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Monitoring during the construction of the Allen Sewer Tunnel, Toronto, Canada

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ABSTRACT: A 3m external diameter sewer tunnel was driven through very dense sands and silts and under an operational subway line. Compensation grouting was used to protect the subway line from the effects of the tunnelling. Monitoring of surface, subsurface and structure movements was used to help control the tunnelling and the grouting.

1 INTRODUCTION

Allen Station is to be constructed at the eastern end of the new Eglinton West Subway route in Toronto. Prior to constructing the station, a 1.980m I.D. trunk sewer had to be realigned to pass beneath the planned level of the station base slab. This realignment involved the reconstruction of nearly 1.1 kilometres of sewer tunnel at a lower elevation, as shown on Figure 1.

The ground conditions are summarized on Figure 1. The tunnel is within a glaciolacustrine deposit of very dense sands and silts with a low uniformity coefficient (typically 3 to 10). The groundwater level before tunnelling was between 6m and 9m above the planned invert level of the tunnel. A pumping test in the glaciolacustrine deposit measured a bulk permeability of $2x10^{-5}$m/second; this value was considered representative of the bulk permeability, but from grain size analysis on samples of the deposit it was assessed that the permeability varies locally between $5x10^{-5}$ and 10^{-7}m/second.

Overlying the tunnel is a deposit of hard silty clay or clayey silt Till. At the southern end of the tunnel deep fills overlie the Till; however, the fill consists largely of Till excavated for the construction of the nearby subway route.

The tunnel was driven from a shaft at the southern limit of the contract to a reception shaft at the northern limit. The first 550m of tunnelling were under a public park, Cedarvale Park. At the north end of the park the tunnel passed under a twin cell subway box with about 6m clearance between the subway and the sewer tunnel.

2 TUNNELLING METHOD AND PROTECTION MEASURES

The specifications, prepared by the designer (D.S.Lea Associates Ltd.), required that the contractor select the tunnelling method to be suitable for the ground conditions and to meet specific criteria for surface and subsurface movement. The contractor (UCL Construction Ltd.) was also responsible for selecting and designing the initial support system, within which a cast-in-situ concrete final lining would be poured.

The tunnel was constructed using a 3m diameter Lovat tunnelling machine equipped with pressure relieving gates and flood doors. Although a screw conveyor was available the contractor chose to rely on the pressure relieving gates. One of the reasons given for the choice was that the Geotechnical Baseline Report indicated the presence of boulders in the glacially derived soils. Initial support consisted of expanded steel ribs and timber lagging.

The criteria for surface and subsurface movement were based on achieving a maximum

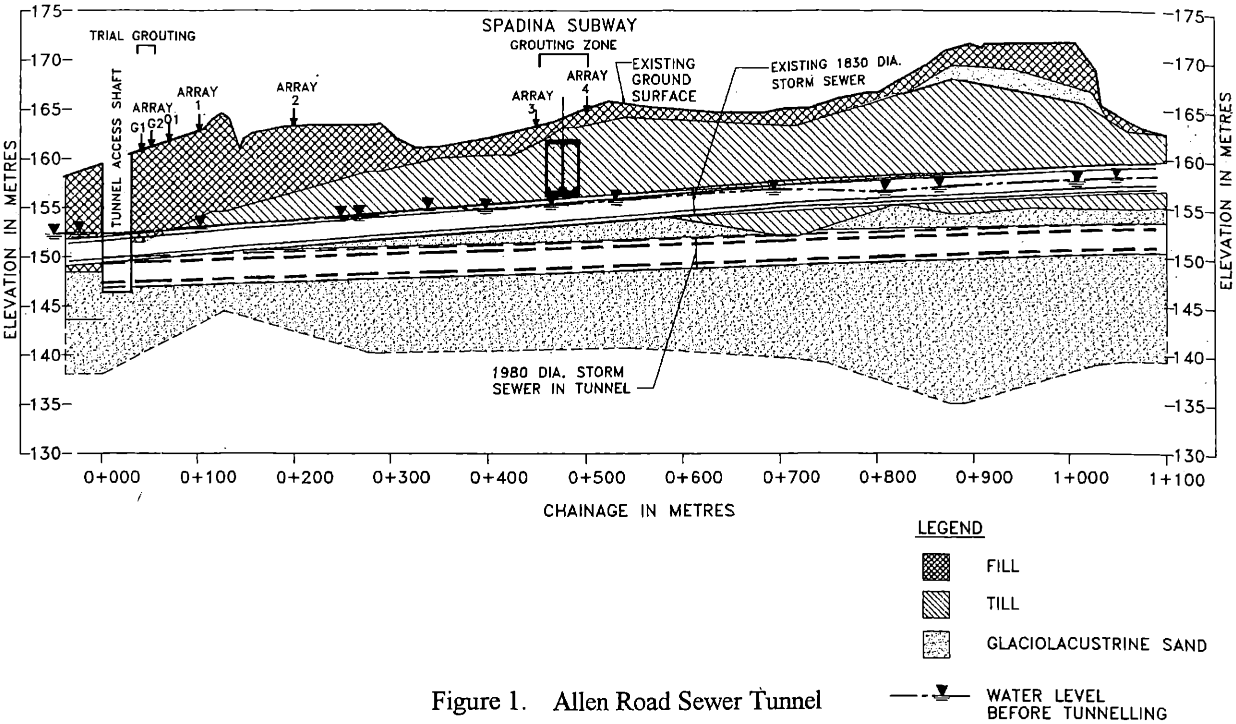

Figure 1. Allen Road Sewer Tunnel

volume loss of 2%, and the assessment of the effects on adjacent buildings and structures was carried out using this value. A structural analysis carried out by D.S.Lea assessed that there would be unacceptable damage to the twin cell subway box if the volume loss reached this figure as the tunnel passed beneath. It was therefore specified that the glaciolacustrine deposit below the subway and above the tunnel springline should be treated by grouting using a silicate/reagent grout. The grouting was designed and specified by ECO Grouting Specialists, as a sub-consultant to D.S.Lea. The grout types to be used were specified, as it was considered necessary to receive acceptance in principle of the use of chemical grouting from the Ministry of the Environment and Energy of Ontario prior to tendering the work.

The contractor, together with the grouting sub-contractor (Geotech Contracting Ltd.), made a Value Engineering Proposal (VEP) to substitute compensation grouting for the specified chemical grouting. This VEP was accepted following agreed changes.

Sleeved grouting tubes were installed horizontally above the tunnel from four 2m diameter shafts (Figure 2), the shafts being constructed using a bored piling rig. The sleeved pipes were installed by jacking and jetting. The proposed grout mix constituents consisted of fly ash, cement and bentonite, although a bentonite grout was used for some of the grouting from the fourth shaft.

One of the changes made during the discussion of the VEP was the addition of a test section to demonstrate the performance of the compensation grouting. The only readily available area for the test section was close to the main access shaft, so the test section was 45m from the start of the tunnel drive.

3 MONITORING

3.1 Planning and installation of the Instrumentation

In order to monitor the performance of the tunnelling and the response of the structure, a detailed instrumentation layout was included in the contract drawings. The instrumentation layout was developed by D.S.Lea in discussion with Golder Associates, the Program Geotechnical Consultant (PGC). The specified instrumentation included:

Surface settlement points. These were installed over the centreline of the tunnel at 25m intervals, with lateral arrays of points at about 200m intervals.

Subsurface settlement points, to be placed to 1m above the tunnel, at each lateral array.

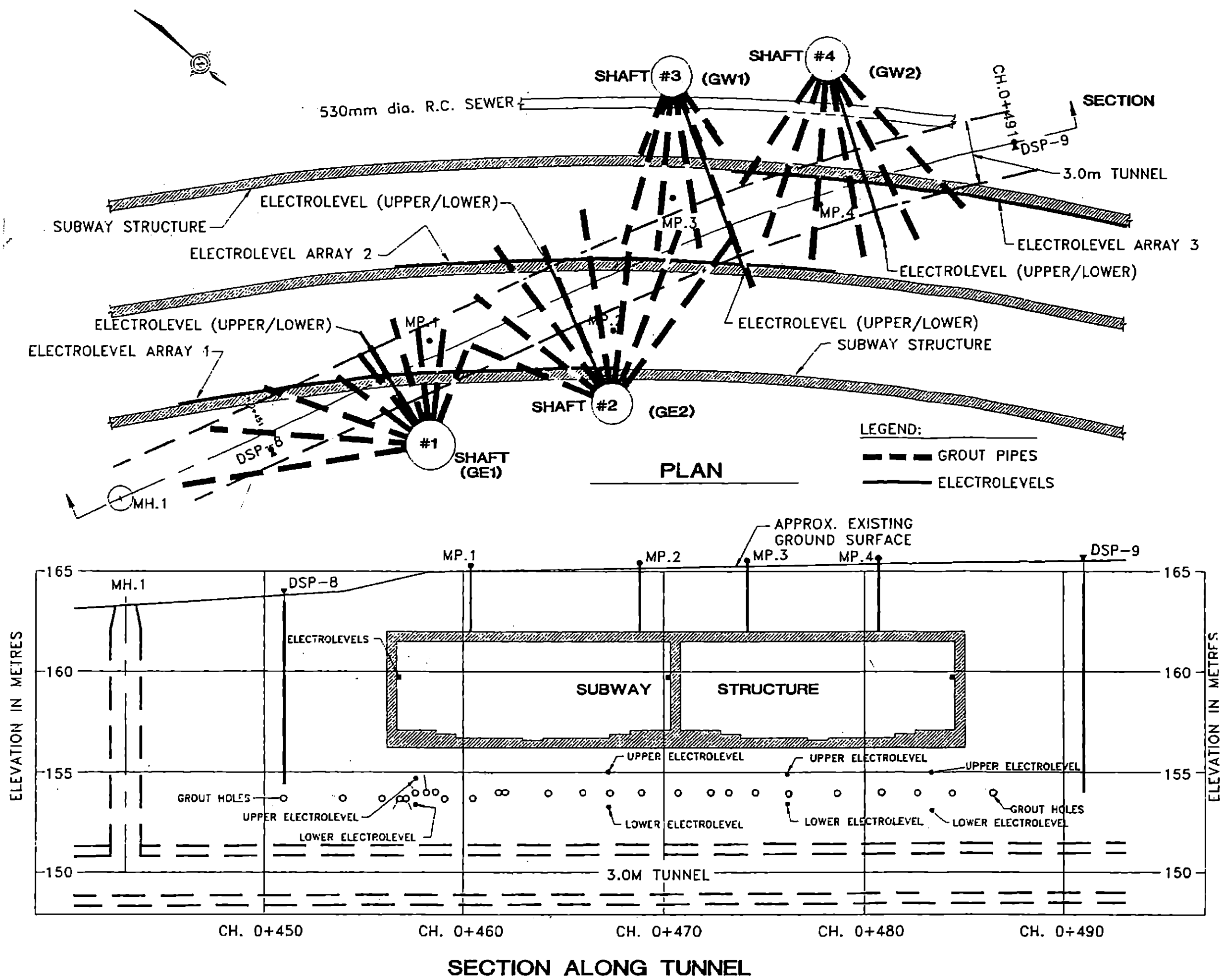

Figure 2. Plan Of Grouting And Monitoring For Tunnelling Under The Spadina Subway

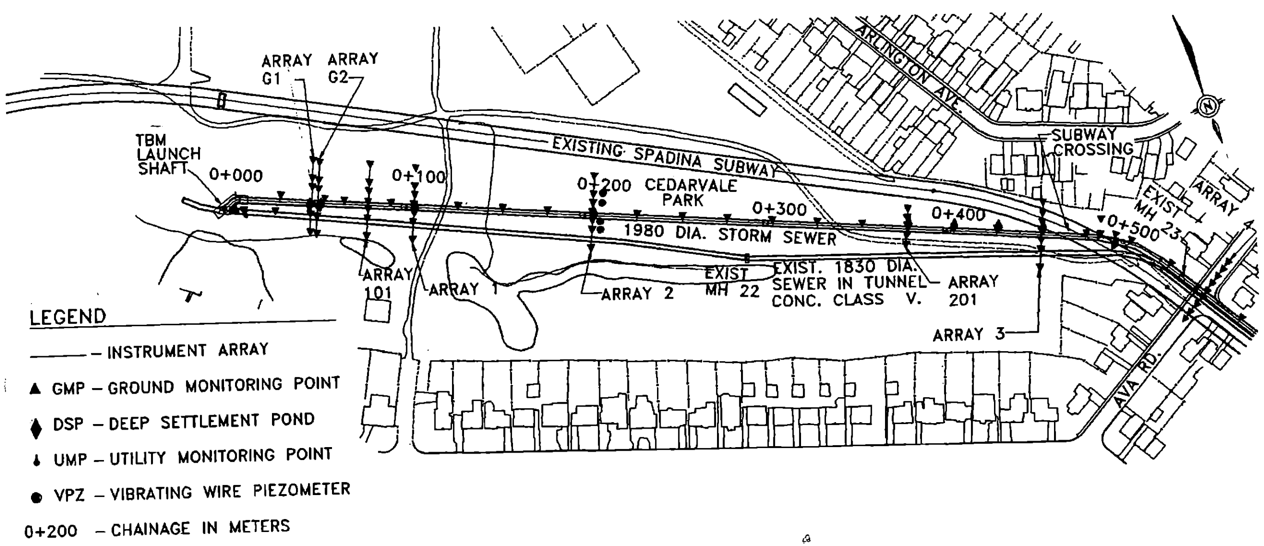

Figure 3. Plan Of Tunnel Monitoring In Cedarvale Park

Piezometers. Four vibrating wire piezometers were specified at Chainage 200m. In addition, standpipe piezometers installed during the soils investigation at about 150m intervals along the alignment were to be monitored.

Beam Electrolevels. Three arrays of beam electrolevels were specified, to monitor the three walls of the subway structure. Monitoring was via a data logger in the tunnel, which was connected to a telephone cable already present there.

In ground electro-levels. Four arrays of electrolevels, to be installed within horizontal inclinometer casings, were specified to be placed below the structure. These instruments were specified because there was a concern that, with a stiff structure overlying hard and very dense soils, subsurface movements could occur without an immediate response from the structure.

The responsibility for the work involved in the monitoring was divided as follows:

Contractor:	Drilling of holes and support services
D.S.Lea:	Reading, by survey level, of surface and subsurface monitoring points
Golder Associates:	Installation of monitoring, reading of instruments other than surface and subsurface points, collection of all data in project database, interpretation of data

Following the acceptance of the VEP, some modifications were made to the monitoring layout, in particular the addition of surface and subsurface settlement points at the grouting test section, and a second line of in ground electrolevels, placed between the tunnel and the grout pipes. The final monitoring layouts for the section of the tunnel under the subway and within the Park are shown in Figures 2 and 3 respectively.

3.2 Monitoring prior to tunnelling under the subway

The monitoring within the park before the subway was used to verify the contractor's methods of tunnelling and grouting. Figure 4 shows the measured surface settlements along the centre line after tunnelling. The settlements shown in the

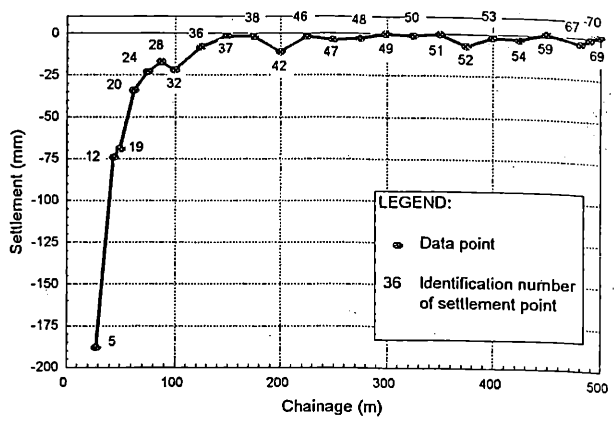

Figure 4. Surface Settlements Recorded In Cedarvale Park

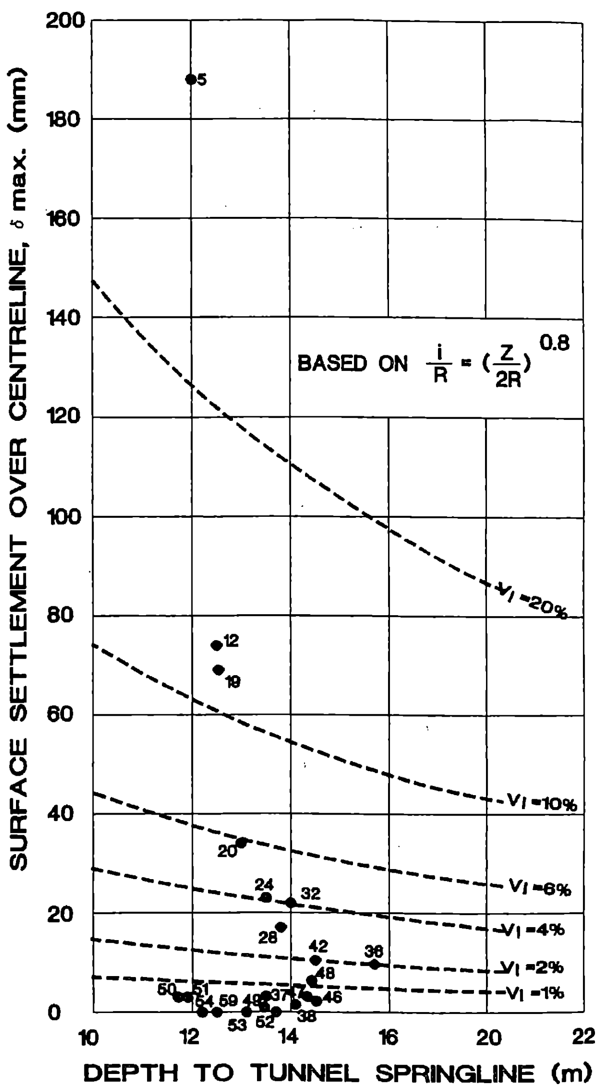

Figure 5. Recorded Values For Relative Volume Loss, Cedarvale Park

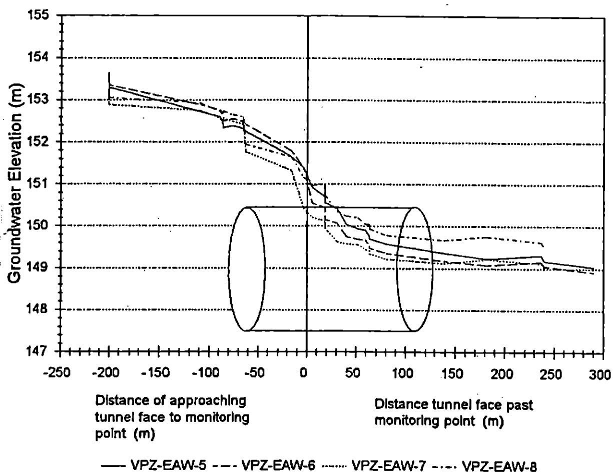

Figure 6. Piezometric Levels Recorded At Chainage 200 m

Figure are those measured about five months after the tunnel had passed. However, almost all of the measured settlement occurred once the tunnel was 20m beyond each point. The volume losses achieved are shown in Figure 5. It can be seen that settlements in the first 100m of tunnelling were higher than specified. The volume of wet sand being removed for each rib advanced was much higher than the theoretical volume, resulting in volume losses much higher than specified. The measured surface settlements do not fully reflect this loss of ground. A subsurface settlement point at Chainage 40m, at the start of the trial grouting section, dropped by 1.2m as the head of the machine passed. The injection of 11.7m^3 of grout in the trial section restricted the surface settlement to about 40mm. Tunnelling performance subsequently improved rapidly, such that by Chainage 120m the surface settlements were within requirements. This improvement in performance was a result of better control of the tunnelling machine, but was probably assisted by the lowering of the groundwater levels due to the use of a permeable rib and lagging lining. Groundwater levels, as monitored in the vibrating wire piezometers at Chainage 200m, are summarised in figure 6. It can be seen that drainage into the tunnel resulted in the groundwater levels being lowered ahead of the tunnel, such that groundwater levels had been reduced to about the crown of the tunnel as the machine passed.

Another factor in the apparently better performance approaching the subway may have been the effect of increasing cover of hard Till. There was some indication from the deep settlement points beyond Chainage 300m that the subsurface movement was not being fully reflected at the ground surface.

Even though settlements in the park were generally of little consequence, it had been decided to specify maximum settlements throughout, to ensure that maximum control was achieved prior to passing under the subway. Figure 5 demonstrates the value of this approach.

3.3 Monitoring during tunnelling under the subway

The electrolevel arrays within the subway structure were installed in February 1995, four months prior to the tunnelling and grouting work. The advance installation was to ensure that reliable, stable readings were obtained prior to the tunnelling. The readings were stable until, on the 13 May a movement of up to 12mm was recorded on the east wall of the subway box. At the time grout pipes were being jetted under the subway, and the movement was initially ascribed to the effects of jetting and an earlier incident when the lining of Shaft 1 buckled during installation. However, checks within the subway and of the data logger showed that the reading was erroneous. The error was a result of incorrect reprogramming of the data logger, which had resulted in compression of the calibration constants. As a result of this incident, it was decided to install subsurface settlement points directly onto the roof of the subway structure. These points would allow movements identified by the electrolevels to be checked without entering the subway. It was also recognised that the acute angle between the subway and the tunnel had made it difficult to extend the electrolevels far enough to ensure a stable datum; the points on the roof would provide an independent reference.

The tunnel passed under the subway between the 15th and 22nd of June. Primary control of the grouting was from observation of grouting pressure as the tunnel passed a grout pipe; the instrumentation was less useful in this respect than anticipated. The in ground electrolevels were found to be difficult to interpret, and the beam electrolevels moved very little. In all some $28m^3$ of grout was injected under the subway at very low pressures or, mainly, under gravity flow. On June 23rd the maximum measured movement on the subway structure was 2mm.

The monitoring of the instruments continued after the tunnel had passed this section. Continuing movement was recorded on the in-ground electrolevels installed from Shaft 3. One month after the main grouting 18mm of movement had been recorded below the structure, although movements within the structure were still less than 2mm. The ground movements accelerated in August, leading to a decision to carry out further grouting from Shaft 3. About $3m^3$ of grout was injected at pressures as low as 70kPa. After the additional grouting, no further movement has been recorded. Final movement of the structure has been restricted to less than 6mm.

4 DISCUSSION AND CONCLUSIONS

Monitoring the effects of tunnelling under Cedervale Park provided useful guidance on the need to improve tunnelling practise before passing under the subway structure. This information tied to a contractual requirement to meet specific values, lead to measures that substantially improved performance. However, part of the improvement was probably due to lowered water levels resulting from drainage into the tunnel.

It is also likely that part of the apparent improvement measured at the surface was due to ground losses not being transmitted through the thick cap of hard glacial Till. This is the likely explanation of the significant difference between the low value for relative volume loss measured in the approach to the subway tunnel (0 to 1%) and the 32 m^3 of grout injected, which represents about 10 per cent of the excavated volume of the tunnel.

The instrumentation placed in and below the subway structure proved to be of limited value in controlling the main grouting program, due to the high stiffness of the structure and the soils. The monitoring was, however, essential to verify the safety of the operational subway line. This instrumentation also identified continued movement in the soil below the subway in one area of the work. As a result of these movements, further grouting was undertaken to stabilize the subsurface movements.

ACKNOWLEDGEMENT

The client for the work was the Toronto Transit Commission. Staff of the Commission provided access and assistance while working in the existing subway structure. Project Managers for the work are Delcan-Hatch Joint Venture, who also provided site supervising staff.