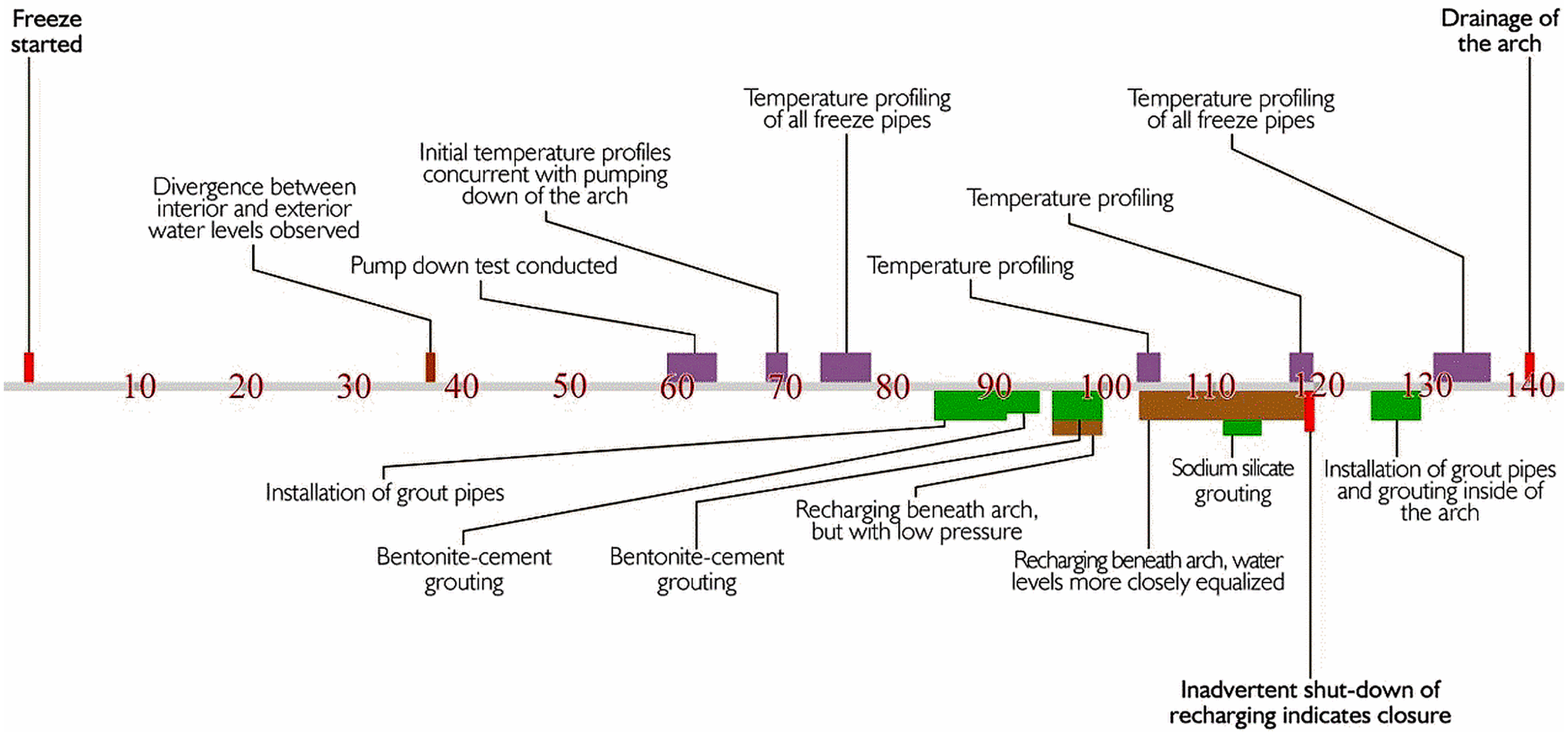

Figure 10 Timeline of events at NBX freeze project.

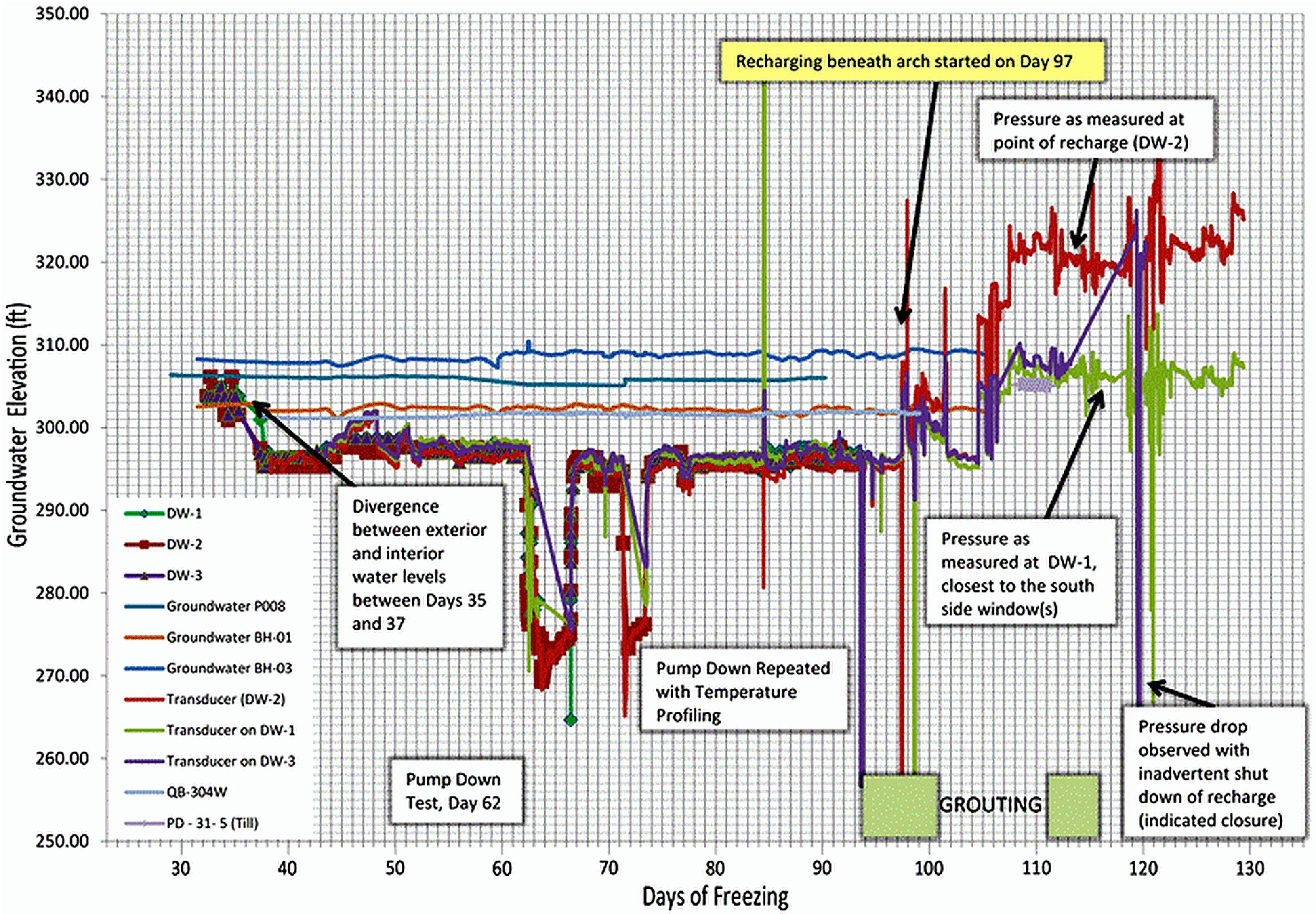

Figure 11 NBX site water level history.

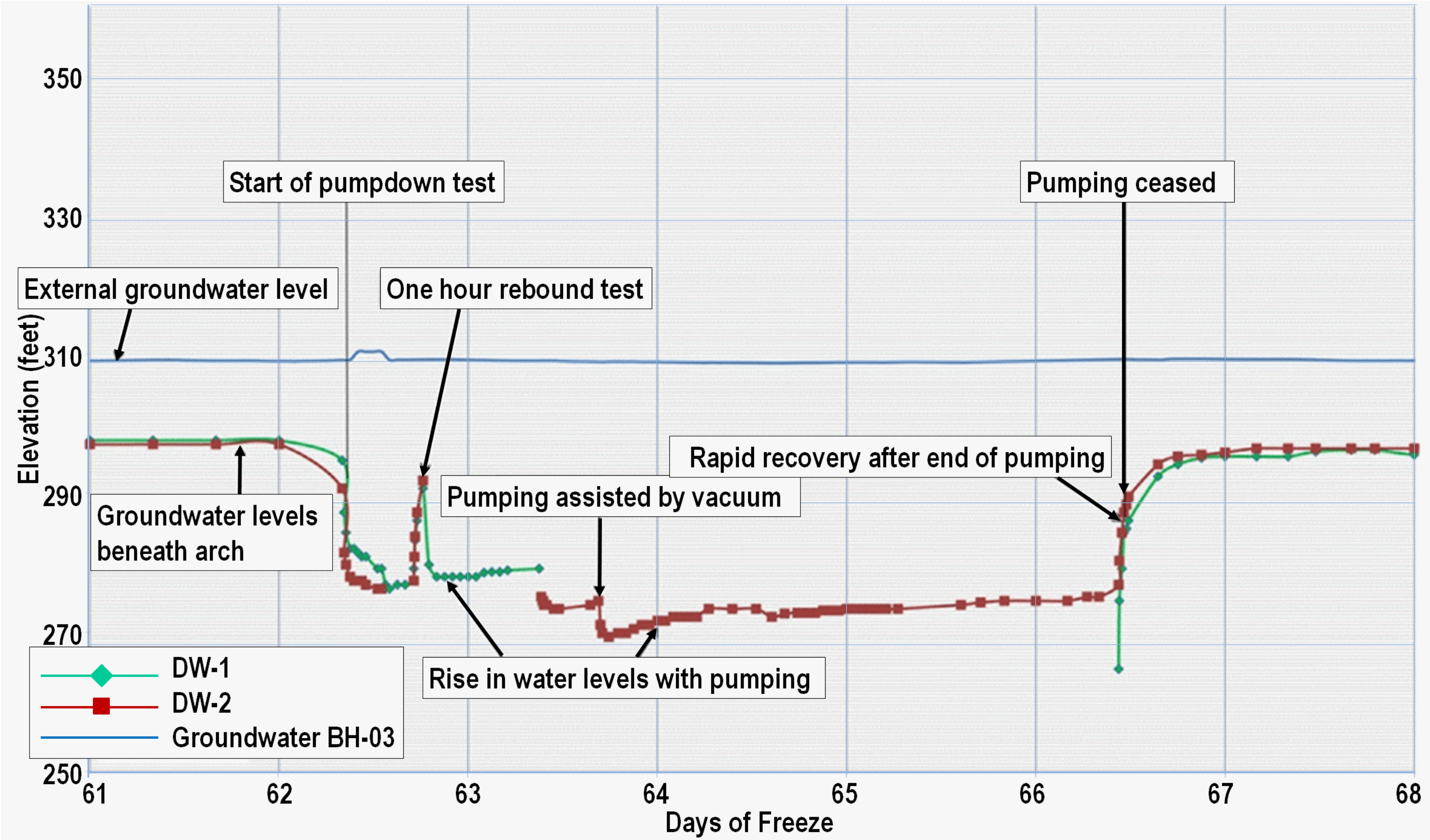

Figure 12 The Day 62 pump-down test at the NBX site.

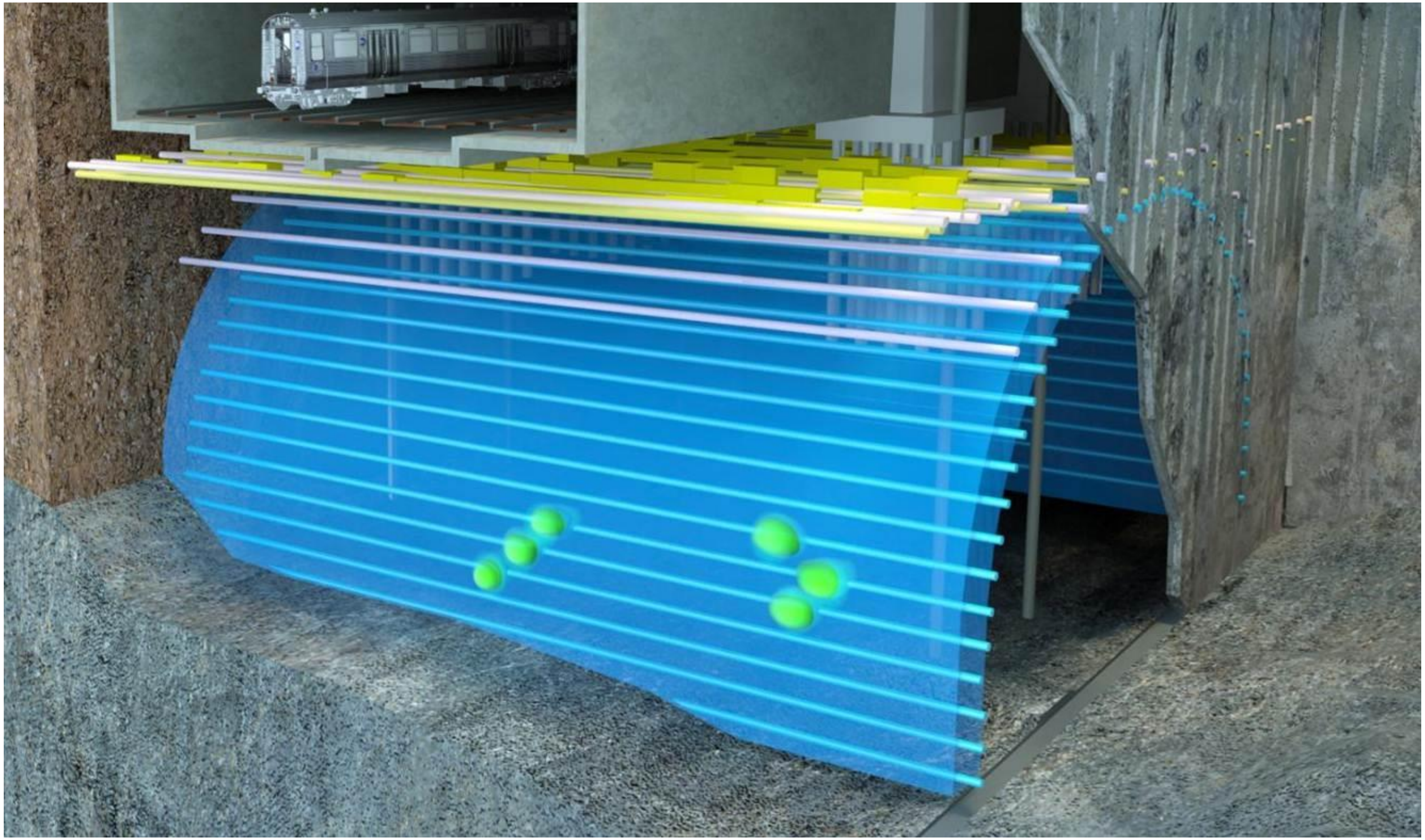

Figure 13 Locations of the warm spots revealed by temperature profiling.

Note: the freeze tubes are shown schematically; their equally-spaced perfect parallelism shows the intended arrangement, not that actually achieved

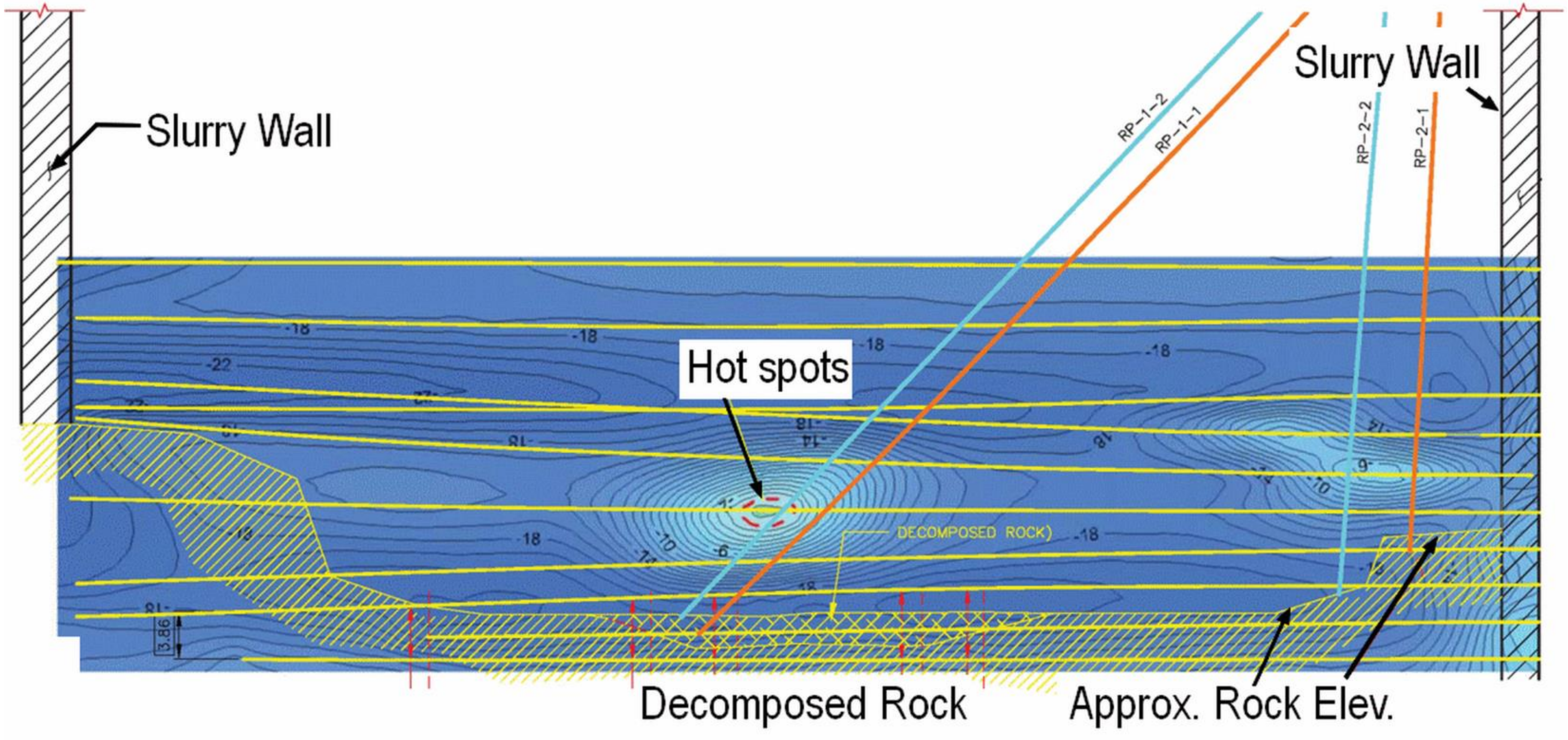

Figure 14 Isotherms of windows and position of grout pipes.

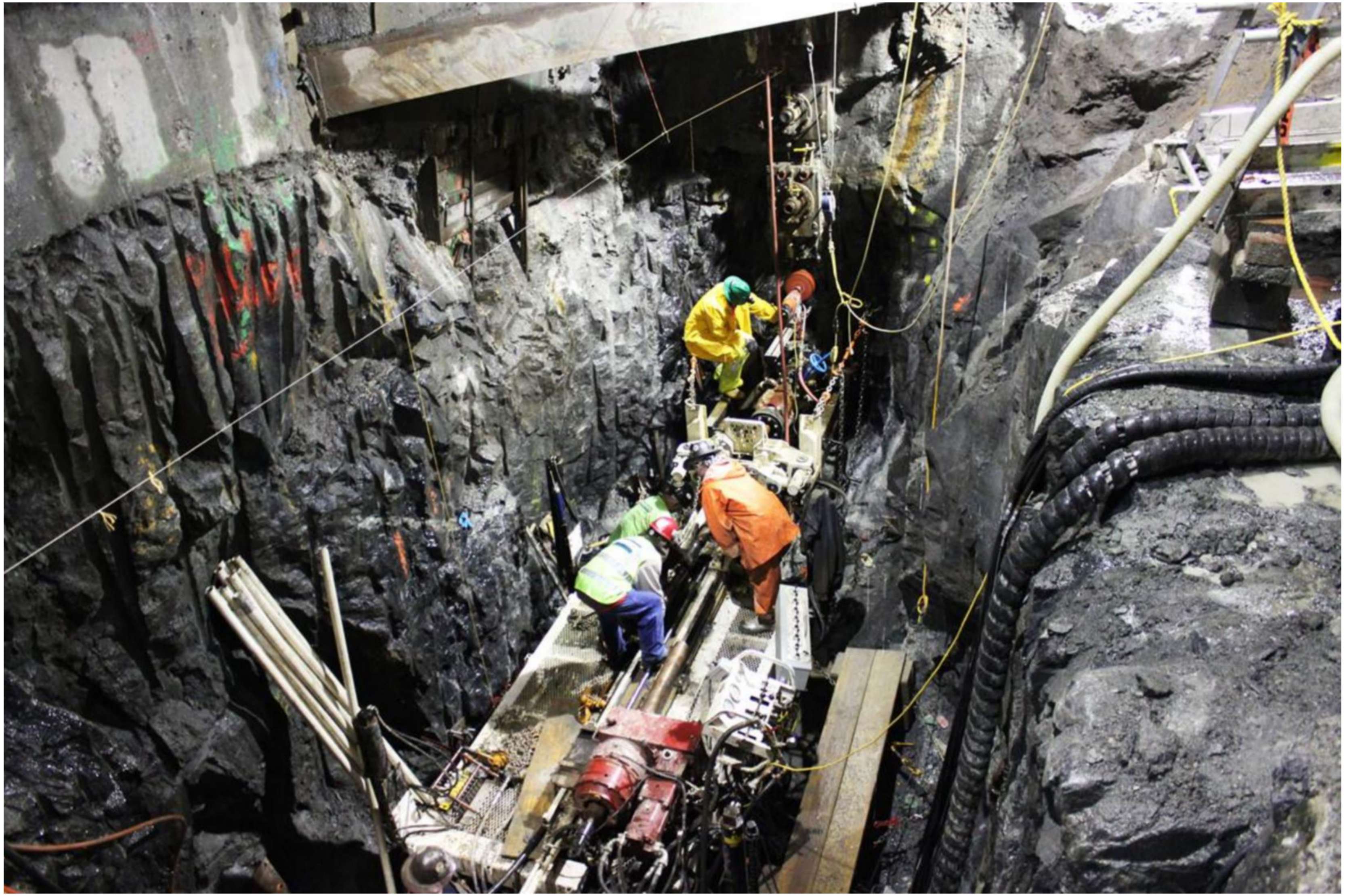

Figure 15 Rock pit excavated at the south side of the arch to permit drilling of the lowermost freeze pipes.

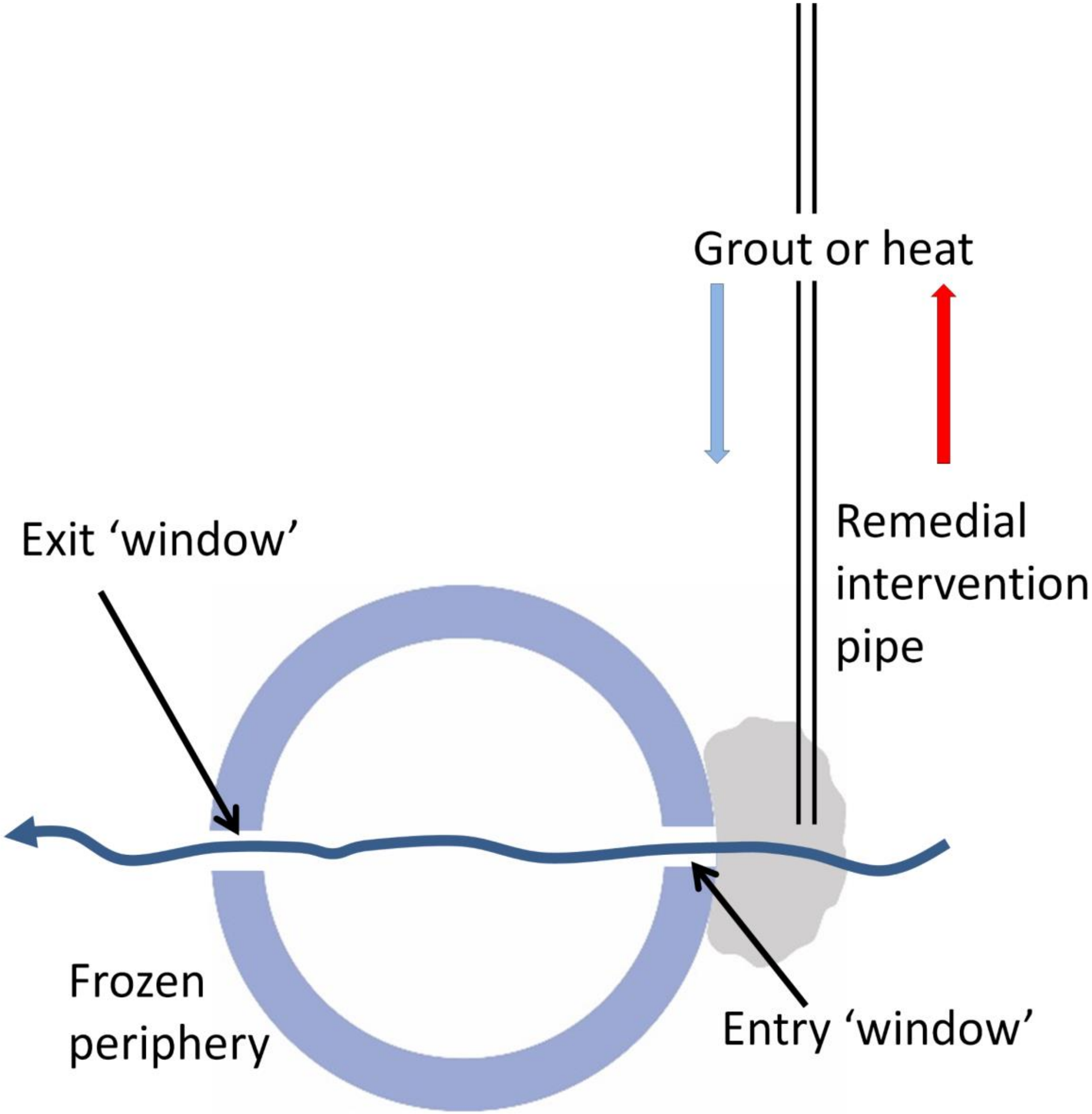

Figure 16 Altering ground conditions over a large area outside a window to encompass the high velocity groundwater flow regime.

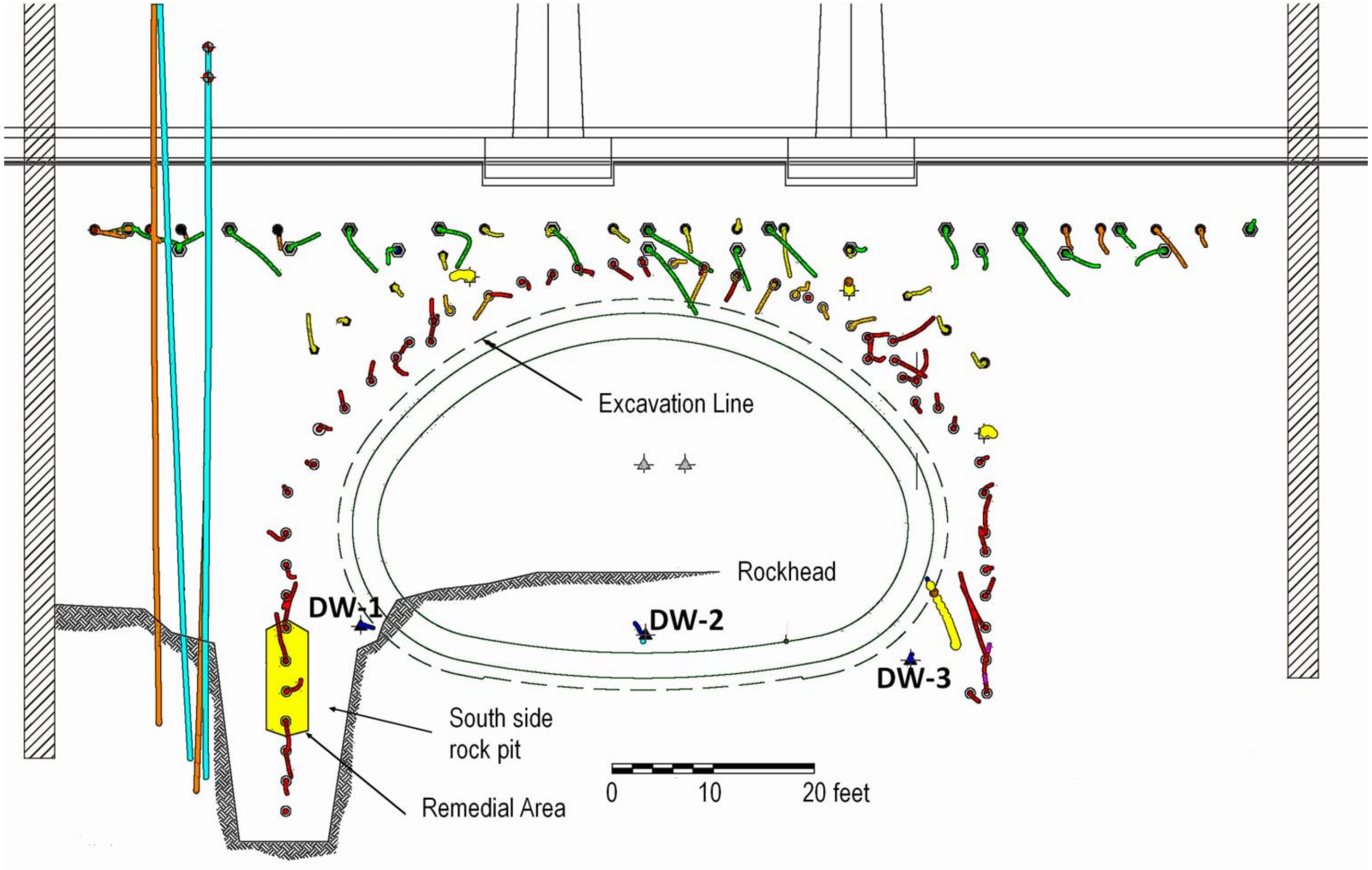

Figure 17 Sectional view of the NBX frozen arch showing freeze pipes, their surveys and interior wells.

Note: Each enumerated dot represents a freeze tube that is intended to run parallel to the tunnel. The as-built hole survey is represented by the coloured line. Freeze tubes are represented in red, interior wells are dark blue, combination heat / compensation grout pipes are yellow, compensation grout pipes are orange, and void pre-grouting holes are green. The orange and turquoise lines on the left represent the grout injection holes installed during attempts to close the windows in the freeze arch.

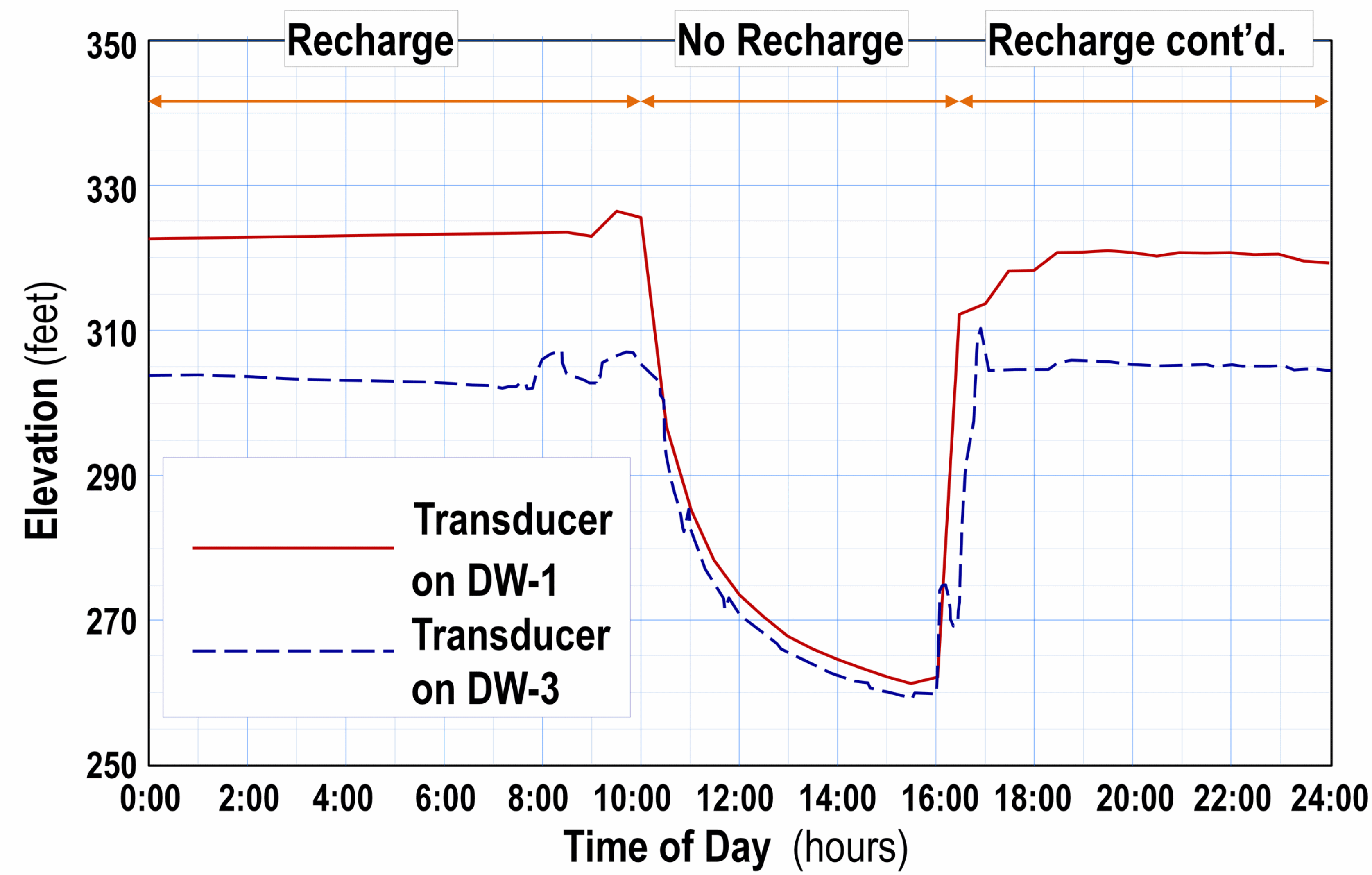

Figure 18 Rapid water level drop beneath the arch upon inadvertent shut-off of the recharge.

Figure 19 Completed tunnel.

Note the frozen ground around the external periphery

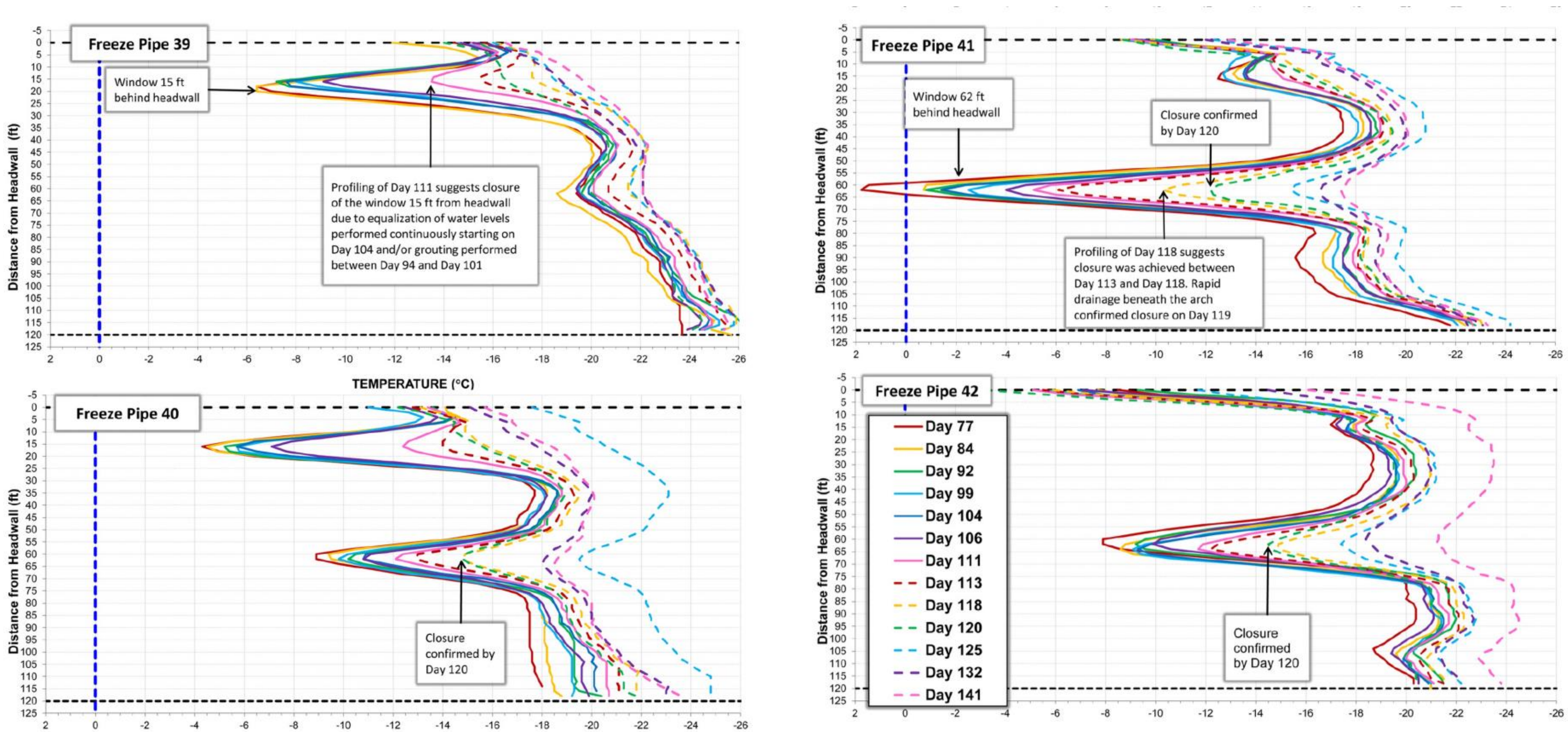

Figure 20 Temperature profiling of the Freeze tubes 39-42 in the windows at various dates.

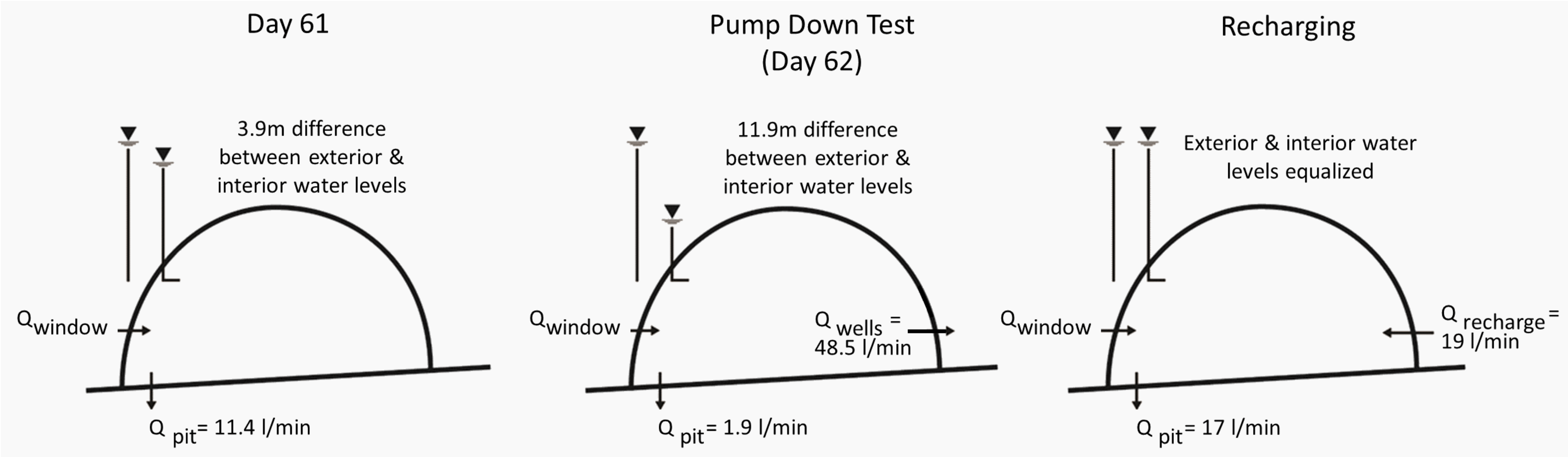

Figure 21 Flow measurements and groundwater level differentials at various stages of the work.

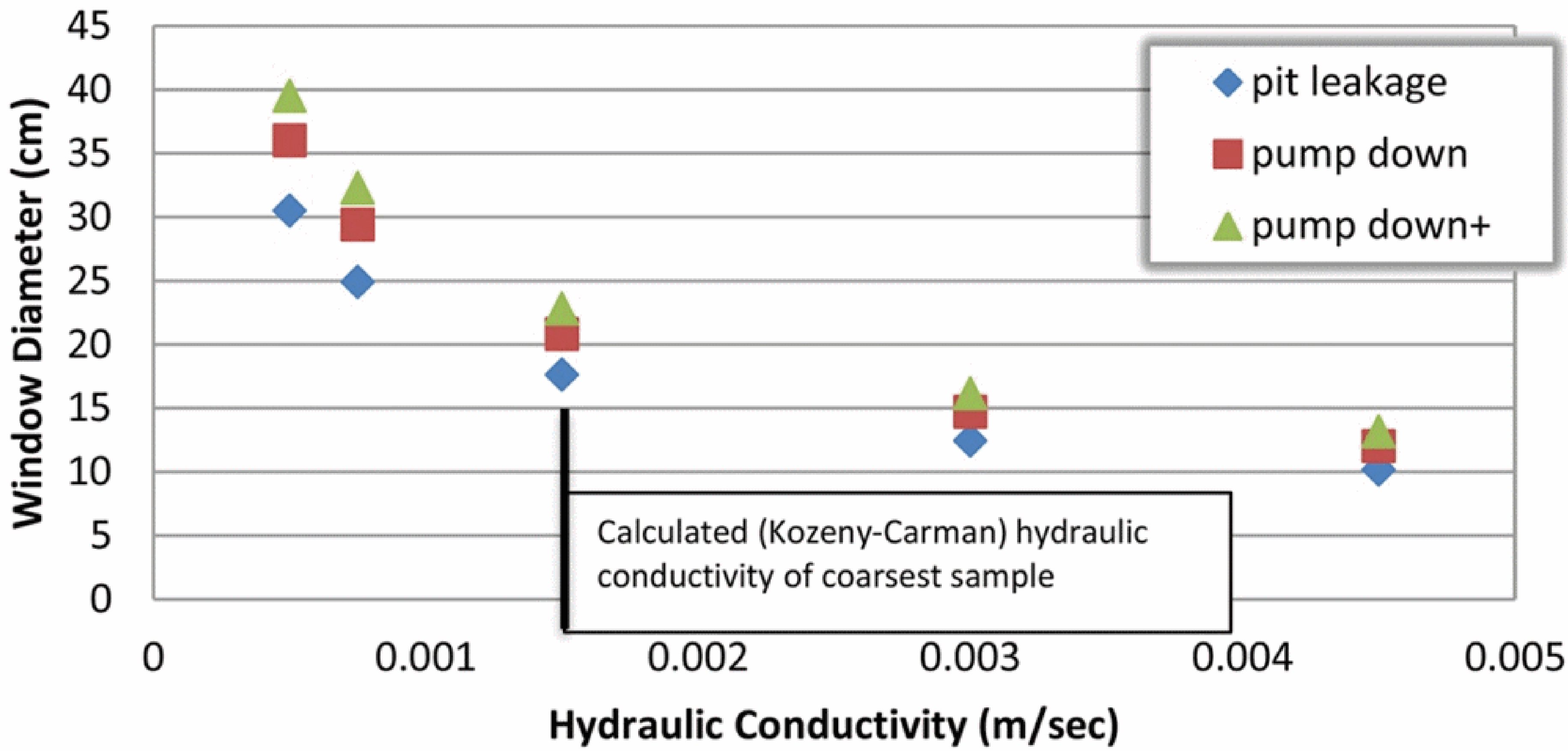

Figure 22 Calculated window diameter based on estimated flow rates and pressure differentials.

Note: Window diameter was calculated based on the hydraulic conductivity estimated from the coarsest grain size curve from the geotechnical investigation. Window diameter was also calculated for hydraulic conductivities as much as three times lower and three times higher.