

Geotechnical Challenges During the Construction of  
Denison Road East Railway Grade Separation

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# **GEOTECHNICAL CHALLENGES DURING THE CONSTRUCTION OF DENISON ROAD EAST RAILWAY GRADE SEPARATION**

by

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### **Abstract**

The Denison Road East (Denison) Railway Grade Separation, located in the Weston neighbourhood of Toronto, Ontario, is an integral component of Metrolinx's Air Rail Link Improvements which provides express commuter service between Union Station and Pearson Airport in Toronto, Ontario. The project includes a new underpass comprising of two reinforced concrete railway bridges, tieback retaining walls, and a 4,000 m<sup>3</sup> precast concrete storm water retention tank. Both the Metrolinx Bridge and CP Rail Bridge are three-span, 45-metre long structures with 1.5-metre thick reinforced concrete decks. The bridges carry four Metrolinx tracks and two CP Rail tracks over the realigned Denison Road.

R.V. Anderson Associates Limited (RVA) and Thurber Engineering Ltd. (Thurber) were retained by Metrolinx to provide the detailed design and construction engineering support services from 2009 to the project's completion in 2015.

Due to the complexity of the project including its proximity to the residential community, the soil stratigraphy and high groundwater table, a sheetpile wall enclosure structure was selected to minimize potential adverse impacts to nearby structures that could result from bulk excavation and groundwater lowering. The enclosure structure also minimized the volume of groundwater being managed during and after construction. A total length of 465 metres of sheetpile wall was installed prior to commencing excavation below the groundwater level. During the construction stages, up to four rows of prestressed tieback anchors were installed along the maximum 15-metre high exposed front face of the sheetpile wall.

The focus of this paper is to show how the engineers overcame the challenges during design and construction. To achieve shoring wall stability and groundwater cut-off, the sheetpiles were penetrated into the underlying very stiff silty clay till/weathered shale. One of the challenges being faced during sheetpile installation was vibration while seating the sheetpiles within the very stiff to hard soils and weathered rock due to the close proximity of the sheetpile walls to adjacent houses and underground utilities. Vibration induced settlement and distress were concerns during construction and, therefore, pre-augured holes were advanced prior to sheetpile installation to mitigate vibration where necessary.

Another challenge was related to the installation of prestressed tiebacks near the bridges which support an active railway corridor. Due to the high groundwater table and the presence of loose soils based on borehole information, fine sands, silts and water were expected to flow out from the tieback holes which would have made the installation of the tiebacks practically impossible. A dewatering scheme was designed and implemented to lower the groundwater table and to depressurize the water-bearing soils in order to facilitate tieback installations.

## Project Background

Opened in June 2015, the Union Pearson Express is considered to be the most critical infrastructure improvement required for Toronto to host the Pan Am Games in the summer of that year. The Georgetown South rail corridor provided the right-of-way necessary to accommodate this service, along with other upgrades to services along the Kitchener rail corridor. Altogether, this project includes seven new overpasses and underpasses, widening or modification of 15 bridges, two additional tracks along its entire length, and several other tracks, signals and drainage improvements.

Prior to the project, Denison Road had intersected two CP tracks and one CN track at a level gate-controlled crossing (as shown in Photo #1). The recommendations of the Environmental Study Report recommended a rail over road grade separation having the capacity for two lanes of road traffic and up to seven rail tracks at this location.



Photo #1 – Level gate-controlled railway crossing

The grade separation is situated within a densely populated neighbourhood with mixed land use including residential, commercial and industrial properties. In the immediate area, there is a residential subdivision with several properties backing very closely to the project site. In addition, a church is immediately adjacent to the grade separation. A solution was required that would have a minimal impact to the pedestrian flow within the community, while still minimizing the overall footprint to avoid extensive and costly property acquisitions. Regardless a number of properties still required acquisition with the residential homes being removed for the construction of the realigned Denison Road.

Following extensive consultation with the local community, property owners and other affected stakeholders during the Environmental Assessment (EA), the recommended design was for an underpass consisting of two separate concrete bridges to carry rail traffic over two lanes of Denison. The envisaged concept was two double span bridges with tall abutments and retaining walls along both sides of Denison with centre piers in the middle of Denison Road.

The EA did not identify several constraints which came to light during the initial stages of design.

- The detailed geotechnical investigation identified a previously unknown thick layer of silt at the level of the proposed lower grade of new Denison. This layer was deemed unsuitable for supporting the construction of the new road. Thurber recommended this layer to be removed.
- Permanent lowering of the groundwater would create unacceptable settlement of neighbouring structures.
- No gravity outfall was available for the permanent lowering of the groundwater to below the grade of the new Denison.
- The estimated flow for lowering the groundwater during construction was in excess of the capacity of local existing sewers.
- Overland storm flow during a 100-year storm event would be intercepted by the new cut of Denison and flood the underpass to a dangerous level. Pumping the overland flow into the existing sewer would require a very large pumping station and new sewer system for several kilometers.
- The construction schedule based on a traditional construction method identified that the critical completion of the construction of the Union Pearson Express commuter line for the Pan Am Games could be in jeopardy if unforeseen construction delay would occur.
- Centre piers are undesirable in a two lane road.

To address the above constraints the RVA's bridge engineers proposed the design recommended by the EA to be amended.

The revised bridge arrangement envisaged three span bridge structures with large centre spans to allow a storm tank to be constructed below Denison of sufficient capacity to store a 100-year storm event and release this storm water by pumping at a rate which could be handled by the existing storm sewer system.

A permanent cut-off wall was added around the depressed area where Denison was dipping below the existing water table to maintain the existing water table in the areas of existing structures.

To ensure that the railway bridges and tracks were completed on time, the design staging required that the railway bridges be constructed prior to the main excavation of the grade separation.

With this proposal, both lanes of Denison run below the centre span; with cut slopes, retaining walls and sidewalks passing underneath the end spans. Figure 1 provides a conceptual rendering of the finished grade separation.

The RVA proposal was accepted by Metrolinx, Canadian National Railway (CN), Canadian Pacific Railway (CP), and the City of Toronto and was considered to comply with the overall intent of the EA so as to not trigger a possible addendum to the EA.



*Figure 1 - Conceptual Rendering*

## **Project Staging**

### Utility Relocations

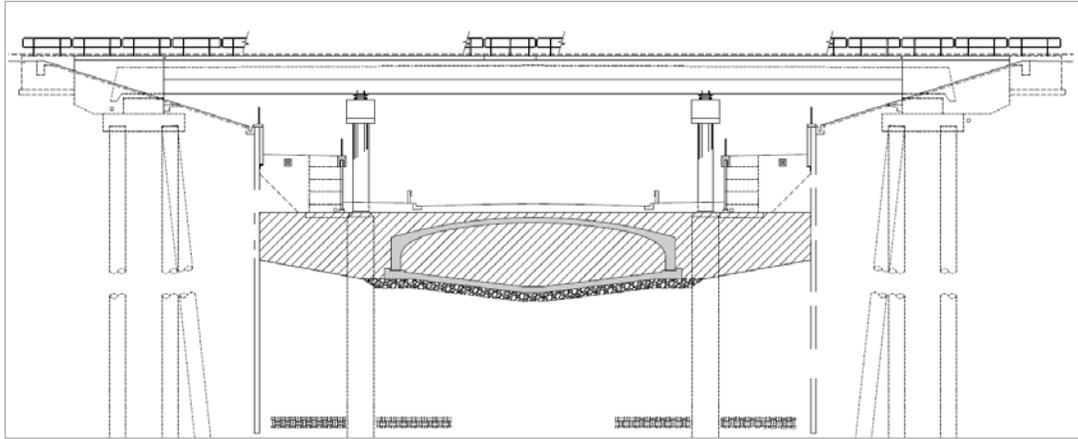
Before construction could commence, extensive utility relocations had to be sequenced to avoid further conflicts during construction. The project required the relocation of several utilities and services to outside of the grade separation. These included a combined trunk sewer, watermain, gas mains, hydro poles and communications infrastructure. The planning of these relocations had commenced a year prior to the grade separation construction, but could not proceed until the necessary easements had been acquired and sewer and watermains relocated to those easements. Therefore several utility relocations could not be relocated until during the construction contract requiring careful coordination with the contractor.

### Track Diversions and Bridge Construction

CN and CP freight, as well as Metrolinx commuter, rail operations were required to remain uninterrupted throughout the construction duration. In order to maintain rail operations, a rail diversion was constructed to provide the necessary space to construct the four-track Metrolinx bridge structure. Once the bridge structure was completed, a second rail diversion was constructed to relocate all trains to the Metrolinx Bridge while construction proceeded on the CP Bridge structure. Once the CP Bridge was constructed, the tracks were constructed to their final configuration and supported by the Metrolinx and CP bridges respectively.

### Grade Separation

Once the railway traffic was supported by the new bridge structures, the work to construct the grade separation could proceed. The staging for the excavation work below track level was even more complex than the staging at track level due to the unique subsurface soil and groundwater conditions as further described in this paper. Refer to Figure 2 for a cross-sectional view of the grade separation.

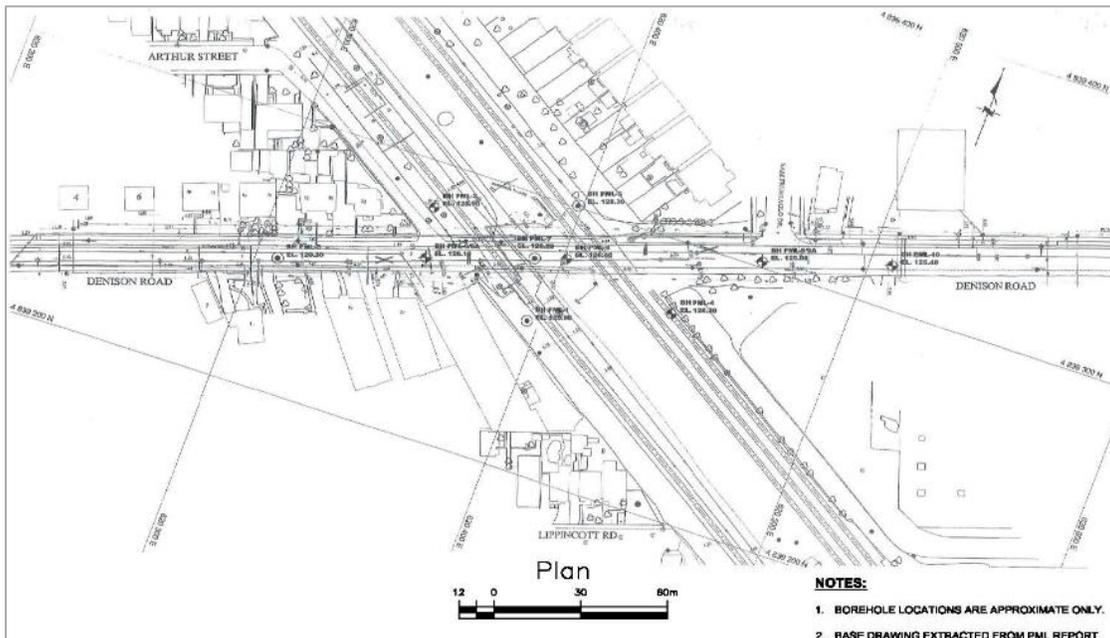


*Figure 2 – West Sectional Elevation*

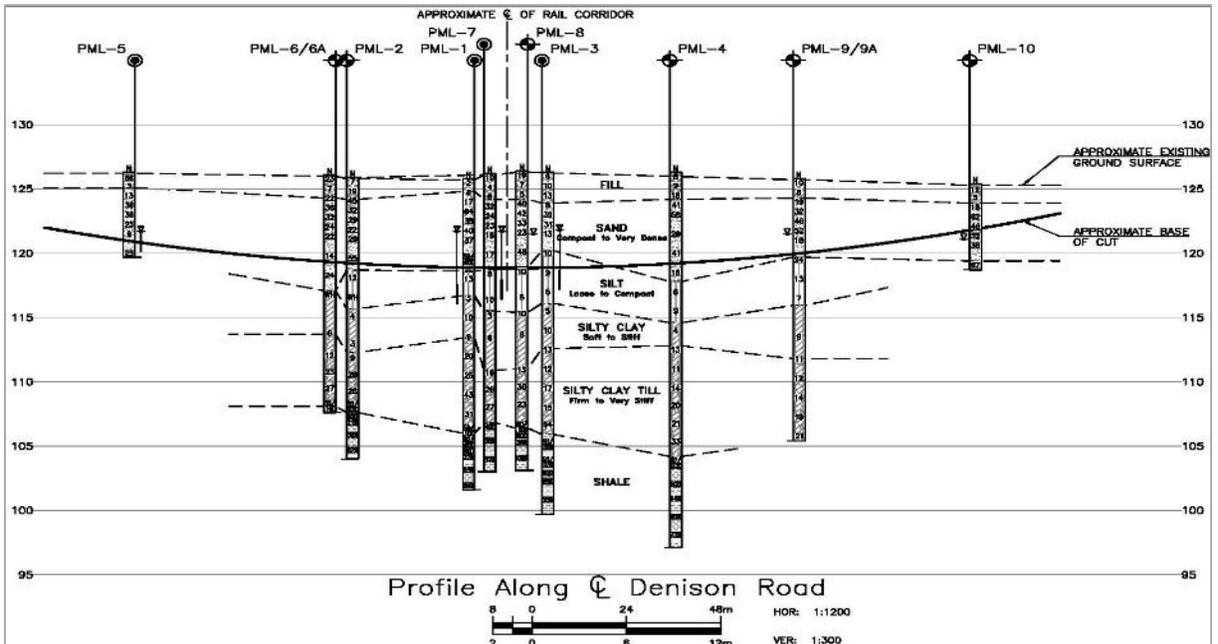
### Overview of Subsurface Soil and Groundwater Conditions

Thurber provided geotechnical recommendations for RVA’s detailed design of the grade separation structure, retaining walls, the permanent cut, storm water retention tank and associated works. Thurber also provided geotechnical comments on the requirements for dewatering and shoring design.

During the geotechnical investigation, a total of twelve boreholes were drilled to depths ranging from 7 to 29 metres. All boreholes were located within a 200 metre long section along Denison Road in the vicinity of the proposed bridge. In all boreholes, the subsurface conditions that were encountered typically consisted of topsoil/pavement/surficial fill overlying compact to dense sand to gravelly sand, overlying loose to compact silt, which is underlain by soft to very stiff silty clay to silty clay till. Grey weathered shale bedrock was encountered in some of the boreholes. The groundwater level was found to be between 4.1 and 4.6 metres below existing ground surface. The borehole location plan (Figure 3) and a stratigraphic profile (Figure 4) are presented below.



*Figure 3 – Borehole Location Plan*



*Figure 4 – Stratigraphic Profile*

The original surface grades were at approximate Elevations 126 to 127 metres. In view of the existing site and subsurface conditions, and the proposed permanent cut configuration with the deepest base of cut at approximate Elevation 118.5 metres, consideration was given to using augered caissons (drilled shafts) socketed into the shale bedrock to provide foundation support. Steel H-piles driven to refusal within the very stiff to hard till or weathered shale were technically feasible but were not permissible due to vibration and noise concerns, and not preferred by RVA's engineers. The soils to be exposed below the base of the cut were too weak to support spread footings. Therefore augered caissons were ultimately selected for the bridge foundations.

In order to minimize the volume of groundwater that would require handling during and after construction, and to minimize the risks of potential impact on adjacent structures associated with groundwater lowering, it was recommended that a continuous wall enclosure such as an interlocking sheetpile wall be used to enclose the cut area. This wall would serve a dual purpose of providing temporary shoring as well as groundwater control during construction. The wall would also serve as a permanent retaining wall to retain the slope behind the walkway and to impede groundwater flow from entering the grade separation wall after construction.

The fully enclosed interlocking sheetpile wall groundwater cut-off enclosure was designed to penetrate a minimum depth of 2 metres below the top of the silty clay till (between Elevations 111 and 113 metres). This penetration depth was to control groundwater during construction and to minimize inflow into the cut following construction. As a result, the estimated long-term groundwater flow was reduced to about 400,000 to 500,000 litres per day.

After the sheetpile enclosure was constructed, pumping from filtered sumps would be required to control any groundwater seepage and any surface water that might be entering the depressed section of Denison. Surface water was also diverted away from the excavation during construction to minimize the water taking.

For the permanently anchored sheetpile wall, the following earth pressure parameters were provided for RVA's design:

Soil Type	Unit Weight (kN/m <sup>3</sup> )	Undrained Shear Strength	Drained Shear Strength	Earth Pressure Coefficients (level ground)		Earth Pressure Coefficients (2H : 1V sloping ground)	
		C <sub>u</sub> (kPa) (φ = 0)	Angle of Friction, γ' (°) (c' = 0)	Active K <sub>a</sub>	Passive K <sub>p</sub>	Active K <sub>a</sub>	Passive K <sub>p</sub>
Engineered Granular A or B Type II Fill	22	0	32	0.31	3.3	0.46	1.73
Sand to Gravelly Sand	21	0	32	0.31	3.3	0.46	1.73
Silt	19	0	28	0.36	2.8	0.65	1.23
Silty Clay	19	25	28	0.36	2.8	-	-
Silty Clay Till	20	125	30	0.33	3.0	-	-
Weathered Shale	22	200	40	0.22	4.6	-	-

Thurber recommended that the permanent wall be designed using the drained shear strength parameters. The undrained shear strengths could be used to check the temporary (short term) conditions for passive resistance using the following equation:

$$p_{p(ult)} = \sigma_v + 2 C_u$$

where  $p_{p(ult)}$  = ultimate passive pressure mobilized by the wall (kPa), (neglecting a depth of 1.2 m below the excavation base for providing lateral resistance) for undrained conditions

$$\sigma_v = \sum_{i=0}^{i=z} [\gamma_{z,i} \Delta_{z,i}]$$

$\gamma_{z,i}$  = unit weight of soil layer i (kN/m<sup>3</sup>)

$\Delta_{z,i}$  = thickness of soil layer i (m)

Passive resistance under long term (drained) conditions could be estimated using the following equation:

$$p_{p(ult)} = K_p \gamma' L + \gamma_w L$$

where  $p_{p(ult)}$  = ultimate passive pressure mobilized by the wall (kPa)

$K_p$  = passive earth pressure coefficient

$\gamma'$  = submerged unit weight of soil (kN/m<sup>3</sup>)

$\gamma_w$  = unit weight of water (kN/m<sup>3</sup>)

$L$  = length of sheetpile embedment below base of excavation (m)

Active pressure distribution as given in the following equation was recommended.

$$p_{a(ult)} = K_a [ \gamma (H - h_w) + \gamma' h_w ] + \gamma_w h_w$$

where  $p_{a(ult)}$  = ultimate active pressure acting on wall (kPa)  
 $K_a$  = active earth pressure coefficient  
 $\gamma$  = unit weight of soil (kN/m<sup>3</sup>)  
 $H$  = retaining height of sheetpile wall above base of excavation (m)  
 $h_w$  = height of groundwater above base of wall (m)

A groundwater level at Elevation 124 metres was assumed for design of the permanent structure on both sides of the railway corridor. The inside of the railway corridor required lowering of the water table permanently by about 2 m. This was considered permissible as the associated settlements in the railway area would not influence the bridge structures as they are founded on rock. These design values take into account the measured groundwater level and the anticipated seasonal fluctuations.

For the design of the watertight cut-off, RVA selected sheet piling over an interlocking caisson wall. Sheetpiling is a proven system providing a (virtually) watertight wall, however, interlocking caisson walls, have proven to leak substantially, with severe ice forming consequences in winter conditions. The height of the cut-off made it necessary to have tiebacks regardless of either of these methods. These tiebacks by necessity were required to be installed below the water table into the silt behind the wall. RVA's engineers were very concerned of a possible unstoppable silt leak or unstoppable loss of ground when drilling for tie back anchors. Any of these events would have disastrous consequences. Consequently RVA specified the cut-off wall would be designed in steel in which it would be possible to stop ground loss by welding quickly a steel plate over the leak.

Further complications arose when Metrolinx was not able to come to a satisfactory arrangement with two property owners to allow subsurface encroachment of the tiebacks anchored to the rock below the owner's property. Anchors were required to be inclined with an angle of more than 60 degrees with the horizontal resulting in very high vertical loads on the sheetpiling to control sheetpile deflections. RVA's engineers therefore specified that the sheeting was to be driven to bedrock with a maximum allowable settlement of 2 mm under full prestress.

For the design of grouted thread-bar or strand tendon soil anchors, the bond length of the soil anchors were to be formed within the silty clay till and/or weathered shale, with the free-stressing length to be determined by calculating the failure plane in the soil, but not less than 4.5 metres. Typical bond lengths of a soil anchor range between 5 and 10 metres, and the nominal drill hole diameters range between 100 mm and 200 mm.

For low pressure gravity grouted anchors with the bond (fixed) length formed within the silty clay till below Elevation 112 metres, an allowable soil to grout bond stress of 50 kPa was recommended for design. In the highly weathered shale below Elevation 106 metres on the north side and Elevation 104 metres on the south side, an allowable shale to grout bond stress of 80 kPa was recommended for the upper 3 metres of weathered shale. Below this weathered zone, an allowable bond stress of 250 kPa was recommended for the intact shale. In order to achieve higher anchor capacity, post-grouting (multi-stage if necessary) of the bond length under higher pressures could be carried out to increase the allowable capacity.

The design unconfined compressive strength of the grout was not less than 30 MPa. No tendon was to be stressed at any time beyond 80% of the specified minimum tendon yield strength ( $F_{pu}$ ).

The allowable axial geotechnical capacity, P, of a single anchor was calculated using the following expression:

$$P_{SLS} = \sum_{i=n}^{i=z} [ \tau_{z,i} A_s \Delta L_{z,i} ]$$

where  $\tau_{z,i}$  = allowable soil/rock to grout bond stress in rock/soil layer i, kPa  
 $A_s$  = surface area per metre of bond length, m<sup>2</sup>/m  
 $\Delta L_{z,i}$  = bond length applicable to the rock/soil layer i, m  
n = the highest rock/soil layers contributing to the resistance.

The contract specified that selected anchors were to be performance tested and the remaining production anchors on site be proof-tested to confirm their carrying capacities.

The proof test loads were determined by RVA and in general varied between 120 to 130 % of the SLS loads with several specific anchors test loaded to 200% SLS. Horizontal ground behavior was monitored by several inclinometers and any movement reported to RVA. Based on ground movement observations, RVA determined the lock-off level of the prestress, which normally was at the 110% SLS with some anchors at 120 to 130 % of SLS. With this method the deflections and corresponding settlements were well controlled. Settlement control due to soil loss was unfortunately more difficult. At several anchor installation locations, the silt flow through the anchor hole was very difficult to control. Soil loss of up to 0.5 m<sup>3</sup> occurred resulting in a corresponding settlement at ground surface of about 10 mm. The railway alert level of the tracks was set at 15 mm differential settlement and consequently Metrolinx and CP had to raise their tracks at several occurrences.

In addition, it was recommended that vertical and lateral ground movement be monitored during pressure grouting. Provisions were incorporated into the construction contract to temporarily terminate grouting and have the situation assessed if excessive movement would be observed. Anchor testing, pressure grouting and other relevant details was in accordance with applicable guidelines such as those recommended in OPSS 942 (November 2009), *Construction Specification for Prestressed Soil and Rock Anchors* and the Post-Tensioning Institute (2004) *Recommendations for Prestressed Rock and Soil Anchors*.

### **The Engineers' Design Solution to the Unique Sub-surface Conditions**

The decision to expedite the bridge construction to ensure the Union Pearson Express tracks would be completed well in advance of the final completion to allow Metrolinx to test their track and signalization had raised some additional challenges for the construction of the pier caissons. The pier columns, which would become exposed during the excavation of Denison, were required to be constructed in the ground. Because of the high water table and the silt deposit it would not be possible to excavate for these pier columns beforehand. RVA designed the piers such that the caissons and the pier columns could be constructed within the augered lining of the caisson. The liner would remain in place until Denison was excavated and the upper portion of the liner could then be removed by cutting the liner out to expose the pier columns.

The bridges were designed as three span continuous beams. Differential settlement between the abutments and piers had to be minimized, and RVA's engineers therefore chose to support both the abutments and piers on caissons embedded in rock, as opposed to steel piles for the abutments and caissons for the piers. Implementing the same type of foundation system made it possible to design the abutments and piers for virtually the same anticipated settlement.

A total of 84 caissons were installed into rock sockets to support the two bridge structures. The bridge abutments are supported by several vertical and battered augered concrete caissons that extend to the shale bedrock. Wingwalls to stabilize the ground near the tracks are connected to the abutments at each end.

The method of pier construction was cost-effective in that the bridge piers were mostly constructed inside steel casings without a deep excavation. With the chosen approach, the ground was simply excavated to the underside of the pier caps and the holes for the caissons were then augered after casings were vibrated into the ground. If another pier configuration had been chosen, the poor soil conditions and high water table would have proven problematic.

After the piers and abutments were completed, construction of the bridge deck proceeded by placing falsework shoring props and slab formwork into the shallow excavation. After this, rebar was placed and the bridge deck was formed. Each solid slab bridge deck was poured continuously over the course of 12-hour time windows. After reaching the required strength, the bridge was strong enough to accept railway traffic. The ballast and tracks were placed and railway traffic was redirected from the diversion tracks onto the newly constructed bridge.

Due to the sensitivity of railway operations to ground movements and the poor ground conditions, a number of special measures to monitor the job site were employed. These measures included track settlement monitoring, inclination monitoring of the sheet piles, groundwater monitoring and monitoring the foundations of nearby buildings for settlement cracking.

Recognizing the importance of controlling ground movements, RVA deemed it necessary to be able to model the interaction between the sheetpile wall tieback anchor stressing, excavation depth and vertical ground displacements at track level. A software tool was developed in house to model this interaction. Site observed displacements closely matched the estimates produced by the tool.

To monitor for settlement, several reference points along the tracks were surveyed at time intervals specified by the railroad authorities involved. If observed movements were to exceed 15 mm, the required railway authorities would be notified and emergency measures would be initiated. Grouted thread bar soil nails were added to the top of the sheetpile walls in areas close to the tracks as an additional measure to limit the ground movements in these areas.

In addition to surveying the tracks, buildings in close proximity to the grade separation were inspected for foundation cracking. Observed cracks were photographed, documented, and some of the observed cracks were monitored with standard crack gauges. These gauges were monitored regularly and movements were counteracted by adjusting tension forces in nearby sheetpile tiebacks.

Inclinometers were installed into the retained ground behind the top of the sheetpile cut-off walls. These inclinometers provided additional monitoring of ground movements. They were also useful in providing feedback for verifying that the site-observed wall deflections throughout the various phases of excavation and tieback installation corresponded to the calculated values.

Numerous aspects of construction of the grade separation required that the groundwater level be monitored. For example, during tieback installation, the groundwater would have to be drawn down as low as possible to prevent water from spouting from the holes in the sheetpiles. To accommodate this, groundwater monitoring wells were installed. The monitoring wells could be easily checked to confirm ground water levels at any time.

## Measures to Mitigate Anticipated Construction Challenges

During initial sheet pile installation, vibration was induced and settlement of the nearby structures was noted. The sheetpile installation was done by using a vibratory hammer. To minimize vibration and to prevent settlement, a trial pre-augering test prior to installation of the sheetpiles was recommended. The purpose of this was to determine the magnitude and distance of attenuation of vibration from the sheetpile installation, and associated vibration induced ground surface settlements. Based on our observations, the sheetpiles penetrated the upper silts and sands without inducing much vibration to the adjacent ground; however, once the sheetpiles encountered the glacial till and bedrock, the vibration was amplified and induced adjacent ground settlement. The following procedure was recommended:

1. Install a linear array of settlement monitoring points at 5 m intervals (i.e. 5 m, 10 m, 15 m, 20 m and 25 m) perpendicularly from the sheetpile alignment. Establish baseline elevation survey readings of the top of pins. All leveling survey should be carried out to a precision of  $\pm 2$  mm.
2. Place at least five (5) geophones, one (1) beside each settlement monitoring point to facilitate vibration and ground settlement monitoring for the full duration of the sheetpile test installation. Extra mobile geophone(s) was available on site for use in case additional measurement locations are deemed necessary during the course of the test installation.
3. Select a panel of six sheetpile sections typical for this project. Sheetpile test installation was carried out in the following sequence:
  - Install the first sheetpile section using 100% of the full vibratory frequency available on the hammer.
  - Install the second, adjacent, sheetpile section using 100% of the vibratory frequency.
  - Install the third, adjacent, sheetpile section using 50% of the vibratory frequency.
  - Install the fourth, adjacent, sheetpile section using 75% of the vibratory frequency.
  - Install the fifth, adjacent sheetpile section using 100% vibratory frequency until 2 m above the target depth/elevation, then decrease to 50% vibratory frequency until completion.
  - Install the sixth, adjacent sheetpile section using 100% vibratory frequency until 2 m above the target depth/elevation, then decrease to 75% vibratory frequency until completion.
4. Repeat Step 3 above in a row of pilot holes pre-augered (cuttings left in place) to 2 m above the target depth/elevation as well as all the way to bedrock surface (three to 2 m above target and three to bedrock). A separate pre-augered hole was drilled and the auger cuttings examined to confirm the subsurface conditions at the test location.



Photo #2 – Pre-augering prior to installing sheetpiles



Photo #3 – Installing sheetpiles after pre-augering

Another challenge during construction was when installing tie-backs, difficulties with water pressures and loss of fine sands and silts were encountered during the installation of the second row of tie-backs at

Elevation 119.7 m near the Metrolinx's structure. This level generally coincides with the sand-silt interface depicted in the boreholes. Foam injection (Photo 4) was implemented to remediate this condition where limited success was achieved. Another method which was introduced was the triple grout method (Photo 5). This method was also met with marginal success.



Photo # 4 – Foam application to prevent loss of fines



Photo #5 – Triple grout method

In light of the limiting success of the various methods to control loss of fines, it was recommended that localized dewatering on a temporary basis be required to facilitate continuing installation of the tie-backs. Instrumentation and monitoring program was put in place and operational prior to implementing this temporary localized dewatering scheme. The purpose of the instrumentation and monitoring is as follows:

- Monitoring of observation wells to assess the rate and effectiveness of groundwater table lowering due to dewatering
- Monitoring of settlement rods to assess potential ground settlement within the dewatering zone of influence.

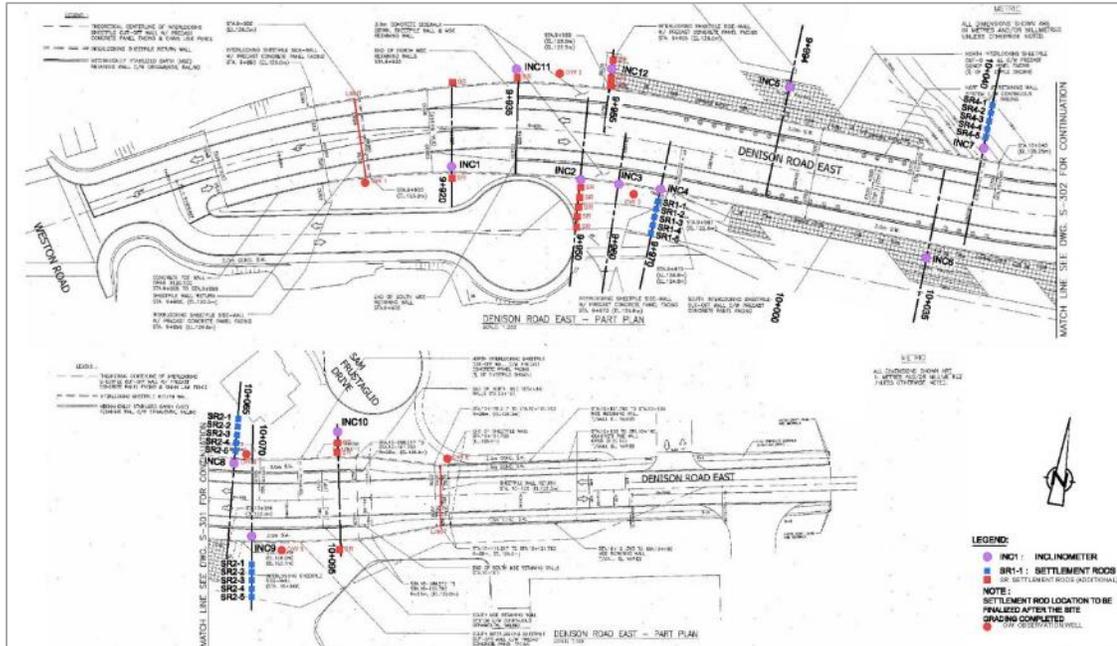
A temporary dewatering scheme was recommended as follows:

- Initially activate the dewatering wells 50 m on either side of the location where tie-backs were being installed to cover a total distance of 100 m.
- Monitor groundwater drawdown response and potential ground settlement, and adjust dewatering operation as required.
- It was anticipated that effective groundwater drawdown would require at least two weeks; tie-back installation could start after the observation wells indicate effective groundwater lowering.
- Deactivate dewatering system once all tie-backs within the zone of dewatering were successfully installed, tested and approved.
- Monitor groundwater recovery and potential ground settlement.
- The above dewatering and installation procedure would be repeated for other sections where required.

The original anchor system design had taken into consideration the groundwater pressure behind the sheet piles and a dewatering system had been specified to operate for the entire duration of the Contract. The dewatering specialist designed and implemented a dewatering system, although some of the tie-backs in the vicinity of the Metrolinx's structure were installed to Elevation 120 m instead of the design tip level at Elevation 116 m. In view of concerns of increased potential settlement and drawdown due to dewatering at steady state conditions, Metrolinx decided on limiting the scope of dewatering. The foregoing decision mitigated also the previously mentioned concern of the likely generation of large volumes of excess water at steady state dewatering conditions that could potentially overwhelm the City's sewer system, and the concern that a potential long-term duration failure of the dewatering system could have disastrous consequences. Therefore the design was revised to remove the full scale dewatering system and the wall anchorage system was redesigned to withstand higher loads due to increased hydrostatic pressure and increase the soil prestress loads to control corresponding deflections.

Additional instruments including settlement rods and water level observation wells that were implemented are as follows:

- Settlement rods at selected accessible locations adjacent to the excavation on both the north and south sides.
- Water level observation wells to monitor groundwater level drawdown and recovery.
- Building monitoring points, crack gauges and track settlement markers would continue to be maintained and monitored.



Instrumentation was implemented in sections and for sufficient duration until such time that tieback installation within any sections was completed. These instruments would be monitored on a daily basis during dewatering.

The following design graph was prepared to show the estimated ground settlement due to drawdown under steady state conditions.



The sheetpile installation was successfully completed despite the challenges of the site's unique subsurface conditions. Refer to Photo 6 which shows the walls supporting the excavation face to allow the subsequent road excavation and storm retention tank construction.



Photo #6 – Completed sheetpiling, subsequent excavation and storm retention tank

### **Project Completion / Success Story**

The Denison grade separation successfully overcame a number of significant engineering challenges to provide improved traffic flow while simultaneously improving pedestrian flow through the surrounding neighbourhood. The bridge structures are now in service and serving the Union Station and Pearson Airport Express Line (refer to Photo #7).



Photo #7 – Aerial photo of grade separation construction

The cooperation between Metrolinx, CP, CN, the City, and local property owners was of utmost importance in the successful delivery of this complex project.

The contractor was Dufferin Construction, and the contract administrator was Morrison Hershfield. The excellent work of both these firms was instrumental to the success. The constant interaction, feedback and responses to the behaviour of the soils during construction by the design team of RVA and Thurber, and the cooperation of the entire team resulted in overcoming challenges and mitigating settlements. Negligible impact to neighbouring structures was achieved and therefore this project had a very successful completion.

The final cost of the project was approximately \$55,000,000.