Case study of a recovery shaft

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ABSTRACT
This paper presents a case history of a deep shaft constructed in disturbed sandy ground with the purpose of recovering a tunnel boring machine. Soil investigations were undertaken to establish soil conditions. Field instrumentation was used to monitor the lateral and vertical movements during excavation and backfilling. The findings reinforce the importance of soil investigation and field monitoring. Field monitoring results show that the actual settlement of the shaft is smaller than the theoretical prediction.

RÉSUMÉ
Cet article présente une étude de cas concernant l'observation du tassement des sols sableux dérangés par la construction d'une fosse profond. La fosse a été construite pour permettre à récupérer une machine tunnelier. Des investigations géotechniques on été enterprises pour établir les états de sol. On a employée de l'instrumentation pour mesurer les mouvements latéraux et verticaux des sols a cours de l'excavation et du remblai. Les résultats de cet investigation renforcent l'importance des investigations géotechniques, et des surveillences in situ. Ces résultats in situ montre que pendant la construction de la fosse, le tassement du sol était moins que dans les prédictions théorique.

1 INTRODUCTION

A significant amount of soil and water entered into the Langstaff Road trunk sewer tunnel just east of Timberview Drive in Vaughan, Ontario during tunnelling in May of 2008. The tunnelling boring machine (TBM) was flooded and had to be abandoned and a 25m by 30m sinkhole formed at ground surface near the westbound curb lane of Langstaff Road. The obvert of the 3.2m dia. tunnel lay at approximately 18.55m depth in the area of the sinkhole. The sinkhole was backfilled with approximately 267m^3 of unshrinkable fill, followed by an additional 600 to 800 m^3 of sand on the same day and the following weekend. A double bulkhead within the tunnel approximately 300m west of the sinkhole location was constructed at the same time. A shaft was constructed to recover the buried TBM and to complete the installation of the trunk sewer pipe in the disturbed section.

During the construction and backfilling of the recovery shaft, a few challenges were faced as follows:

- Ground condition evaluation;
- Geotechnical parameters for the shaft and sewer pipe support on disturbed ground;
- Long term stability of the shaft and sewer.

This paper describes the soil investigation and field monitoring prior to the shaft construction, during the shaft construction as well as during and after backfilling. The soil investigation and monitoring results are presented and discussed. Lessons obtained from the project are discussed.

2 SOIL CONDITIONS

Subsurface investigations by boreholes were carried out from the existing ground surface prior to the shaft construction as well as with borings advanced inside the shaft after the excavation reached the bottom of the shaft.

2.1 Soil Investigation Prior to Shaft Construction

Two boreholes were drilled just beyond the edge of the sinkhole and one borehole inside the sinkhole prior to shaft construction. All of the boreholes were sampled in association with the ASTM D1586 Standard Penetration Test (SPT) method using the conventional 50mm ID split spoon sampler at depth intervals of 1.5m.

All of the boreholes encountered a variable thickness of surficial pavement structure/fill deposits overlying a predominantly sandy silt to silty sand glacial till deposit extending approximately 5 to 6m below grade. Underlying the upper glacial till are predominantly cohesionless deposits ranging in texture from silt, sand and silt to gravelly sand. The SPT 'N' values measured in the cohesionless deposits in the two boreholes drilled outside the sinkhole were in excess of 50 blows per 300mm penetration and thus, considered to be in a very dense state of packing. No voids or zones of apparent loosening were encountered or inferred in these two boreholes drilled outside the sinkhole. At a depth of approximately 4.3m in the borehole inside the sinkhole, the augers dropped approximately 0.76m, suggesting a possible zone of loosening or void at this level. Groundwater was encountered within all of the boreholes at a depth of approximately 5.5 to 6m (El. 203.2 to 202.7m) below existing grade.

This investigation showed the soils outside the sinkhole were not disturbed. Unfortunately, soil investigation work was not carried out below the shaft base where the soil conditions were not clearly understood but soil disturbance due to ground loss was suspected. This raised concerns about the suitability of the soils at the shaft base to support the secant pile walls and weight of backfill as discussed in the following section.

2.2 Soil Investigation after Shaft Construction

In early 2009, a 5m by 30m recovery shaft was constructed at the location of the sinkhole to remove the buried TBM. The shaft consisted of contiguous 1m diameter caissons. The caisson walls were toed at 25.5m (El. 183.2m) below the existing ground surface. The buried TBM was found and removed when the excavation inside the shaft reached about 21.7m below ground level (approximate El. 187.0m), and then an approximately 600mm thick concrete slab was cast on the bottom of the excavation. It was noted that the collapsed tunnel segments, cables and some equipment were not removed and remained in the soils beneath the shaft base. Photograph 1 shows the recovery shaft. Prior to the construction of the recovery shaft, construction dewatering was commenced to draw down the groundwater levels to depths of 14.5 to 20.5m (El. 188.2 to 194.2m) below the existing ground surface. At a certain stage of the backfilling, the dewatering operations ceased (August 31, 2009) and the groundwater table was found to have recovered to between El. 196 and 197m in October, 2009.

Shortly after the casting of the concrete base slab inside the shaft in May 2009, permeation grouting using ordinary Portland cement was carried out to improve the soils below the concrete slab inside the shaft since these soils would have to support the trunk sewer pipes and shaft backfill. The proposed grouting program called for 80 grout holes at a spacing of 1.5m by 0.88m. The grouting was proposed to extend to 8m below the base slab surface (El. 179.6m). Unfortunately, grouting was unsuccessful due to insufficient soil information below the base slab. Of forty primary grouting holes, nineteen holes were grouted to depths ranging 5 to 8m below the concrete slab, seventeen holes were not grouted but simply backfilled due to encountering obstructions at depths ranging from 1 to 3.6m below the concrete slab, and four holes were not drilled.

Of the forty secondary holes, three were grouted to a depth of 8m below the concrete slab, one hole was not grouted but backfilled due to an obstruction found at a depth of 1m below the concrete slab, and the remaining thirty six holes were not grouted.

The area in which obstructions were found within 3.5m below the concrete slab coincided with the area where equipment and the precast concrete segmental tunnel liners were buried. The soil conditions below and within the buried equipment and segments could not be explored.

In order to establish soil conditions under the base slab and also outside the caisson walls, two boreholes were advanced to a depth of 27.1m (El. 181.6m) below the existing ground surface outside the shaft and two boreholes were drilled from the concrete slab to depths of 3.5 to 9.7m (El. 184.1 to 177.9m) below the slab inside the shaft. These boreholes were sampled in association with SPTs at depth intervals of 0.75 to 1.5m. Figure 1 shows the simplified soil profile and variation of SPT ‘N’ values with depth. The ground water level (GWL) before and after dewatering are also shown in Figure 1.

The boreholes drilled outside the shaft encountered 4.6 to 6.7m of fill overlying predominantly cohesionless deposits ranging in texture from silt to gravelly sand. From the existing ground surface to the base of shaft, the

Photograph 1. Recovery shaft

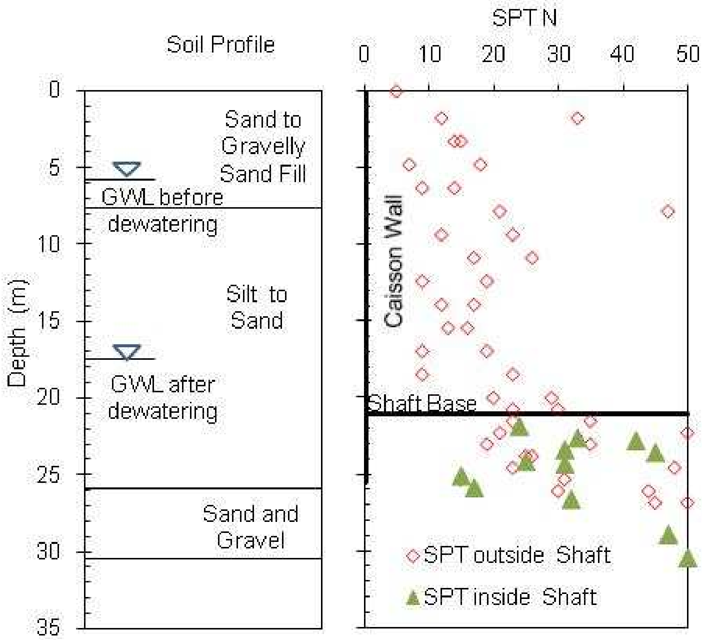

Figure 1. Soil profile and SPT ‘N’ values

SPT ‘N’ values measured in the cohesionless deposits ranged from 9 to 47 blows per 300mm penetration and thus, indicating loose to dense conditions. From 21.1 to 25.5m below the existing ground surface, the soils were compact to very dense, as inferred from SPT ‘N’ values of 15 to greater than 50 blows per 0.3m penetration. Below the toe of the caisson walls, the soils were dense to very dense with SPT ‘N’ values of 30 to greater than 50 blows per 0.3m penetration. No voids were encountered or inferred in any of the boreholes.

The boreholes drilled inside the shaft encountered 0.6 to 0.7m thick concrete overlying a predominantly cohesionless deposits ranging in texture from silt to gravelly sand. The soils were compact to very dense, as inferred from SPT ‘N’ values of 15 to greater than 50 blows per 0.3m penetration. One of the two boreholes encountered an obstruction, probably a buried segment at a depth of 3.5 below the slab (El. 184.1m).

The ground investigation showed that it was unlikely that voids existed below the buried liner segments and equipment. However, the soils below the base of shaft to 5m below the base were considered to be disturbed relative to the original condition of the ground.

Based on the investigation results, the existing ground conditions below the base slab of the shaft were not deemed to be sufficiently competent to support the weight of the pipe and over 20m of backfill without the risk of unacceptably large settlements (greater than 25mm). This finding turned over the initial design assumption that soils below the base slab improved by grouting could support the pipe and backfill.

Two options were proposed to solve this problem. The first option was to support the pipe and backfill by the shoring caisson walls which were socketed into dense to very dense sandy soils and were considered to have sufficient capacities to support the backfill. However, this approach needs specialized connections between the pipe or grade beams and the walls and was uneconomical.

The second approach was to consider the pipe supported by both base slab and the caisson walls. A 30 MPa concrete mass to form the cast-in-place sewer within the shaft would replace the tunnel. The thickness of the concrete mass was approximately 4.6m including an overlying, 1.4m thick concrete cover. Then the remainder of the shaft would be backfilled with 0.2 MPa unshrinkable fill. The soil below the base slab was considered to have a bearing capacity of 150 to 200 kPa under serviceability limit state. The structural connection between the base slab/mass concrete and walls through the welding of steel struts to the king piles was sufficient to transfer the remaining loading to the caisson walls. The estimated settlement under the load of backfill was less than 25mm. This approach was simple and easily constructed and thus adopted. In order to verify the approach, settlement monitoring was carried out through the whole process of pipe casting and shaft backfilling.

3 FIELD MONITORING

A comprehensive ground movement monitoring program was carried out prior to and during the shaft construction and after backfilling.

3.1 Ground Settlement Prior to Shaft Construction

A series of ground surface settlement monitoring points were established on the surface of the nearby roadway/sidewalk and the top of the sinkhole backfill.

The monitoring results showed that the majority of the ground displacement occurred generally within about a week of the initial loss of ground for the cohesionless sands and silts.

3.2 Lateral Movement of Caisson Wall

Two inclinometers were installed behind the caisson walls prior to shaft excavation. The inclinometers were monitored during and after the excavation. Figure 2 shows the monitoring results of one inclinometer. The maximum cumulative lateral deflection of the wall was 8.1mm occurring 4m below the existing ground surface. At the excavation level, the lateral deflection of the wall was 2.8mm. It is also noted that there was 2mm lateral movement in the soil at the bottom of the caisson wall. The lateral movement measured in the second inclinometer was also not greater than 8.1mm.

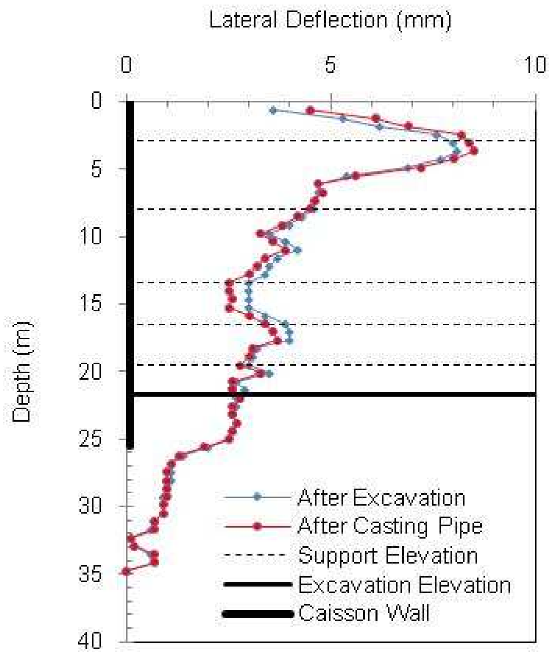

Figure 2. Lateral deflection of caisson wall

3.3 Monitoring of Shaft during Pipe Casting and Backfilling

Four surface settlement monitoring points were established on the top of the shaft wall at the shaft corners prior to the casting of the sewer pipe. The locations of the four monitoring points labelled as NE, SE, SW and NW are shown in Figure 3. The settlement points were monitored prior to pipe casting and during pipe casting and shaft backfilling. Figure 4 shows the monitoring results. The results show negligibly small movements in the time period between pipe casting and the shaft backfilling to 0.5m below the ground surface (BGS). The recorded movements of the wall, ±4mm are within the accuracy of the survey. It is also noted that the ceasing of construction dewatering did not affect the movement of walls. The lateral movement of the walls was less than 1mm during the pipe casting as shown in Figure 2. Thus the shaft wall can be considered not to have moved during the backfilling.

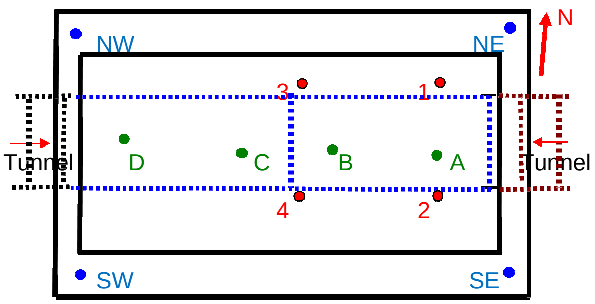

Figure 3. Locations of monitoring points.

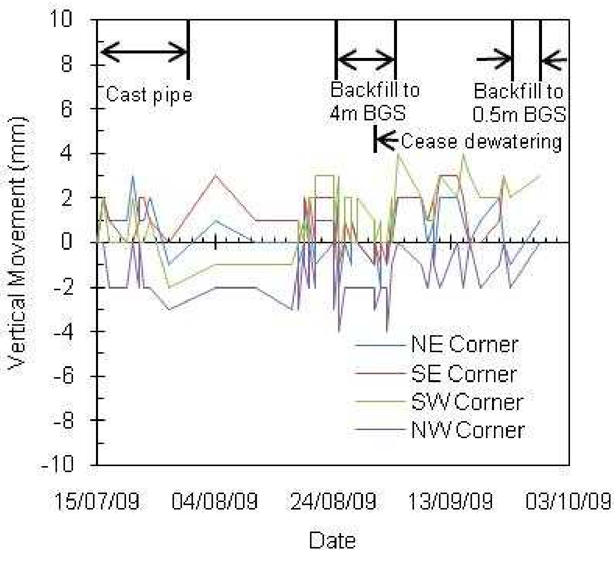

Figure 4. Vertical movements of wall during pipe casting and shaft backfilling

Four in-ground settlement monitoring rods were installed through the concrete base slab with the settlement rod base plates set below the underside of the slab prior to casting of concrete mass to form the cast-in-place section of the sewer. The settlement rods were sleeved above the base plate through the concrete base slab and pipe casting zone to allow free movement of the rod. The locations of the monitoring rods labelled as 1, 2, 3 and 4 are shown in Figure 3. The monitoring rods were monitored prior to, during and after pipe casting. Figure 5 shows the monitoring results. The settlement rods show negligibly small movements. The recorded movements of the base slab, ±2mm are near to the accuracy of the survey, prior to and after the concrete casting. Since the observed movements are within the precision of the monitoring, it can be concluded that the casting of the mass concrete to form the cast-in-place concrete sewer pipe caused no settlement.

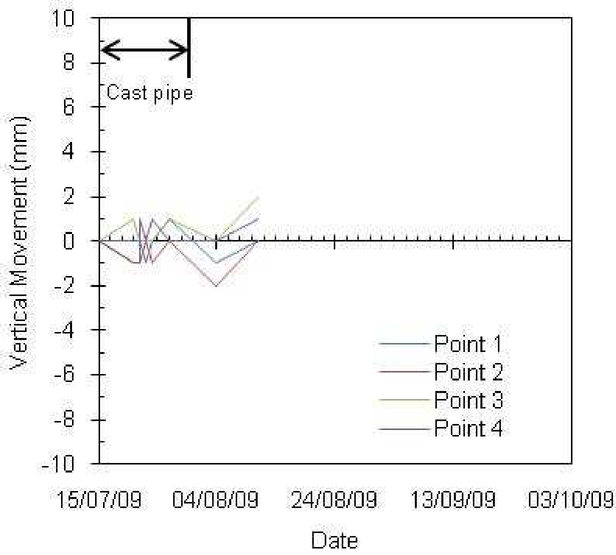

Figure 5. Vertical movements of base slab during and after pipe casting

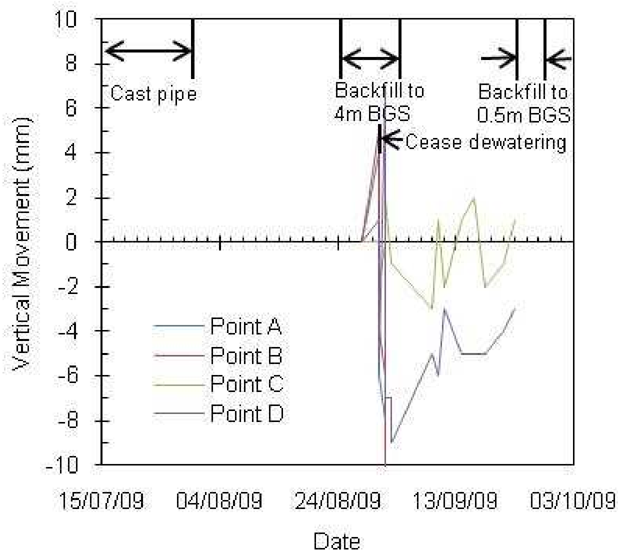

Figure 6. Vertical movements of base slab during backfill to 0.5m below existing ground surface

Prior to backfilling, four settlement monitoring rods were installed along the sewer pipe alignment on August 28, 2009. These settlement monitoring rods were installed in the concrete mass that formed the cast-in-place sewer inside the shaft. The location of the four settlement rods labelled as A, B, C and D are shown in Figure 3. Figure 6 shows the monitoring results. On September 3, 2009, no settlement was observed at monitoring points A, B and D; the settlement at monitoring point C was 1mm which is well within the precision of the survey. During the period from August 28 to September 3, 2009, a maximum settlement of 7mm was observed.

The invert and obvert elevations of the cast-in-place sewer pipe were monitored before, during and after backfilling at 9 locations. Photograph 2 shows the settlement monitoring inside the sewer pipe. Figure 7 shows the monitoing results. During backfilling from August 25 to December 15, 2009, the settlement at invert level ranged from 0 to 2mm and the apparent heave was from 0 to 4mm; the settlement at obvert ranged from 0 to 6mm and the apparent heave was from 0 to 1mm. Since all these movements are within the accuracy of the survey one can conclude that neither the shaft base slab nor the cast-in-place sewer settled during the backfilling of the shaft.

Photograph 2. Settlement monitoring inside cast-in-place sewer pipe

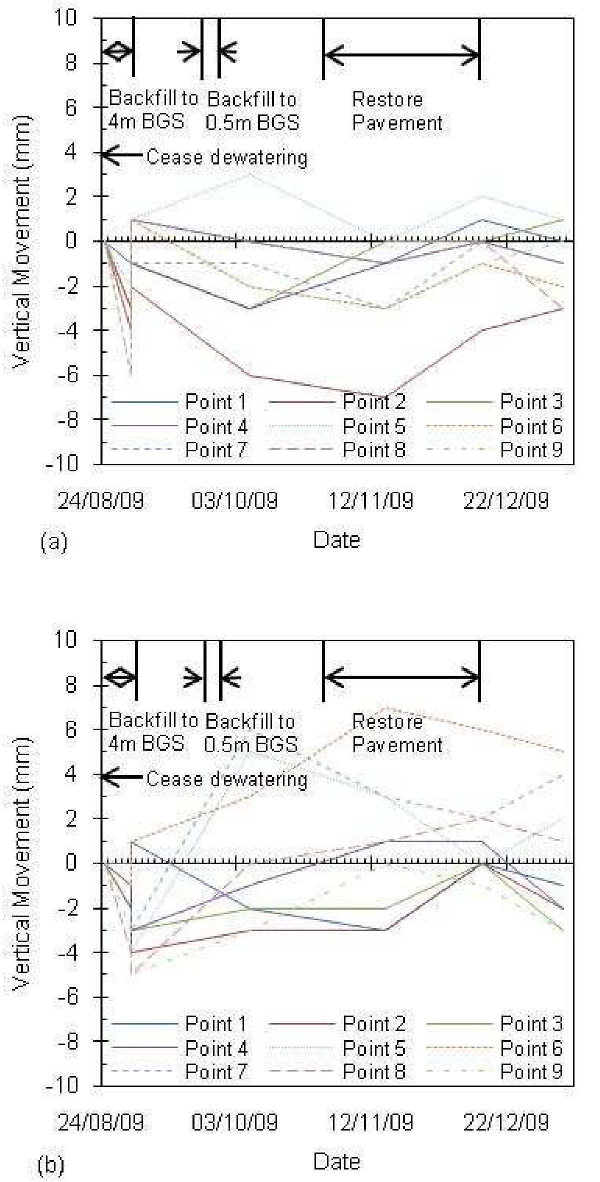

Figure 7. Vertical movements of cast-in-place sewer pipe (a) invert and (b) obvert

From the end of the backfilling of the shaft and the restoration of the pavement to January 8, 2010, the monitoring of the invert and obvert levels of the cast-in-place sewer showed that the movement of the sewer at the location of the recovery shaft was ±3mm, which is within the accuracy of the survey. The sewer and the shaft were therefore considered to be stable.

3.4 Monitoring of Ground Surface during and after Backfilling

The surrounding area of the shaft was monitored using the in-ground settlement points during and after the backfilling of the shaft. The monitoring results show that no significant ground movement occurred from April to December 2009 which was prior to and after the backfilling of the shaft. The maximum ground movement at these monitoring points was less than 5mm during the backfilling of the shaft. Thus the ground surrounding the shaft is considered to be stable.

Surface settlement monitoring points were established on the surface of the restored pavement at the location of recovery shaft in January of 2010 and monitored from January to April, 2010. No ground settlement was observed. A maximum 5mm heave was observed. The heave could be due to the ground frost in winter.

4 CONCLUSIONS

Sufficient soil investigation prior to the structural design and construction is extremely important. The structural design had to be modified and the soil improvement was ineffective in this project due to insufficient soil investigation.

Field monitoring data was vital to the evaluation of the soil bearing capacity. From the result of field monitoring, it was found that the majority of the ground displacement in the cohesionless sands and silts occurs relatively rapidly, i.e. within about a week of the initial loss of ground. The negligibly small movements of the shaft under the loading of over 20m of backfill in this project could be due to the following reasons:

- The modulus of disturbed soil after preloading is much higher than that without preloading;
- The loading of backfill was taken by both the resistance of soils below the shaft base and the friction between the walls and soils. The loading actually transferred to the base of shaft walls may have been very small.

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