University of Alberta

Performance of EPB-TBM in Mixed Face Conditions:

City of Edmonton South LRT Extension – University Station to Health Sciences Station

by

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Abstract

The Edmonton South LRT Extension involved the construction of two 6.3m tunnels in mixed-face condition using a Lovat EPB-TBM. Three structures overlaid the tunnel alignment, and four major stratigraphic layers were identified in the tunnelling zone. The stratigraphic layer of greatest concern due to the risk of excessive settlements was the Outwash Sands and Silts.

Assessment of the performance of the EPB-TBM was carried out using information recorded during construction. Soil conditioning increased the moisture content of the spoil by 8% on average. Mining times were between 18 and 24 minutes in Outwash Sands and Silts, 15 and 20 minutes in Glacial Till, and 30 to 50 minutes in Bedrock. Settlements due to over-mining were well controlled and only observed in the initial 100m of Southbound construction. The largest source of settlement was the tail shield-segmental lining transition. Improvements in grouting procedure during construction reduced settlements by ~40%. At the end of construction the Utility Corridor, Education Car Park, and St. Joseph's College had maximum settlements of -15.8mm, -20.5mm, and -7.8mm respectively.

Preface

At the end of the third year of my undergraduate degree it was mentioned in one of the classes I was enrolled in that the Construction Engineering department was looking for two summer students. In need of a job for the summer I applied for the position, and was hired. On my first day of work in May, 2001, I was asked, "Are you claustrophobic", I replied, "No." I spent the next 4 months below ground with the City of Edmonton's tunnelling section as they built the North and South Edmonton Sewer Trunks.

In the spring of 2003 I learned the thesis topic I had been assigned was the Edmonton South LRT Extension. For the opportunity to be involved with tunnelling in Edmonton a second time I would like to thank a number of people.

I would like to thank my supervisor, Dr. Derek Martin. His exceptional patience, advice and expertise were an immense asset in accomplishing this research. I can only imagine what torment I put him through.

I am grateful for the assistance and co-operation extended by Rokus Broere and UMA Group. I would also like to thank Aecon-McNally for providing the computer logs recorded by the EPB-TBM during construction.

I would like to thank Daryl Krupa for a series of discussions on the geomorphology of the Edmonton region that I had with him in 2003. I would also like to mention Doreen Foster, my editor, who was immensely helpful in the completion of this work.. I would lastly like to thank Jennifer Li, who acting as my right hand and long arm helped to finish this work off since I move to British Columbia.

I extend my warmest regards to the faculty and staff of the Geotechnical Group of the Department of Civil and Environmental Engineering, you made my time here interesting and fun.

Table of Contents

List of Tables

List of Figures

1.0 Introduction

1.1 Introduction to Edmonton SLRT Project

Construction of the Edmonton Light Rail Transit (LRT) system began in the mid 1970s as part of a general infrastructure improvement that coincided with the City of Edmonton hosting the Commonwealth Games in 1978. The initial line included 5 stations, from Central Station to Belvedere Station (Figure 1-1, p 5). The line used a double track running from the downtown area towards the north east end of the city. Between 1981 and 1989 the line was extended and four stations were added to the Edmonton LRT system, extending the reach of the line from Clareview Station to Grandin Station. In 1992, the LRT system was extended across the North Saskatchewan River valley with the addition of a tenth station at the University of Alberta.

Prior to the completion of the University Station in 1992, two geotechnical investigations (1986, and 1991) had been undertaken along the proposed alignment extending from the University Station to the proposed Health Sciences Station (Figure 1-2, p 5). However a period of economic retrenchment in the early 1990s, greatly reduced the funds available for infrastructure improvements. As the general economic climate improved towards the year 2000, increases in capital funding allowed the City to move the proposed extension of the Edmonton LRT from the University Station to the Health Sciences Station towards becoming a reality.

The Edmonton South Light Rail Transit (SLRT) extension from the University Station to the Health Sciences Station is one component of a multiphase project to extend LRT service to the south of the city, nearly doubling the amount of track in service (Figure 1-1 and 1-2, p 5). For economic reasons, the City decided to construct as much of the extension above grade as possible. This decision required raising the track from a depth of 25m at the University Station to the surface at the Health Sciences Station over a distance of 355m, resulting in a 7% grade.

Construction of the portal began in March of 2003 with completion of the basic portal in early June of 2003. Construction of the removal shaft at the end of construction began in early June of 2003 and was complete in late August. Assembly of the TBM began in early June of 2005 with the completion of portal excavation and launch area. Construction of the Southbound tunnel began in August of 2003 but technical issues delayed significant production until September 14, 2003. Completion of the Southbound tunnel was on November 8th. Construction of the Northbound Tunnel took place between the 7th of January and the 7th of March, 2004. Upon completion of the tunnels work shifted towards fitting out the tunnels with the intention of brining the extension into service in 2006.

In traversing the distance between the University Station and the Health Sciences Station, the tunnel alignment passed under two structures, St. Joseph's College and the Education Car Park. St. Joseph's College, completed in 1926, is of masonry construction using a shallow foundation, the Education Car Park is a modern concrete parking facility. Ensuring that the settlements resulting from the construction of the twin tunnels of the Edmonton SLRT extension did not damage the overlying structures was a major concern.

Four major stratigraphic components were identified along the tunnel alignment from the University Station to the Health Sciences Station: (1) Lake Edmonton Clay, (2) Outwash Sands and Silts, (3) Glacial Till and (4) Bedrock. The Lake Edmonton Clay forms an overlying mantle that is between 4 and 6m thick in the project area. Deposited post-glacially during the existence of Glacial Lake Edmonton, Lake Edmonton Clay is a medium to low plastic, varved clay that fines upward. As the Lake Edmonton Clay forms the overlying mantel, tunnelling through this material was not anticipated. The construction of the launch portal and the removal shaft for the Tunnel Boring Machine (TBM) constituted the only two aspects of the SLRT extension that involved significant activity within the Lake Edmonton Clay.

The Outwash Sands and Silts underlie the Lake Edmonton Clay, and are between 6 and 10m thick over the project area. Deposited during the retreat of the last glaciation, the

Outwash Sands and Silts were deposited from a series of changing glacial outflows in the Edmonton region. Deposited hydraulically, the Outwash Sands and Silts are a medium-dense to dense sand with SPT(N) values of between 15 and 40 in the tunnelling zone. The in-situ moisture content of the material ranged between 2% and 10% with siltier regions of the material having moisture contents as high as 20%. Tunnelling within the Outwash Sands and Silts was anticipated over 140m of the tunnel alignment, between Station (Stn.) +690m and +550m, during the construction of the Southbound and Northbound Tunnels.

The Glacial Till underlies the Outwash Sands and Silts and forms a layer between 6 and 9m thick over the project area. The Glacial Till was deposited sub-glacially during the Wisconsin glaciation. It is heavily over-consolidated and contains numerous water charged pockets of intra-till sand. The undrained shear strength of the material is between 200 and 300 KPa in the tunnelling zone with SPT(N) values of between 30 per 300mm and 50 per 50mm. Tunnelling within the Glacial Till was anticipated from the region of Stn. +640m through the end of construction in the region of Stn. +400m.

The Bedrock present over the tunnel alignment is part of the Edmonton formation and is composed primarily of claystone with limited quantities of sandstone. During the glaciation of the Edmonton region, the Bedrock was heavily weathered and deformed, resulting in numerous slickensides in the upper reaches of the bedrock. The weak Bedrock is a hard to very-hard cohesive soil with observed SPT(N) values greater than 48 blows per 300mm in the claystone, and 57 blows per 300mm in the sandstone. The maximum number of observed blows in both regions was in the order of 50 blows per 25mm. Tunnelling within the Bedrock was anticipated from Stn. +525m in the Southbound Tunnel and from Stn. +475m in the Northbound Tunnel through the end of construction.

Tunnelling within the Outwash Sands and Silts presented the greatest risk associated with the Edmonton SLRT extension. The Outwash Sands and Silts, susceptible to flowing ground conditions and over-mining, would occupy a significant portion of the TBM face when the TBM began to advance beneath the Education Car Park. This vulnerability to over-mining and subsequent settlement made the control of settlements a primary consideration of the Edmonton SLRT extension.

Through the tendering process, a joint venture was established with PCL Maxam taking the position of general construction contractor and Aecon-McNally taking the position of primary tunnelling contractor. UMA Group and Stantec were retained as consultants on the project. Construction of the Edmonton SLRT extension was accomplished using a 6.56m Equalised Pressure Balance Tunnel Boring Machine (EPB-TBM) manufactured by Lovat.

1.2 Objective and Scope

This thesis consists of an evaluation of the construction of the Edmonton SLRT extension, from the existing University to the proposed Health Sciences Station. Two aspects of the performance of construction provided the focus of this thesis.

The first aspect of the evaluation was a review of the performance of the Equalised Pressure Balance Tunnel Boring Machine as it traversed the different strata. This review was accomplished by examining the information recorded manually, and by computer data logging, during construction of the Southbound and Northbound Tunnels. The reviewed information included the properties and quantities of the excavated soil, grout injection procedures, operational torque during advance, excavation times, and face pressure.

The second aspect of the evaluation was a review of the settlements observed during construction and an assessment of the performance of EPB-TBM operation designed to mitigate settlement (grout injection and face pressure). The review also takes into consideration the impact of geology on settlement.

Figure 1-1 Edmonton LRT Line prior to construction of SLRT Extension.

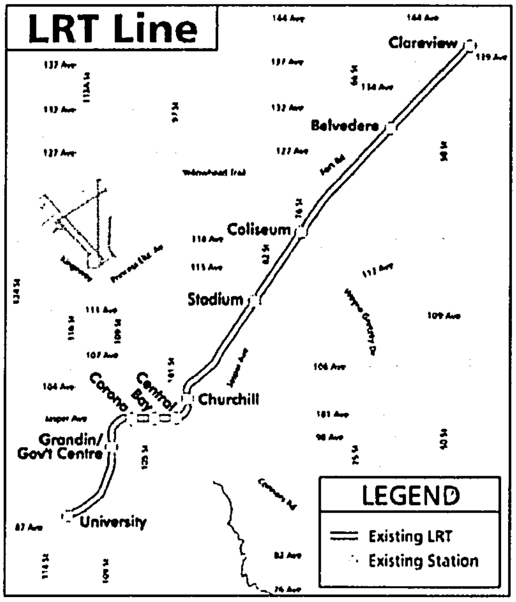
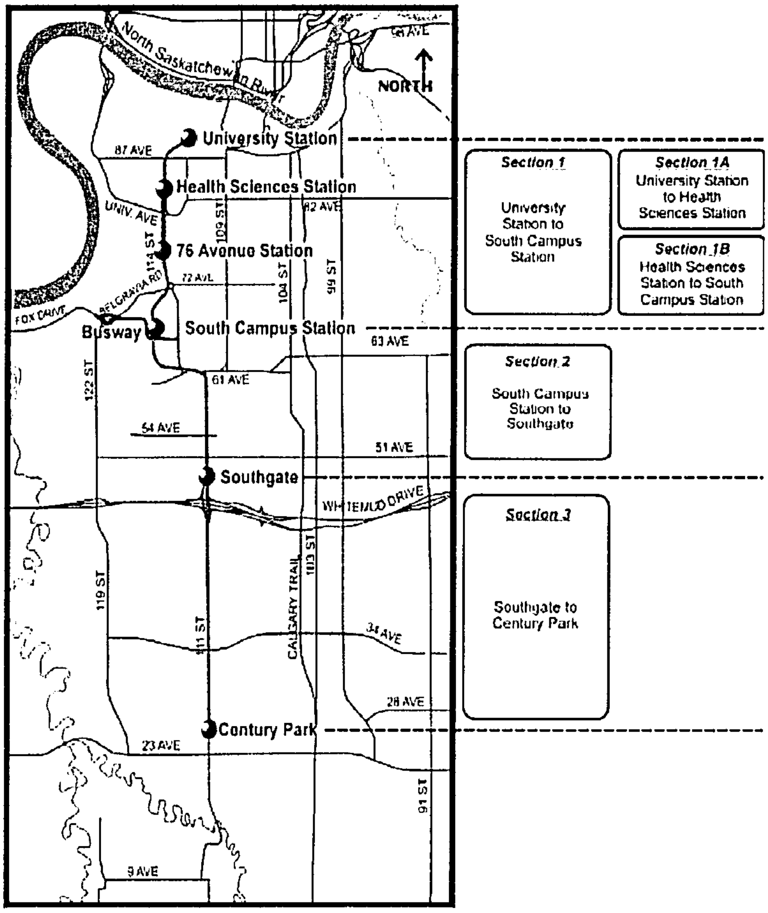


Figure 1-2 Proposed Edmonton SLRT Extensions.



1.3 Outline of Thesis Content

Section 1 provides an introduction to the issues surrounding the Edmonton SLRT project. This introduction includes a brief history of the Edmonton SLRT system, and the technical challenges associated with the construction of the University-Health Sciences Station extension.

Section 2 provides an introduction to the operation of Equalised Pressure Balance Tunnel Boring Machines (EPB-TBMs). The discussion focused on the design and components specific to EPB-TBMs, the control of ground movements associated with the operation of EPB-TBMs, and the specific properties of the EPB-TBM used on the Edmonton SLRT project.

Section 3 investigates the geology encountered along the Edmonton SLRT site. The extents, formation process, and characterisation of the Lake Edmonton Clay, Outwash Sands and Silts, Glacial Till, and Bedrock are examined in this section.

Section 4 presents the TBM related information collected during the construction of the Southbound and Northbound Tunnels of the Edmonton SLRT project. The sources considered were the excavated soil, the information logged manually and by the TBM computer during tunnelling, the observed settlements induced by tunnelling, and the strength development of the grout used for tail shield grout injection.

Section 5 evaluates the TBM measurements and observed settlements as a measure of performance. The process control of soil conditioning, advance rates in changing ground conditions, operational cutting head torque, face control, effectiveness of grout injection, and geology as factor in settlement were considered in this section.

Section 6 summarises the findings in the thesis and provides conclusions on both the assessment of TBM performance, and the control of settlements.

2.0 Introduction to Equalized Pressure Balance Tunnel Boring Machines

Modern tunnelling techniques are separated into two broad groups; hard and soft ground tunnelling. Hard ground tunnelling can be characterized as tunnelling through competent materials which can remain unsupported for considerable periods of time without undue risk of collapse. In contrast, soft ground tunnelling operates at relatively shallow depths in surficial deposits of various origins. Soft ground tunnels typically must be supported at the face and along the tunnel length to prevent collapse.

Modern soft ground tunnelling is often carried out using open- or closed-face tunnel boring machines (TBM). When first put into service open-faced TBMs represented a considerable advance in the state of tunnelling. However, the open-face design imposed a number of restrictions on the operation of such machines. As the excavation face is exposed to the atmosphere, the use of open-face TBMs is limited to ground conditions where the soil is sufficiently stable to allow for the installation of the ground support. The next advance in soft ground tunnelling was the development of closed-face TBMs. By isolating the excavation face from the atmosphere, closed-face TBMs permitted the maintenance of a controlled positive pressure within the isolated zone. By providing a passive pressure to support the excavation face, the range of soil conditions practical for tunnel construction was increased over those for open-face tunnel boring machines.

The first use of an Equalized Pressure Balance Tunnel Boring Machine came in 1974, in Tokyo, Japan (Goss, 2001). Built by the Ishikawa-Jima-Harima company, the design was based on contemporary slurry-shield TBMs. Slurry-shield TBMs were of a closed-face design which typically used injected bentonite slurry to provide positive support to the face of the excavation. Slurry-shield TBMs worked well in coarse grained soils but had significant drawbacks in fine grained soils as the separation of fines from the bentonite slurry was difficult. As construction in Tokyo involved working with a large proportion of fine grained soils, the EPB-TBM was developed.

Figure 2-1 (p 9) shows a typical cross-section of a closed-face EPB-TBM. In an EPB-TBM, the soil is passed from the cutting head into the excavation chamber. Within the excavation chamber, the soil is mixed by the rotating action of the cutting head and conditioned by the controlled injection of foams and polymers. The resulting mixture of soil and conditioning agents (spoil) is removed through a screw conveyor fixed at the bottom of the excavation chamber. The screw conveyor allows a pressure balance to be maintained between the excavation chamber and the end of the screw conveyor where the spoil is deposited onto a conveyor belt (Figure 2-2, p 10). The conveyor belt at the end of the screw conveyor transports the spoil along the length of the tunnel to be deposited in rail cars that will ultimately remove the spoil from the tunnel.

The advance of the TBM through the ground is accomplished through the use of hydraulic rams which press against the completed tunnel lining. Once a sufficient amount of soil has been excavated, the hydraulic rams of the TBM can be retracted and the tunnel extended the length of the advance, using pre-cast concrete segments. Typical advances for EPB-TBMs are similar to open-faced TBMs, between 1 and 1.5m, depending on the design and size of the TBM. Once the construction of the tunnel lining, typically using pre-cast concrete, is complete the tunnel cycle starts over.

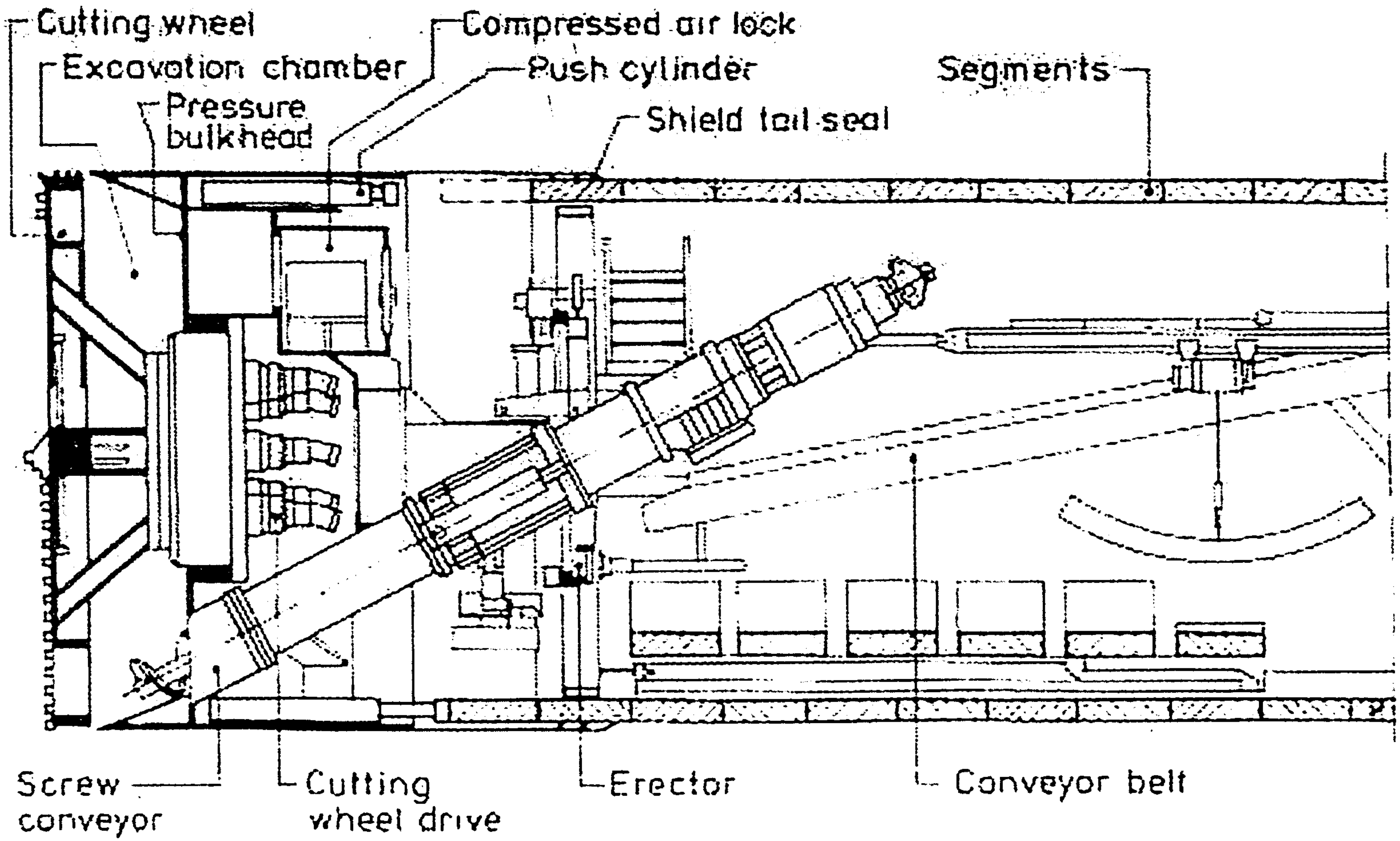


Figure 2-1 Generalised cross-section of an Equalised Pressure Balance Tunnel Boring Machine (Maidl et al., 1996)

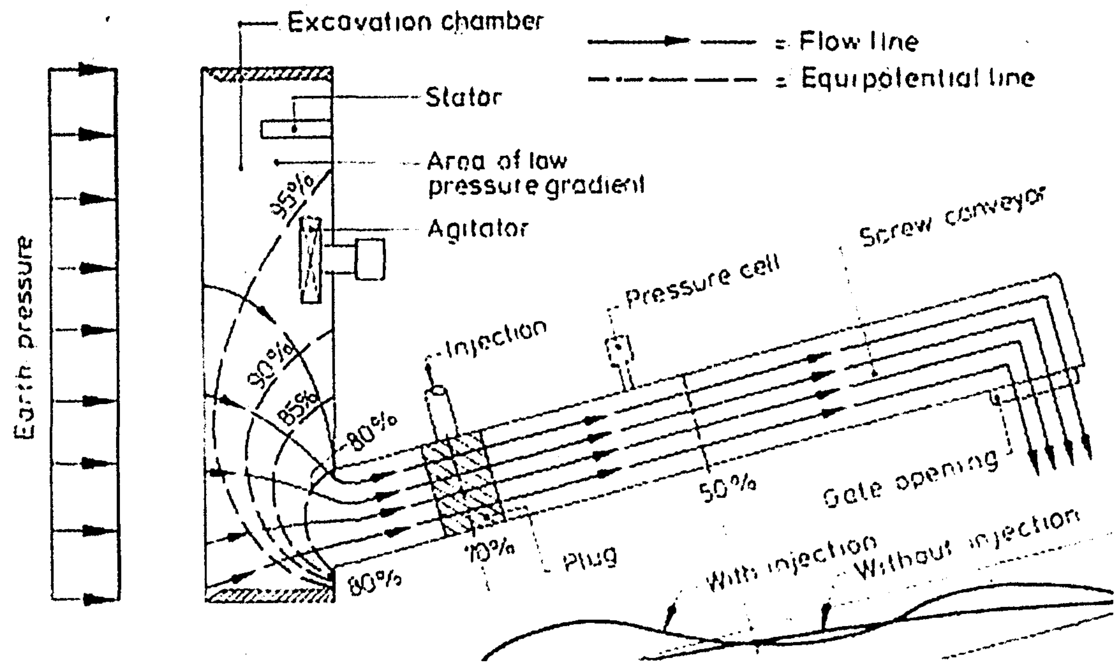


Figure 2-2 Generalised pressure drop in EPB-TBM machine from the face through the screw conveyor (Maidl et al., 1996.)

When operating EPB-TBMs, effective soil conditioning is a significant factor in ensuring adequate face control. In slurry shield TBMs, injected slurry (typically bentonite-based) is used in the excavation chamber as a medium to provide positive pressure to the excavation face. As the slurry is manufactured, it can be made to specification during tunnelling. In the case of EPB-TBMs, the excavated soil is used as the medium to provide positive support to the face. Conditioning of the soil into a suitable medium is accomplished through the controlled injection of two primary conditioning agents: surfactants, and polymers. The resulting mixture of conditioning agents (~1% by volume), water (~9% by volume), and air (~90% by volume) is typically referred to as foam (Milligan, 2000).

These conditioning agents are added in controlled quantities to a volume of water. This mixture of conditioning agents and water is pumped towards the face of the EPB machine, where it is eventually charged with compressed air in the foam lances. The foam lances are designed to provide turbulent mixing of the liquid and gas phase of the foam and typically contain wire mesh to assist in the creation of a uniform bubble size. Upon mixing, the foam is pumped into the excavation chamber and the screw conveyor. Injection of foam into the excavation chamber of the EPB-TBM is carried out through a number of injection ports scattered across the bulkhead of the machine. Foams can also be injected into the screw conveyor. Foam injection into the conveyor ensures a flow-able spoil that reduces the necessary torque and wear on the screw conveyor (Figure 2-3, p 11).

The effect of foams in EPB tunnelling is to lubricate the excavated soil allowing it to flow more readily (Milligan, 2000). This lubrication of the soil allows a more homogeneous mixing in the excavation chamber, ensuring a more even application of pressure against the face and improving face stability. Spoil viscosity is also important in ensuring that the spoil in the screw conveyor is able to maintain the pressure balance required during tunnelling. Foam treatment also reduces the wear on the cutter-head and screw conveyor by diminishing the effective cohesion of the soil by the lubricating properties of the foam injected.

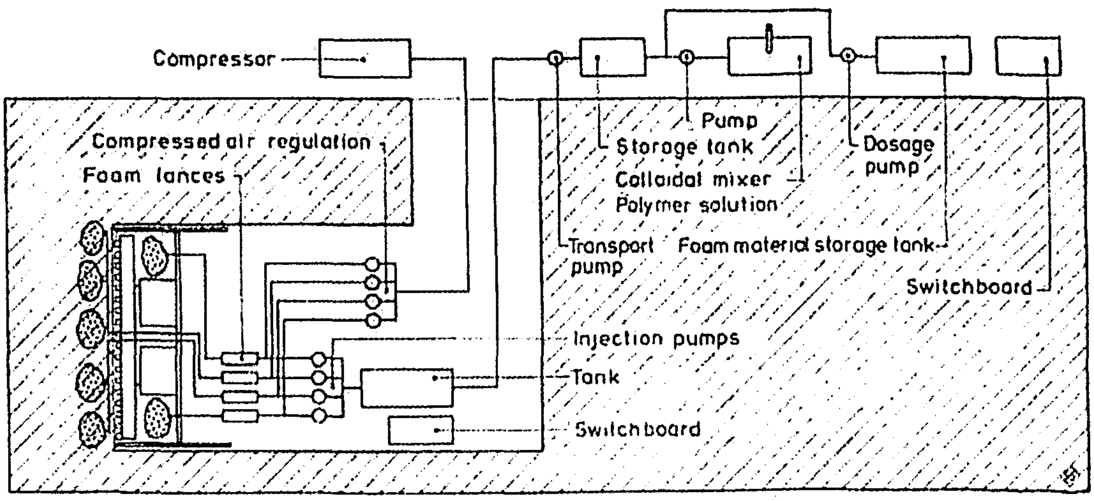


Figure 2-3 Generalised schematic of foam plant (Maidl et al. 1996)

Surfactants, include materials such as soaps and are available commercially possessing a broad array of chemical characteristics (anionic, cationic, non-ionic or amphoteric properties). The environmental impact associated with surfactants can be negligible. In Japan (Mitsubishi, 1991), and for the Sheppard Subway extension in Toronto (Goss, 2001) the surfactants used for construction were examined for toxicity. In both situations the chemical composition of the foams met the environmental standards of each nation.

The development of effective water soluble polymers suitable for tunnel construction did not come about until 1994 (Goss, 2001). While polymers as additives had been used previously, it was not until this advent of effective water soluble polymers that the application of polymers in EPB tunnelling became widespread. Polymers are introduced in controlled quantities to the excavation chamber through the foam stream. Typically used in coarse grained soils, polymers serve as binding agents between the large grains. This binding reduces the hydraulic conductivity of the soil in the excavation chamber thus helping to reduce the inflow of water. These polymers also provide the cohesion necessary for coarse grained soils to maintain a pressure balance over the length of the screw conveyor. The addition of polymers also stabilizes the foam, greatly increasing the time needed for air bubbles to break down while in the excavation chamber. This stabilising property is of benefit during periods of slow advances, or periods of shutdown. Like the surfactants commonly used in tunnelling, the polymers used do not pose a significant environmental threat.

2.1 Control of Ground Movements using EPB-TBM's

Tunnel Boring Machines of any type will induce some degree of settlement as they advance through the ground. Figure 2-4 (p 13) shows the general character of this settlement for an EPB-TBM and can be summarised as follows (Kunito and Sugden, 2001).

- Point a - Settlements ahead of the face of the TBM are the result of over-mining during the advance of the TBM.
- Point b - Settlements along the shield of the TBM are a function of the cut at the face being a larger diameter than the tail shield.
- Point c - Settlements at the transition between the tail shield and the segmental lining are the result of the variation in diameters between the two linings.
- Point d - Additional settlements after the passage of the tail shield of the TBM can be viewed as initial lining deflections.
- Point e - Long term settlement.

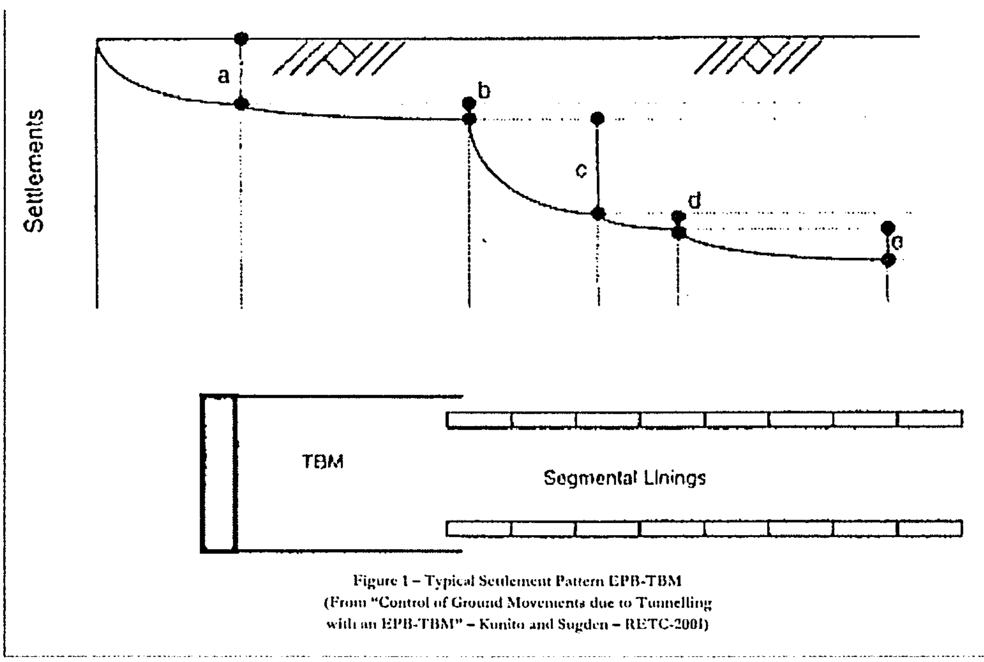


Figure 2-4 Generalised settlement profile for EPB-TBM, by region (Kunito and Sugden, 2001)

2.1.1 Ground Movements Ahead of the Face

Ground movements ahead of the face are a function of the balance between the earth pressures in-situ and the pressures maintained in the excavation chamber of the TBM by the conditioned soil (Figure 2-5, p 14). In order to achieve complete control over the system, the pressure maintained in the excavation chamber of the TBM should be as large as the combined earth and water pressure acting on the face. However, in the operation of an EPB-TBM, the pressure maintained in the excavation chamber should be large enough to prevent the active failure of the face, and the large soil movements expected as the excavation chamber pressure nears the active failure criteria. Passive failure of the soil face will result in the heaving of the ground ahead of the EPB-TBM. This mode of failure is likely only at shallow depths, when the over-thrusting of the TBM during advance can cause the soil face to fail passively. By increasing the overburden the likelihood of this failure mode is reduced.

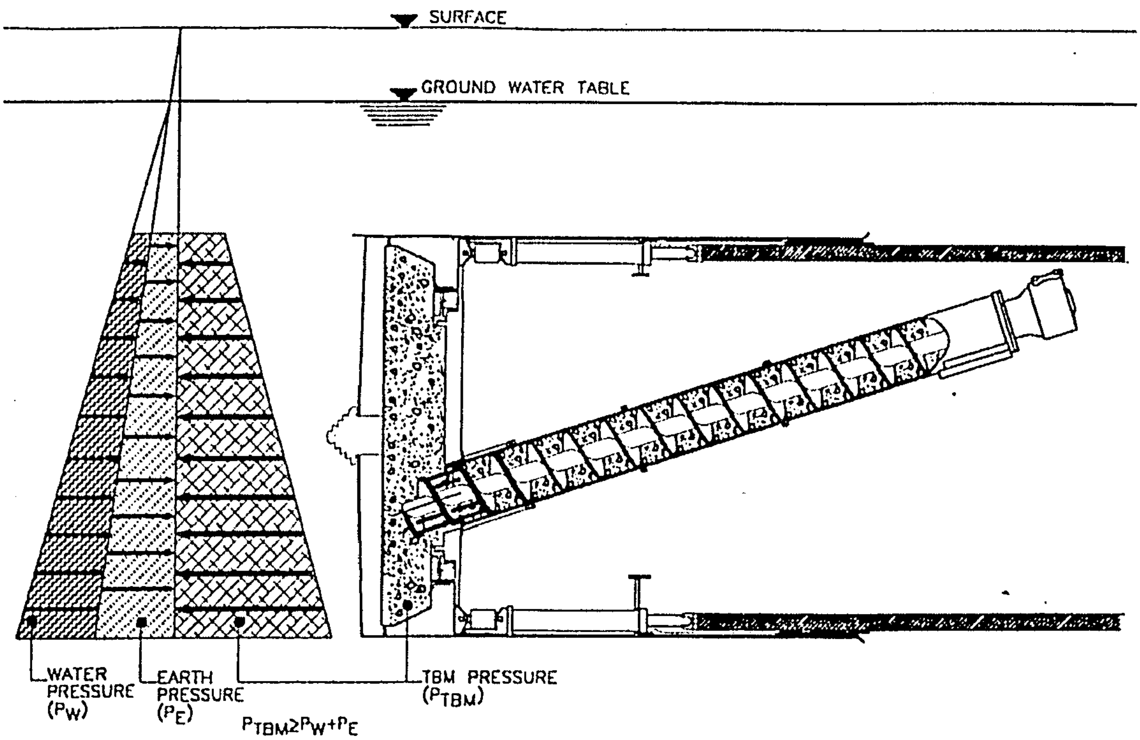


Figure 2-5 Principle of Equalised Pressure Balance (Lovat)

The most relevant parameter in ensuring even distribution of the face pressure is the effectiveness of the soil conditioning on the mined soil. Poor soil conditioning increases the heterogeneity of the soil in the excavation chamber. This heterogeneity hinders the application of an even pressure through the excavation chamber. Lack of even pressure across the excavation face increases the likelihood of soil movement into the excavation chamber, increasing the possibility of settlement ahead of the face of the TBM.

Provided adequate face pressure is maintained during the operation of an EPB-TBM, it is possible to minimise the amount of over-mining. In Japanese experience, face losses on the order of 0.5% and less were readily obtainable provided effective controls were in place (Kunito and Sugden, 2001). However, in case of poor control or exceptionally difficult ground, the degree of over-mining can increase considerably (Figure 2-6, p 16). During the Anacostia River Tunnel project for the Washington Metro, over-mine rates on the order of 5%, 8.4%-29.6%, 20.1% and 19.4% were reported in different zones of construction (Clough and Leca, 1993). As would be expected with the observed over-mining at Anacostia River, the project was plagued by large settlements (~150mm) and the formation of a series of deep sink holes. Similar difficulties were present during the construction of the Oporto Metro in Portugal (Anon, 2002). Over the course of the construction of a 7.8m inner diameter tunnel, structural damage was inflicted on series of buildings and ultimately caused the collapse of one structure resulting in the death of one person.

A very clear example of the impact of reduced face pressures is found in the case of the St. Claire Rail Tunnel (Harrison et al, 1994). An 8.4m inner diameter tunnel was constructed through glacial till, passing under the St. Claire River at Sarnia, Ontario. Construction of the tunnel initially produced very little settlement while the tunnelling proceeded through slightly over consolidated till. At this stage of construction damage was noted to the seals on the main bearing. After halting construction to perform maintenance, during which time a recovery shaft had to be cut, construction resumed. To mitigate future damage to the seals during construction, the face pressures applied for the remainder of construction were roughly ¼ of those originally intended. As a result, settlements of over 1m were recorded.

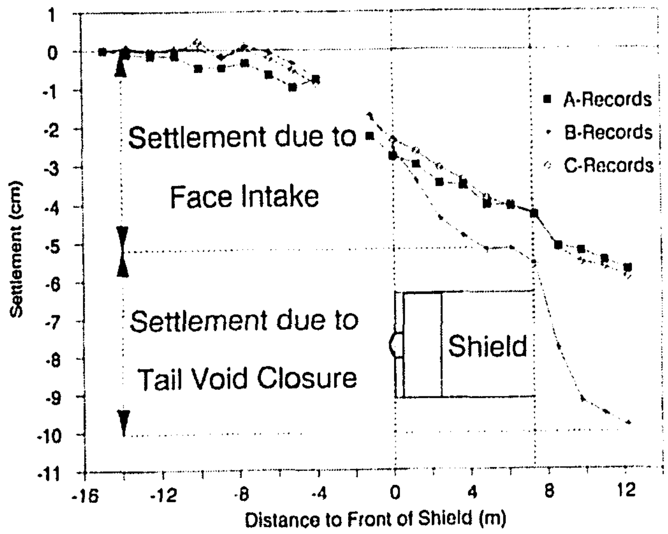


Figure 2-6 Settlement with proximity to EPB-TBM – Tunnel crown in sand, invert in clay (Clough and Leca, 1993)

2.1.2 Ground Movements along the TBM Shield

Ground movements that occur at points overlying the shielded portion of the TBM are beyond the operator's control. When TBMs are manufactured, the diameter of the cutting head is larger than the diameter of the shield. This over cut is necessary to reduce the drag on the TBM and allow the advance of the TBM. In most TBMs, the over cuts present are in the order of 20 to 30mm.

2.1.3 Ground Movements when transitioning between the Tail Shield and the Erected Segmental Lining

In Tunnel Boring Machines, there is a transition between the shield of the TBM and the segmental liner. As the segmented liner is installed inside the shield, it is impossible for it to be of the same diameter as the shield. As a result, a gap is always formed when transitioning from the shield to the installed segmental liner. To mitigate this source of ground movement, the void created by the transition is injected with grout.

At this time, there are two major methods for the injection of grout (Lovat). Spot grouting involves injecting grout through ports built directly into the segmental lining of the tunnel. This procedure is used when ground conditions are good and where the resultant settlements above the tunnel are not a cause for concern or control. This procedure can take place at any point along the tunnel alignment when required. Continuous grouting is the second method for minimising ground movements. Continuous grouting involves injecting grout through the tail seal, which is the plug between the inner diameter of the tail shield and the outer diameter of the segmented lining. By injecting grout into the void as it is being created, ground movements can be minimised.

Ensuring effective grouting procedures during tunnelling is critical in minimising the observed ground movements. In considering Figure 2-4 (p 13) it appears that in only two regions, 'a' and 'c', are settlements within the operator's ability to influence. Therefore, ensuring an effective grouting procedure is of considerable importance in attempting to minimise settlements.

2.1.4 Ground Movements Due to Deflection of the Tunnel Lining

Segmental linings serve two important functions in tunnel construction. They provide the actual structure of the tunnel; of equal consideration, is that they provide a thrust base for the advance of the tunnel boring machine. In both capacities, the segmental lining is subjected to considerable forces which cause deflection of the lining. In addition, the segmental lining is also subject to the loads imparted by the ground. Provided grout injection has been adequately performed (liner is surrounded by grout), the ground movements due to tunnel deflection will be very small (Kunito and Sugden, 2001).

2.1.5 Ground Movements Due to Long Term Settlements

Long-term settlements are typically associated with long term consolidation of the soil after construction of the tunnel. This consolidation occurs when a tunnel constructed below the water table acts as a drain, drawing down the water table. This reduction in water table increases the effective stress in the soil causing the ground to consolidate and the surface to settle. Modern tunnel linings often include water-stop systems that virtually eliminate tunnel seepage and thus effectively retard consolidation.

2.2 Introduction to Lovat EPB-TBM used for Construction of the Edmonton SLRT

For the Edmonton SLRT project, a refurbished Lovat RME257SE Series 17900 Equalised Pressure Balance Tunnel Boring machine was selected (Figure 2-7, p 20). The information presented in this section was obtained from the technical manual for the TBM. Manufactured in 1999 by Lovat, it had seen previous use in Singapore. Delivery of the TBM to the Edmonton site began in early June of 2003, and assembly-in-place began a short time later on the June 13, 2003. Fully assembled, the TBM shield was 9m in length with the ultimate length of the TBM and gantry being 67m (Table 2.1, p 21). Assembly of the TBM began with the assembly of the forward section of the shield. The forward shield provided the mounts for the cutting head of the TBM, the mountings for the hydraulic motors to power the TBM, and contained the excavation chamber. A solid bulkhead separated the forward and aft shields with access between the two limited to an airlock for personnel and the screw conveyor for spoil removal.

The cutting head diameter of the TBM was 6564mm, with the diameter of the segmental tunnel lining, 6300mm (Table 2-1, Figure 2-8, p 21). This difference in diameters (264mm) created a void of 4.1% which could only be filled through grout injection. Filling this void would require 2.66m^3/m of grout, or 3.2m^3 of grout for each ring of advance (1.2m).

The cutting head used during construction was of a mixed face design and had 8 flood doors (Figure 2-9, p 22). When the flood doors were fully opened, an opening over 35% of the area of the face was created. The flood doors also come with the option of having Grizzly Bars installed across the openings to retard the entry of large pieces of earth. To break up the soil mass, the face of the cutting-head was covered with a number of cutting tools: ripper teeth, a centre nose cutter, and scraper teeth.

The cutting head of the TBM was powered by a series of 8 hydraulic motors mounted on the forward shield (Figure 2-10, p 23). This series of hydraulic motors could generate a sustained cutting-head torque of 449 t*m, and a maximum torque of 501 t*m at start-up (Table 2.2, p 22).

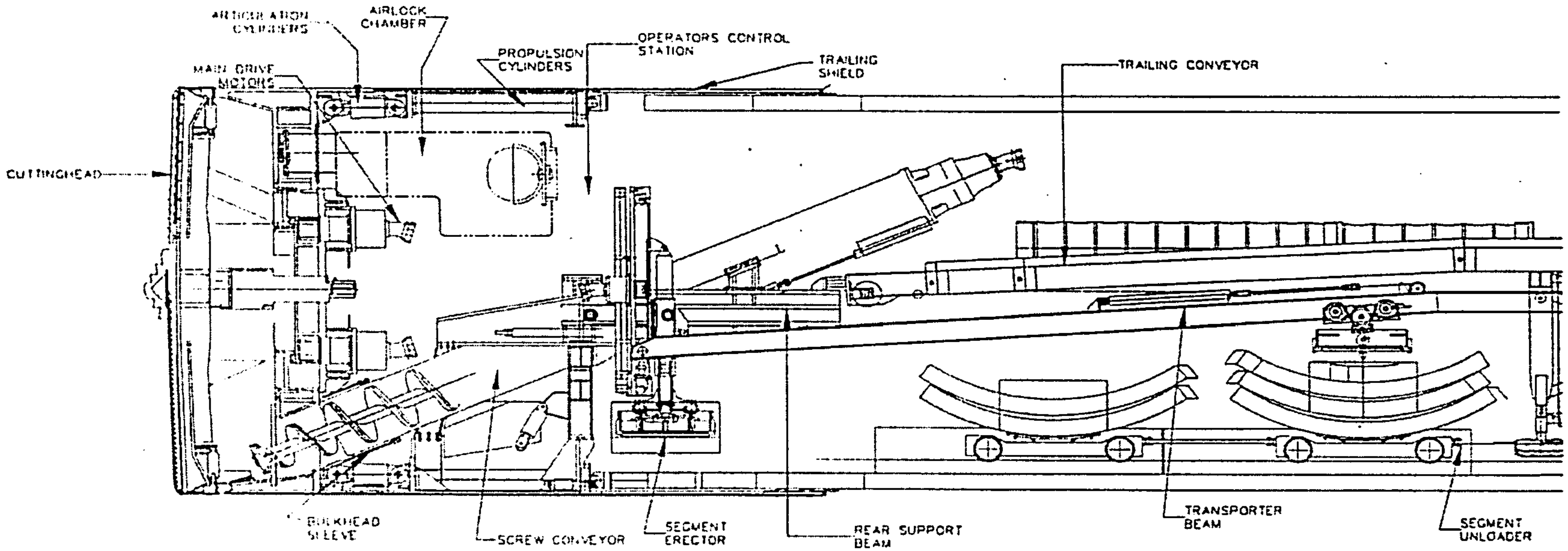


Figure 2-7 Limited cross section of EPB-TBM selected for use on the Edmonton SLRT (Lovat)

Table 2-1 Basic Tunnel Dimensions

Cut Diameter	6564 mm
Bore Diameter	6539 mm
Shield Diameter	6526 mm
TBM Length	9.0 m
TBM Length + Back-up	67.0 m
Ring Outer Diameter	6300 mm
Ring Inner Diameter	5800 mm

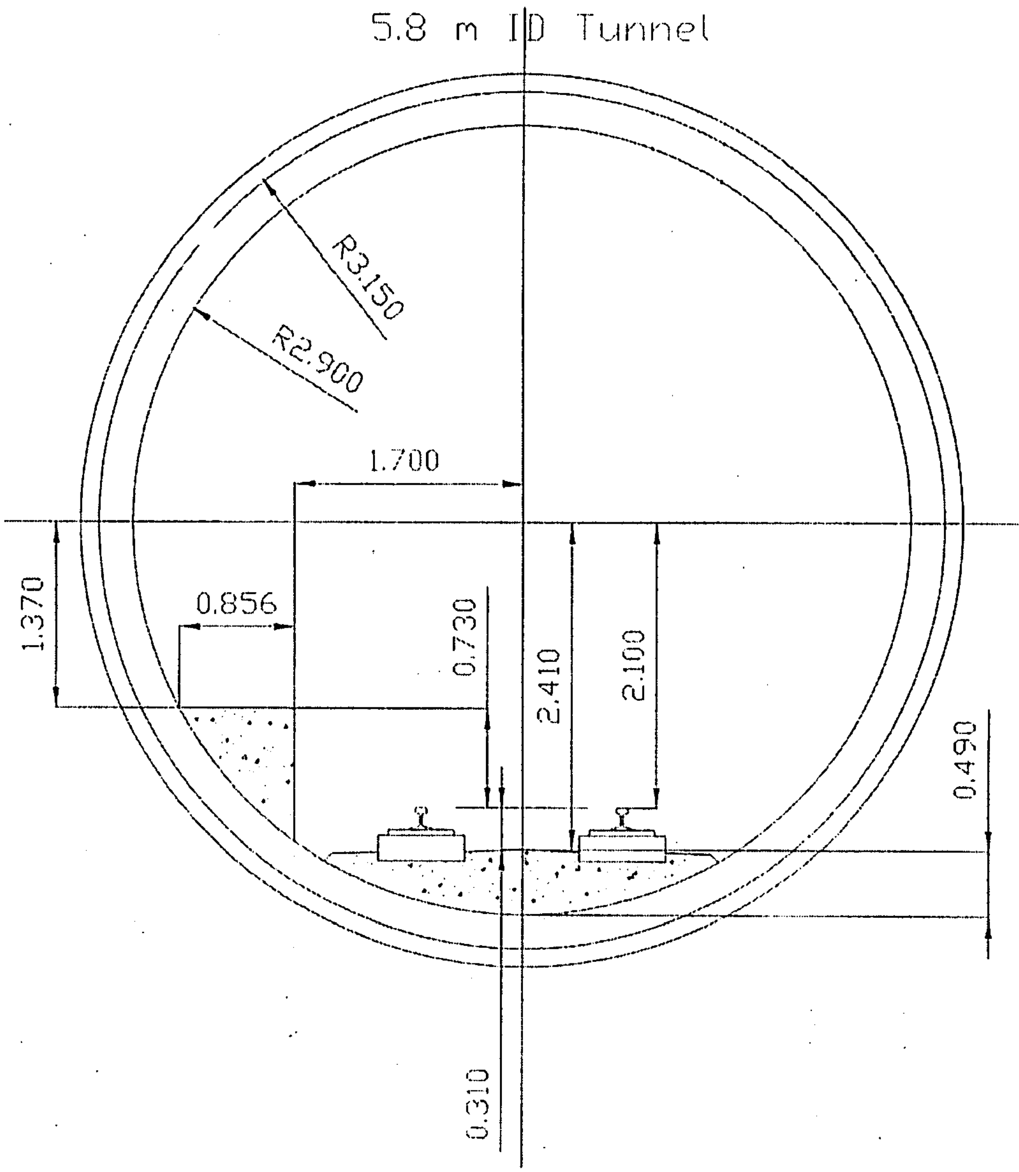


Figure 2-8 Dimensions of Finished Tunnel Cross Section

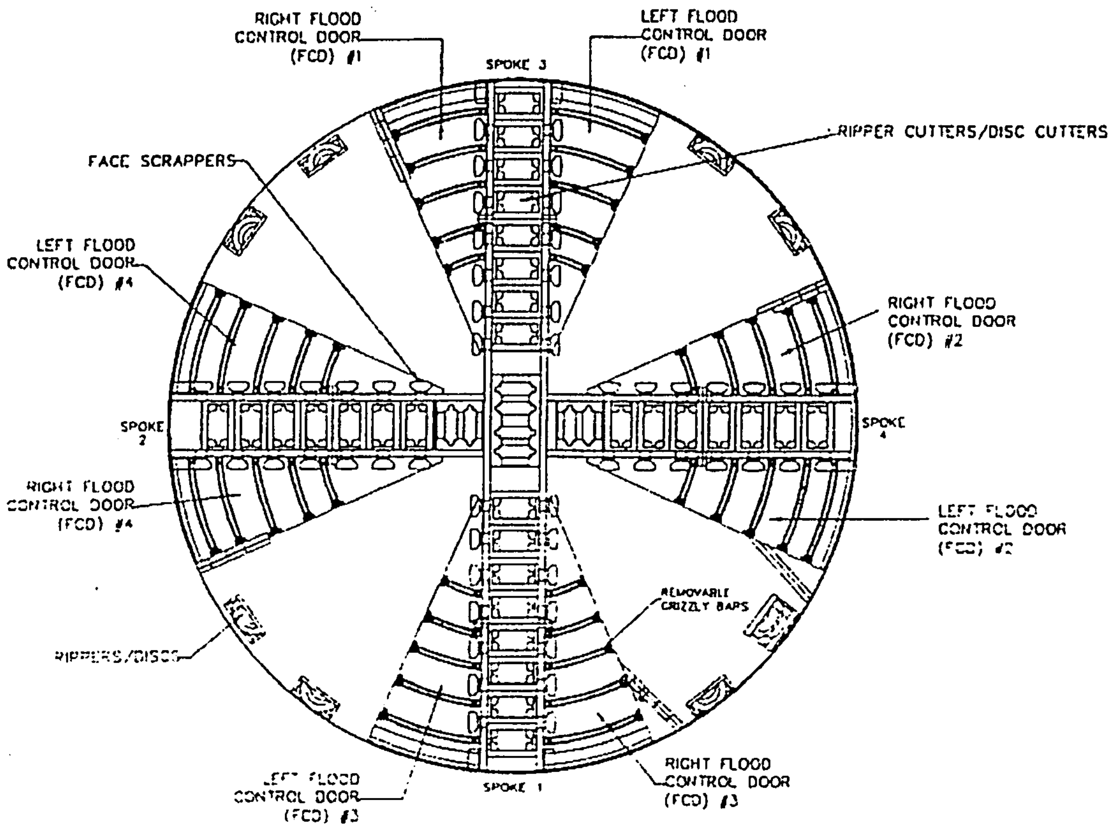


Figure 2-9 Cutting Head of EPB-TBM used for construction of Edmonton SLRT (Lovat)

Table 2-2 Cutting-Head Drive System

Maximum Torque	449 t*m @ 0.0 to 1.95 rpm
Minimum Torque	215 t*m @ 4.07 rpm
Maximum Starting Torque	501 t*m

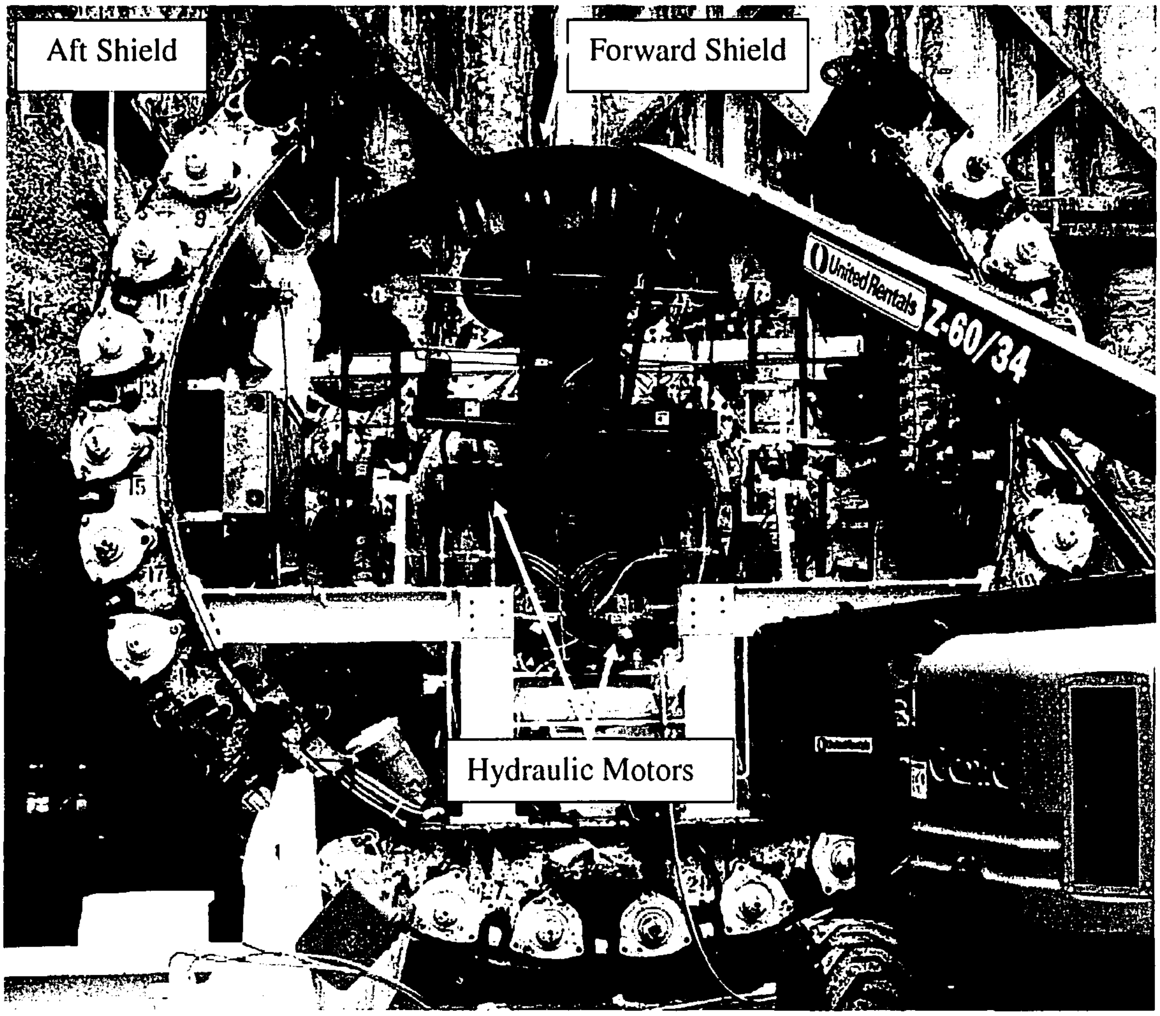


Figure 2-10 Fore and Aft Shields during assembly. June 23, 2003

(8 hydraulic motors visible at center of forwards shield)

Assembly of the aft shield, which provided mountings for the screw conveyor, conveyor belt, erector arm, and expansion cylinders, began shortly after the initial construction of the forward shield. One benefit of a design incorporating two shields is the possibility of incorporating articulation cylinders between the fore and aft shields. By articulating these hydraulic cylinders it is possible to warp the angle between the forward and aft shields. This warp allows greater manoeuvrability while mining, and reduces the turning radius of the TBM. In the Lovat TBM, used on the Edmonton SLRT project, articulations of up to 2.0 degrees in any direction were possible between the fore and aft shield. When fully assembled, the forward and aft shields would be joined by the articulation cylinders. Figure 2-11 (p 25), taken on June 25, 2003, shows the assembled but un-joined appearance of the fore and aft shields.

The propulsion of the TBM is accomplished through a series of 30 propulsion cylinders which are visible in Figure 2-11 on the perimeter of the aft shield. Each of the propulsion cylinders presses against a floating ring which then rests against the segmented lining. In this way, the thrusting force of the TBM is applied to the lining. Each cylinder was rated at 180 tonnes capacity giving an overall thrust for the TBM of 5400 tonnes with a maximum stroke of 2m (Table 2-3, p 25).

Also mounted on the aft shield are the screw conveyor and conveyor belt. The screw conveyor controls the removal of material from the excavation chamber to ensure an adequate face pressure is maintained. The screw conveyor within the excavation chamber has a caliper door which can seal off the chamber and allows the retraction of the screw. Similarly, guillotine doors at the discharge of the screw conveyor permit the discharge to be closed off. Table 2-4 (p 26) sets out the basic capacities of the screw conveyor used.

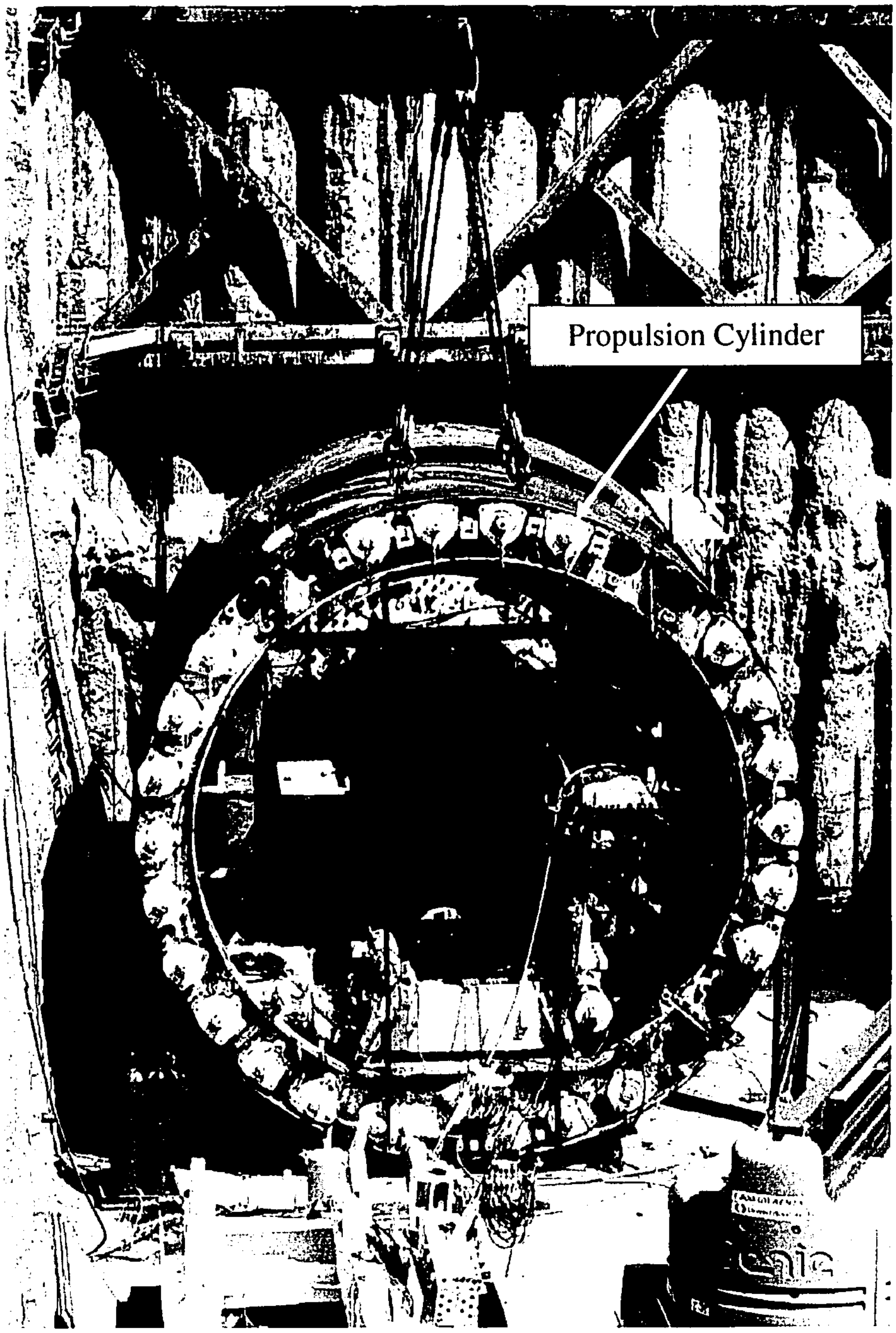


Figure 2-11 Fore and Aft Shields During Assembly, June 25, 2003

Table 2-3 Shield Propulsion System

Propulsion Cylinders	30 cylinders @ 180 tonnes
Maximum Thrust	5400 tonne @ 340 bar
Propulsion Stroke	2000 mm

Table 2-4 Screw Conveyor Capacity and Dimensions

Diameter	930 mm
Length	11.4 m
Capacity	450 m^3/hr

Also part of the spoil removal system is the conveyor belt. The conveyor is mounted within the aft shield, but is extended rearwards by the use of extensions. These allow the spoil to be carried aft to the muck cars. Table 2-5 (p 27) outlines the capacities of the conveyor system used.

After the assembly of the fore and aft shield, the final shield component to be installed is the tail can. The tail can extends the aft shield sufficiently to allow the assembly of the segmental lining within the shield. While it is part of the shield, the tail can was not added until late in the construction schedule in order to accommodate the construction of components within the TBM shield.

After the construction of the fore and aft shield, assembly focused on the placement and fitting out of the gantry and the installation of the tail can. The gantry when fully assembled is nearly 60m in length and carried much of the trailing gear needed to operate the TBM. This gear includes the foam plant needed for soil conditioning, the four electric motors that provide power for the cutting head, the power transformer, and the conveyor belt.

Figure 2-12 (p 27) shows a partially assembled gantry in use during construction. At this point the gantry is only half assembled due to space constraints. Only as the TBM advanced along the tunnel alignment could additional sections be added to bring the gantry up to its full length. Fitting out the TBM was completed in late July of 2003, and the first advance of the TBM took place at that time. Figure 2-13 (p 28) shows the TBM in the early stages of construction.

Table 2-5 Trailing Conveyor Dimensions, and Capacity

Width	1219 mm
Length	40 m
Speed	0 to 100m/min
Capacity	725 m^3/hr

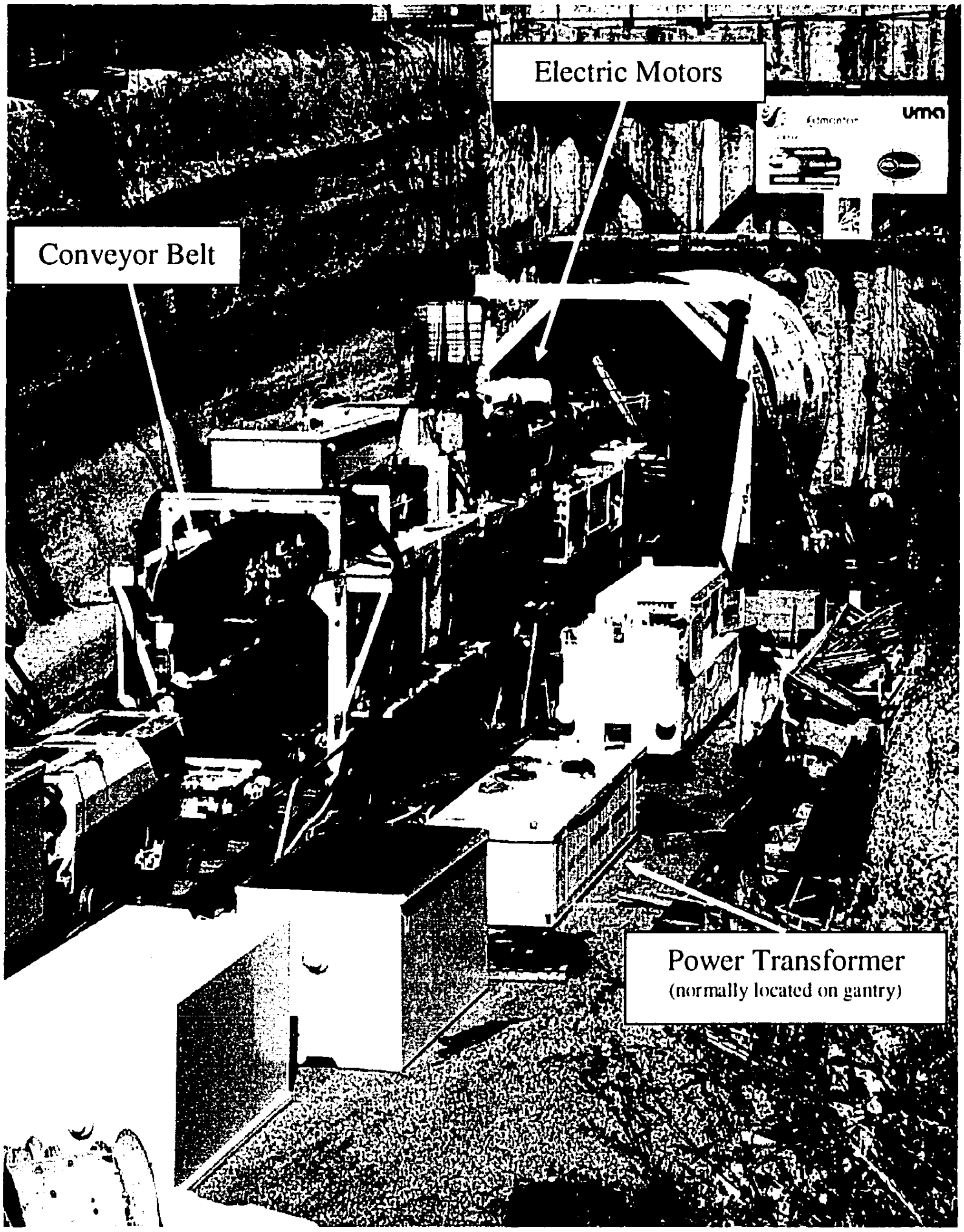


Figure 2-12 Partially Assembled Gantry, August 12, 2003

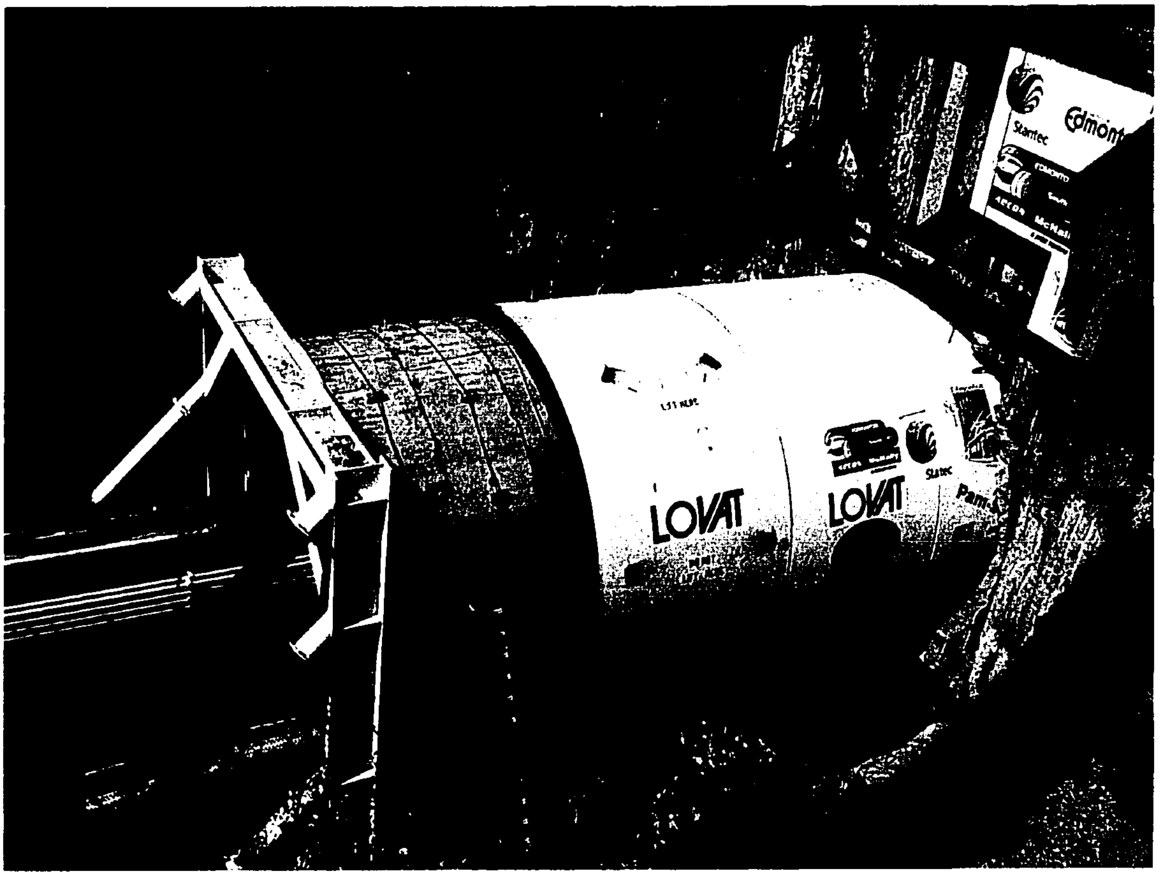


Figure 2-13 Start of Construction of Southbound SLRT Tunnel. August 12, 2003

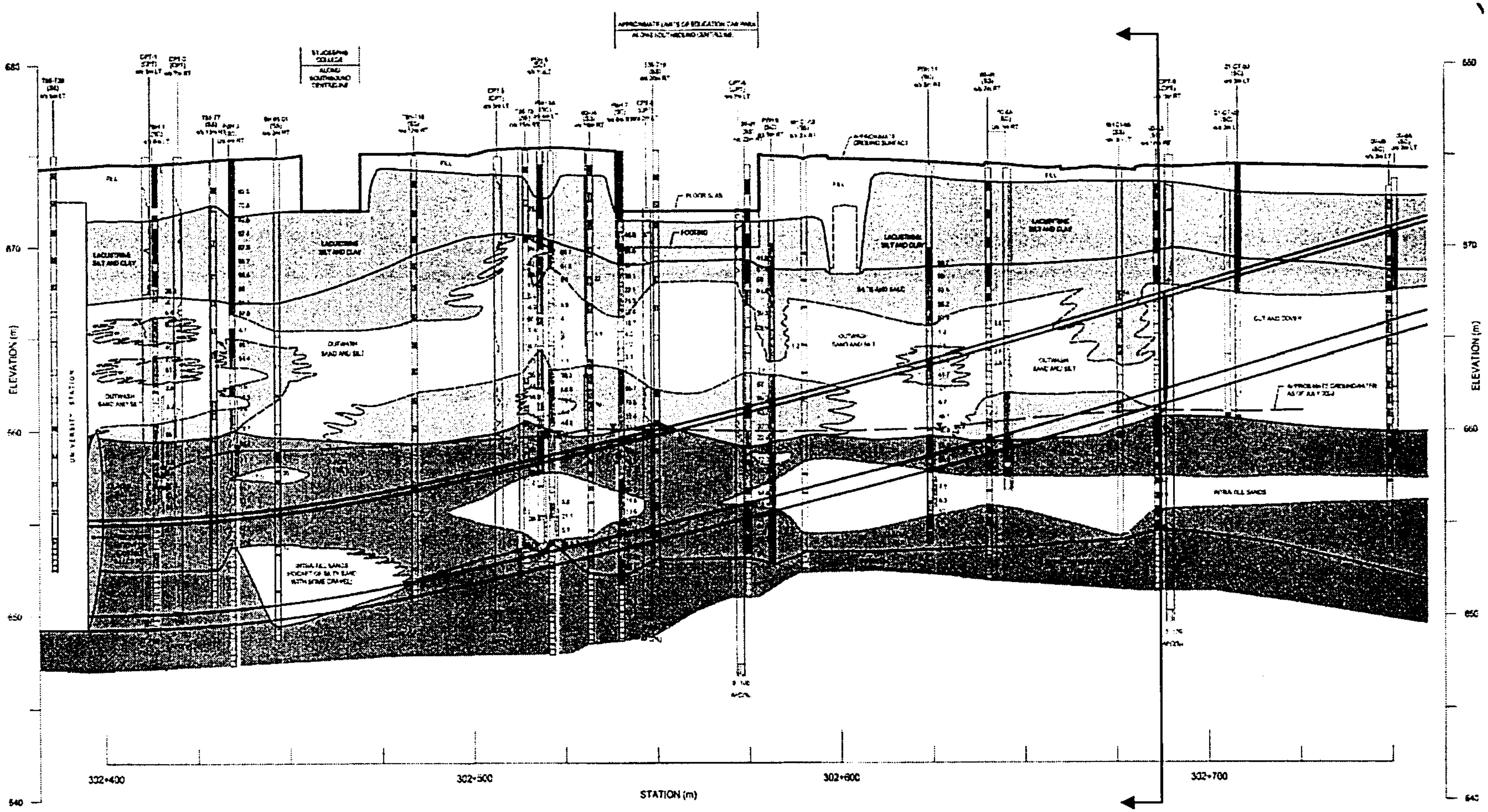
3.0 Site Characterisation of the Edmonton SLRT

3.1 Site Investigation

Over the past 20 years, a series of four major site investigations have been undertaken to characterise the geology of the current extension from the University Station to the Health Sciences Station. Initial construction of the Edmonton LRT system began in the middle of the 1970s with public service starting in 1978. From the LRT's earliest stage, the City's intention was that the LRT system would be expanded to the southern end of the city. The initial site investigation for the proposed SLRT extension began in 1985.

Thurber Engineering Consultants Ltd. was retained by the City of Edmonton, in 1985, for a preliminary geotechnical report for the proposed LRT alignment. Submitted in March of 1986, the report was based on a series of eight boreholes. In 1990, a second site investigation for the proposed SLRT alignment was commissioned and undertaken by Hardy BBT Limited, (now AMEC Earth and Environmental Ltd.). Submitted in June of 1991, this report made use of five boreholes along the alignment (AMEC, 2001). It should be noted that in the case of the first two site investigations carried out, that the LRT had not yet crossed the North Saskatchewan River, and that the University LRT Station was yet to be built. Completion of the University LRT Station took place in 1992.

In the year 2000, AMEC Earth and Environmental Ltd. was again commissioned to undertake a site investigation of the proposed LRT alignment. Submitted in October of 2000, this report made use of an additional eight boreholes along the alignment. The final site investigation for the proposed Edmonton SLRT extension was undertaken by Golder Associates. Making use of nine boreholes and ten cone penetration tests, this report was submitted in March of 2002. Figures 3-1 (p 30) and 3-2 (p31) provide an interpretation of the stratigraphy along the SLRT alignment, based on the information collected during the previous four site investigations.



Portal - Stn. +686.6m

Figure 3-1 Inferred stratigraphy for Southbound SLRT tunnel. (UMA).

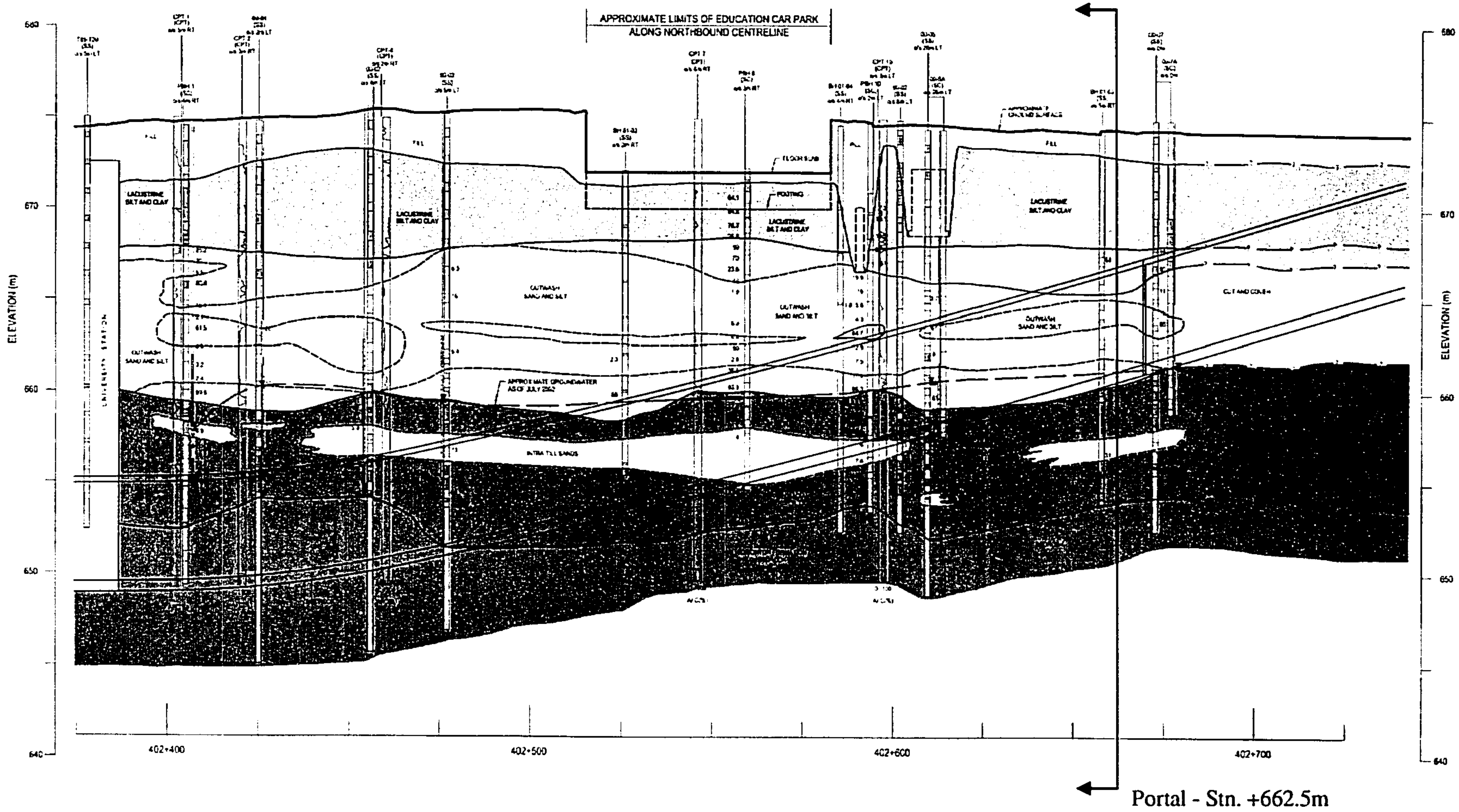


Figure 3-2 Inferred stratigraphy for Northbound SLRT tunnel. (UMA).

3.2 Lake Edmonton Clay

The uppermost of the glacial deposits, lacustrine Lake Edmonton Clay, can be found over the extents of the SLRT site as an overlying mantle. However over the extents of project area, the clays only reach elevations ~666m at the deepest extent. As a result, the only parts of the Edmonton SLRT project that involved construction in this material were the excavation of the portal, and of the removal shaft. Although there is the possibility of limited Lake Edmonton Clay at the crown of the tunnel, it was not expected to be present for more than the first few meters at the launch of the TBM. The presence of Lake Edmonton Clay is of note as both the Education Car Park and St. Joseph's College have their foundations in the clay.

3.2.1 Formation Process of Lake Edmonton Clay

Lake Edmonton Clay was deposited within the waters of Glacial Lake Edmonton during the retreat of the glaciers at the end of the Wisconsin glaciation some 12,000 year ago. The clays deposited are gradational with depth with observable varves. The material deposited at higher elevations is generally more plastic and more finely grained compared with deposits at lower elevations. The gradation of Lake Edmonton Clay is heavily influenced by the glacial retreat. Being deposited by glacial outflow the varves were formed by the seasonal variations in run-off. In addition, the proximity of any given location to glacial outflow sources changes the characteristic grain size: the material becoming finer with greater distance from the outflows. Figure 3-3 (p 33) shows a section of Lake Edmonton Clay during the initial excavation of the portal on March 28, 2003.

Figure 3-4 (p 34) illustrates the gradation present in the Lake Edmonton Clay, and the transition between the underlying Outwash Sands and Silts and Lake Edmonton Clay. The photo is divided into three zones. Zone 1, is clean sand typical of the Outwash Sands and Silts, and occupies only the lower area of the photo. The uppermost zone, Zone 3, is Lake Edmonton Clay and can be seen dipping to the east across the exposed face. Zone 2 shows the transition between the Outwash Sands and Silts and the Lake Edmonton Clay. At the base of Zone 2, a rapid transition from relatively clean sand to silty sand can be observed. The appearance and character of the Outwash Sands and Silts will be discussed in Section 3-3 Progressing in elevation up Zone 2, increasing silt is shown, and at the upper reaches of Zone 2, a swift transition to Lake Edmonton Clay can be observed.

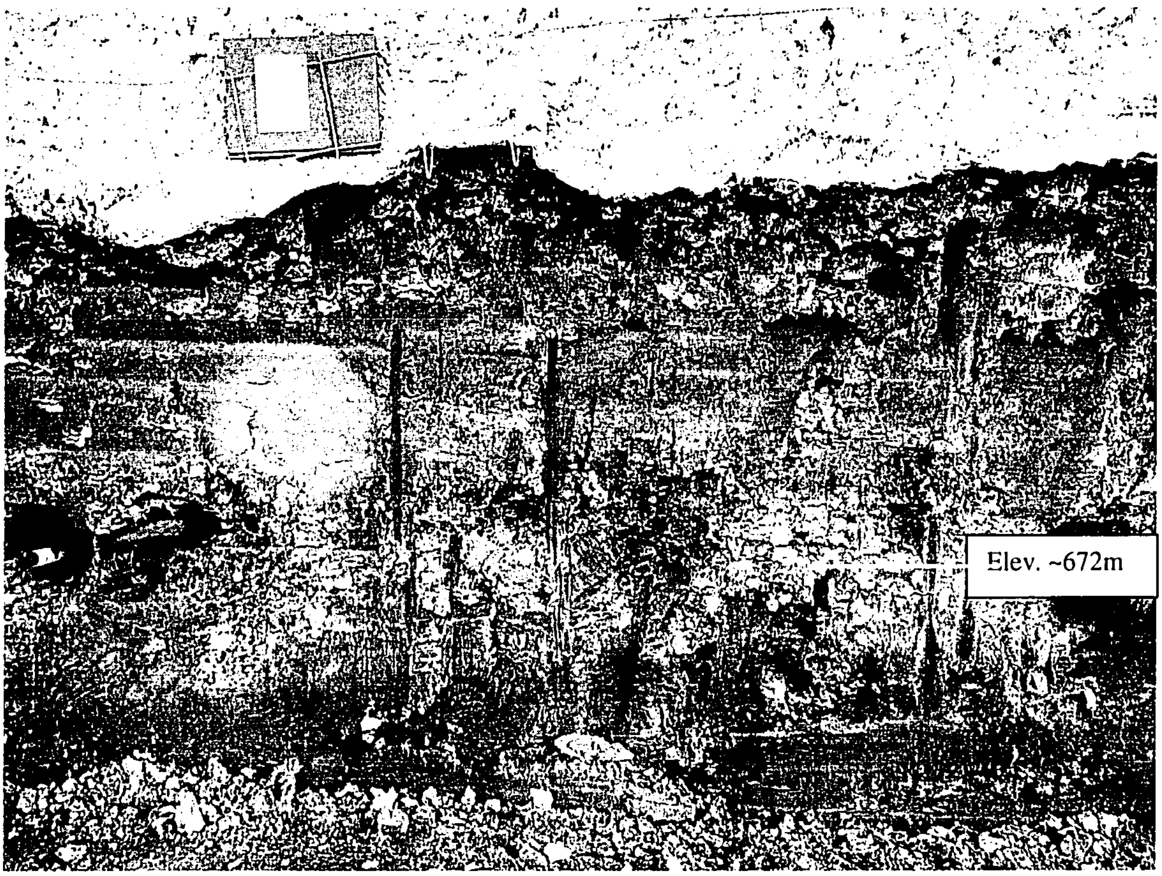


Figure 3-3 Exposed face of Lake Edmonton Clay during portal excavation, soil anchors present in left of photo at Elev. 672m.

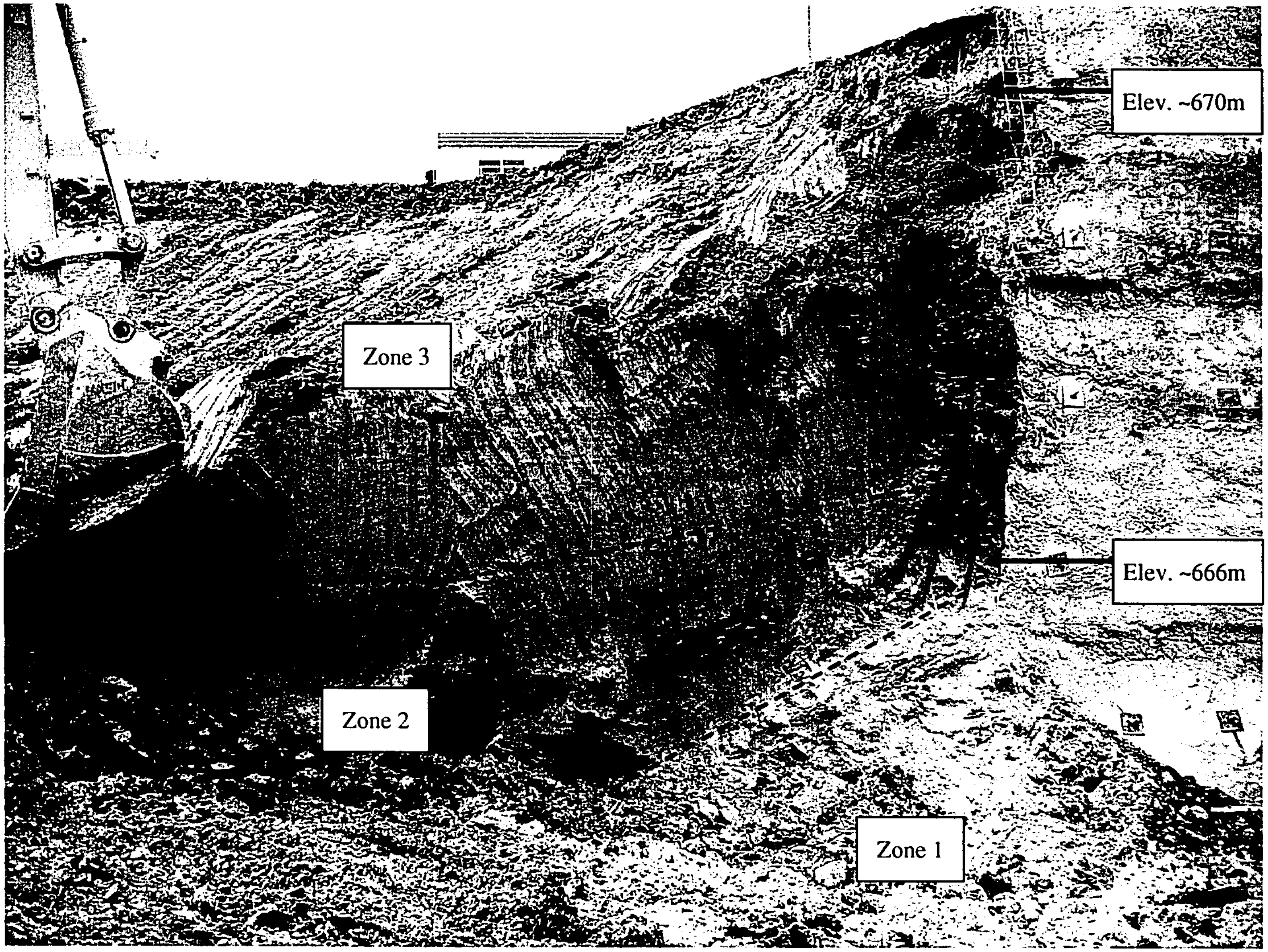


Figure 3-4 Transition zone between Outwash Sands and Silts and Lake Edmonton Clay.

3.2.2 Characterisation of Lake Edmonton Clay

Lake Edmonton Clay (LEC) was anticipated during the initial stages of construction, prior to the shield of the TBM being fully buried within the earth. As the TBM progressed down at a 6% grade during tunnelling, the removal of the Lake Edmonton Clay from the crown of the face occurred within a relatively short distance from the portal.

At this depth, in the tunnel zone, the character of the Lake Edmonton Clay was a medium to low plastic varved clay; the varves being formed of alternating layers of clayey silts and clay laminations. Within the zone of tunnelling, during the AMEC site investigation, SPT(N) values between 19 and 39 were obtained for the Lake Edmonton Clay in the region of tunnelling. In comparison, Lake Edmonton Clay at higher elevations produced SPT(N) values of between 9 and 17 per 300mm, and an undrained shear strength of some 70 to 80 KPa. The moisture content of the clayey silt in the launch region varied in the range of 20 to 22%, with unit weights between 1840 and 1870 kg/m^3 in both AMEC's and Golders' site investigations.

Over the course of the site investigation, two boreholes (00-06, 00-07), and one CPT test (CPT-9), were conducted in close proximity to the portal. The portion of the logs and tests relevant to Lake Edmonton Clay can be found in 'Appendix B – Lake Edmonton Clay' (p 183).

A sample collected at borehole 00-07 was subjected to an unconfined compressive strength test by AMEC in September of 2000. The sample was collected from a depth of between 5.3 to 5.7m and was described as silty, sandy clay that was very stiff, of medium plasticity with coal chips, rust stains, and sand lenses with stringers. The sample was at a saturation of 92.3%, with a moisture content of 25.8%, and void ratio of 0.755. The maximum compressive strength of the sample was 160 KPa at an axial strain of 4.4%.

3.3 Outwash Sands and Silts

The Outwash Sands and Silts (OSS) were found over the extent of the project area. At the launch of the TBM for Southbound and Northbound Tunnels the TBM was full face in this material. The presence of the Outwash Sands and Silts at the face of the machine was anticipated for the initial 140 meters, from Stn. +690m to +550m. Because of the steep grade of 6%, it was anticipated that the face would begin to transition to Glacial Till at Stn. +640m based on the inferred stratigraphy, and that by Stn. +550m, the TBM would be full face in the Glacial Till.

The Outwash Sands and Silts posed the greatest potential source of risk during construction due to the possibility of flowing ground conditions and settlements induced by over-mining. The upper boundary of the Outwash Sands and Silts followed along the base of the Lake Edmonton Clay at an elevation of ~668m. The base of the Outwash Sands and Silts rested on the underlying layer of Glacial Till. The elevation of this transition was far more erratic than the OSS-LEC transition. An interpretation of this boundary can be seen in Figures 3-1 (p 31) and 3-2 (p 32).

3.3.1 Formation Process of Outwash Sands and Silts

The Outwash Sands and Silts present over much of the Edmonton Region and the SLRT site were deposited near to outflows from the glacial front during the end of the Wisconsin glaciation. As the ice front was stalled for various lengths of time over the Edmonton region, melting in areas of the glacier away from the front generated large volumes of water and large pressure heads. The large movement of water within the glacier caused the excavation and movement of considerable masses of material. At the outflows along the glacial front, these movements led to the creation of alluvial fans, which in time accumulated to form the Outwash Sands and Silts.

Figure 3-5 (p 40) provides a general picture of conditions within the Outwash Sands and Silts present on the SLRT site. In the coarser fraction of the outwash material, the sand is what one would expect to find on a beach or in a sandbox with little to no fines. The photograph was taken on May 23, 2003, and shows the material present near the base of the excavation.

Figures 3-6 (p 41) and 3-7 (p 42) provide examples of the variability present in the Outwash Sands and Silts as a result of their formation process. In both Figures the lowermost soil anchor visible, or rebar, in the case of Figure 3-7, is at an elevation of 661m, and the visible lines of red spray paint are spaced at 1m intervals. Figure 3-7 was located along the West Wall of the excavation and faced south; it presented a face of relatively clean sand with some visible silt layering striking steeply across the face to the west with paleocurrents trending north-east. Over the remainder of the exposed face, few obvious silt inclusions are visible and the material was predominantly clean sand. Figures 3-8 (p 43) and 3-9 (p 44) provide a closer view of two areas of the sand face; Figure 3-8 provides a scale of the silt banding present, and Figure 3-9 provides a scale for a siltier zone of material within a mass of cleaner sand.

Figure 3-7 is located at the same elevation as Figure 3-6, along the East Wall of the excavation facing south. In contrast to Figure 3-6, Figure 3-7 presents a face with large quantities of silt present. At elevations below ~660.8m the appearance of the East face was of silt filaments within sand. Figure 3-10 (p 45) provides a more detailed view of the scale of this formation. Above an elevation ~660.8m, the material quickly transitions to sandy silt which then transitions several more times between sandy silt and sand with trace silt content and coal flakes. Another significant feature of the East Wall was the ripple formations present containing coal flakes. Throughout the excavation, the occurrence of coal flakes within the Outwash Sands and Silts was not particularly unusual. Although it was observed that the coal flakes appeared to largely be confined to areas of cleaner sand. Figure 3-11 (p 46) provides a more detailed view of this formation of coal flakes within the paleoripples.

While the East and West faces are not of particular note: that they are on the same elevation, separated horizontally by less than 10 meters brings attention to the variability present in the Outwash Sands and silts. Another illustration of this variability can be found in Figure 3-12 (p 47). During the construction of the portal, chemical grout was injected into the area adjacent to the portal wall. Prior to the construction of the TBM, a section of piling was removed to allow mapping of the grout penetration.

The exposed area was 3m vertical, and approximately 1.3m in width. The area exposed was located towards the West Wall of the portal, and some 6m behind the soil face that was shown in Figure 3-6 (p 41). The lines of red spray paint visible in the figure mark off 1m intervals. Visible in the exposed face, a number of clear soil transitions can be observed. Figure 3-13 (p 48) is a closer view of the exposed face at an elevation of 665.5m. In this photo, the rapid transition from relatively silty material to clean sand with large amounts of coal occurs over a very short vertical distance. Another example of this rapid transition can be found in Figure 3-14 (p 49) which shows the transition from relatively clean sand to sand with extensive silt stringers.

It is common practice to undertake a degree of 'beneficial self-deception' in viewing and describing geologic conditions in design: to consider one particular strata 'homogeneous' or to subdivide a large strata into several 'homogeneous' layers. In presenting Figures 3-5 to 3-14 it was the intention of the author to illustrate in a direct fashion the difficulty of attempting this sort of design simplification in the Outwash Sands and Silts. A good example of this variability was the comparison of West Wall Face and the East Wall Face. They are within 10 meters of each other, over the same range of elevations, but each presents a very different appearance and subsequently different material characteristics. These variations are a direct consequence of the formation process and its inherent variability.

Off-flow from the glacier over the course of the formation of the Outwash Sands and Silts was at no point uniform. Seasonal variation in off-flow, driven by temperature, could lead to the formation of varves over the alluvial fan. Equally, the occurrence of breakthrough flood events from the glacier could dramatically rework the material already deposited, eliminating any recently deposited material and leading to the formation of new channels. Influence from other off-flows could also have an impact on the deposition of material in any given location by contributing materials of different fineness and by causing adjacent alluvial fans to run over each other. In addition, the source of the material within the glacier could shift over the course of time changing the character of the material deposited. Channelling within the alluvial fan when above and below waters also contributed a powerful mechanism to rework the material within the already deposited alluvial fan. While it is easy to characterise the Outwash Sands and Silts in a gross sense, short of excavating all materials within the project area it is impossible to know the actual arrangement of the sands and silts directly through boreholes.

Figure 3-5 Edmonton SLRT portal near end of excavation, within the Outwash Sands and Silts – May 23, 2003.

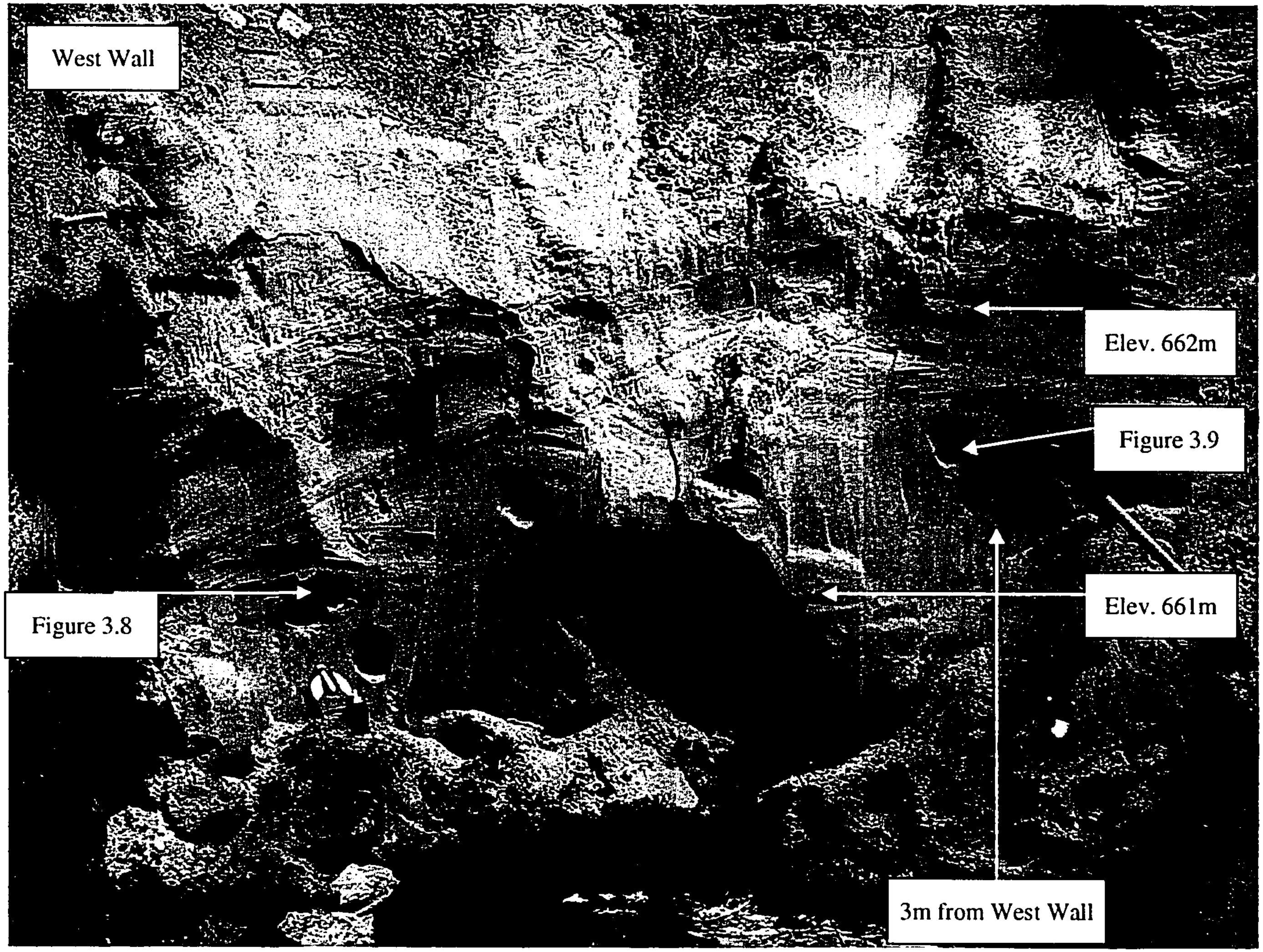


Figure 3-6 Mapping of exposed face of Outwash Sands and Silts during portal excavation, south facing along West Wall

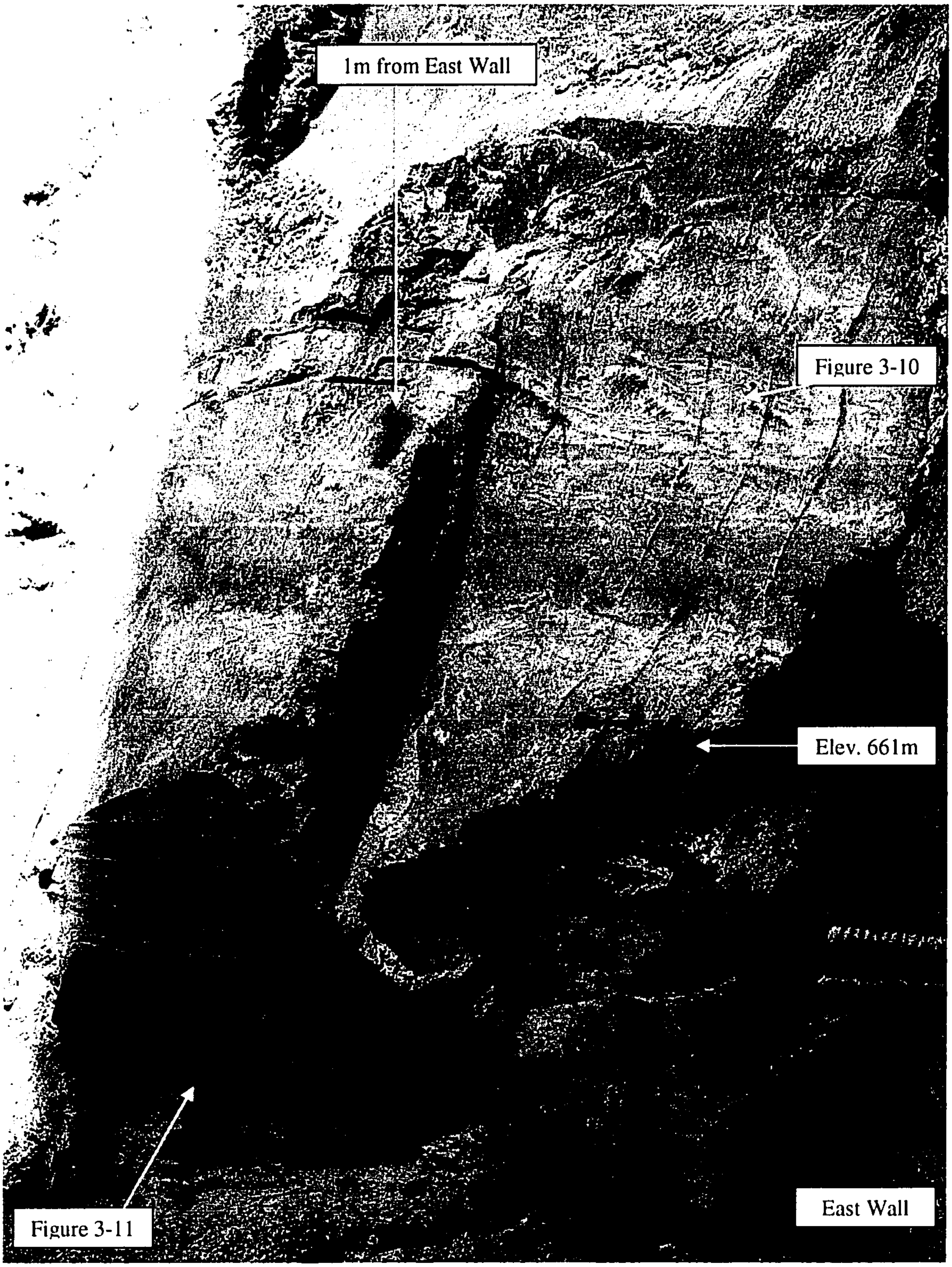


Figure 3-7 Mapping of exposed face of Outwash Sands and Silts during portal excavation, south facing along East Wall.

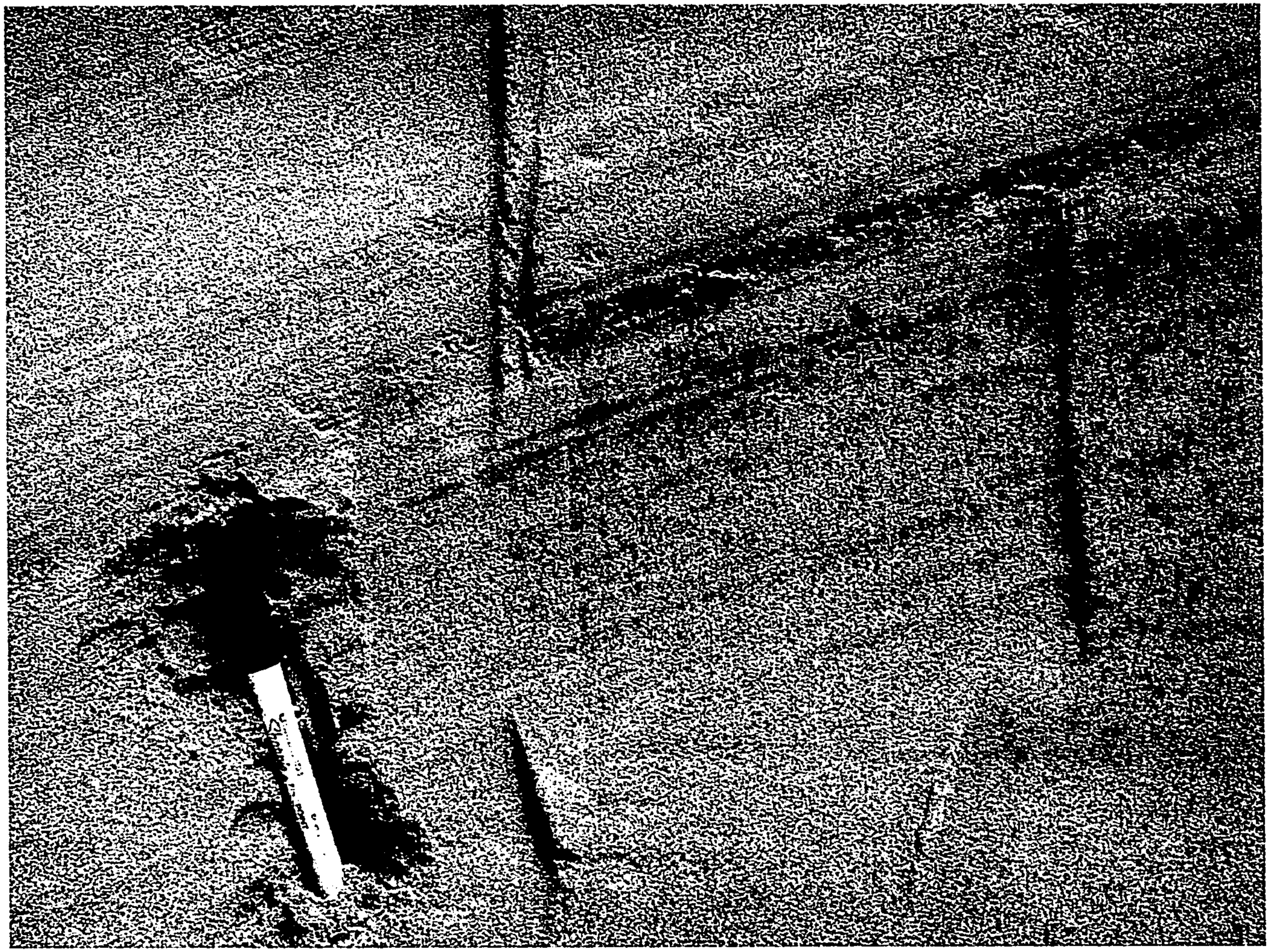


Figure 3-8 Enlargement of silt stringers within sand visible in Figure 3-6 (elevation ~661.5m, ~1m east of West Wall).

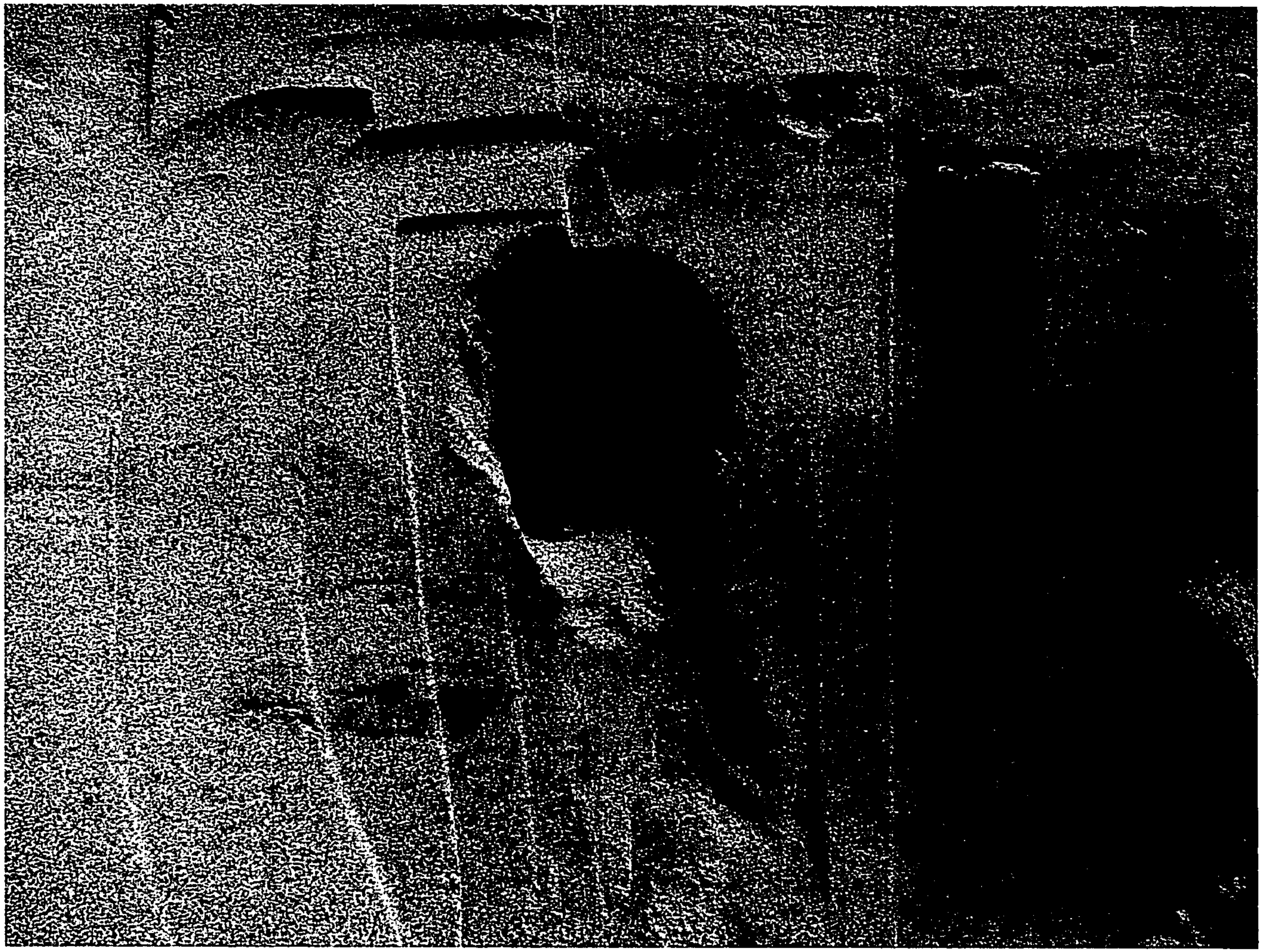


Figure 3-9 Enlargement of a siltier zone within the Outwash Sands and Silts from Figure 3-6 (elevation of ~661.5m, ~3m east of West Wall).

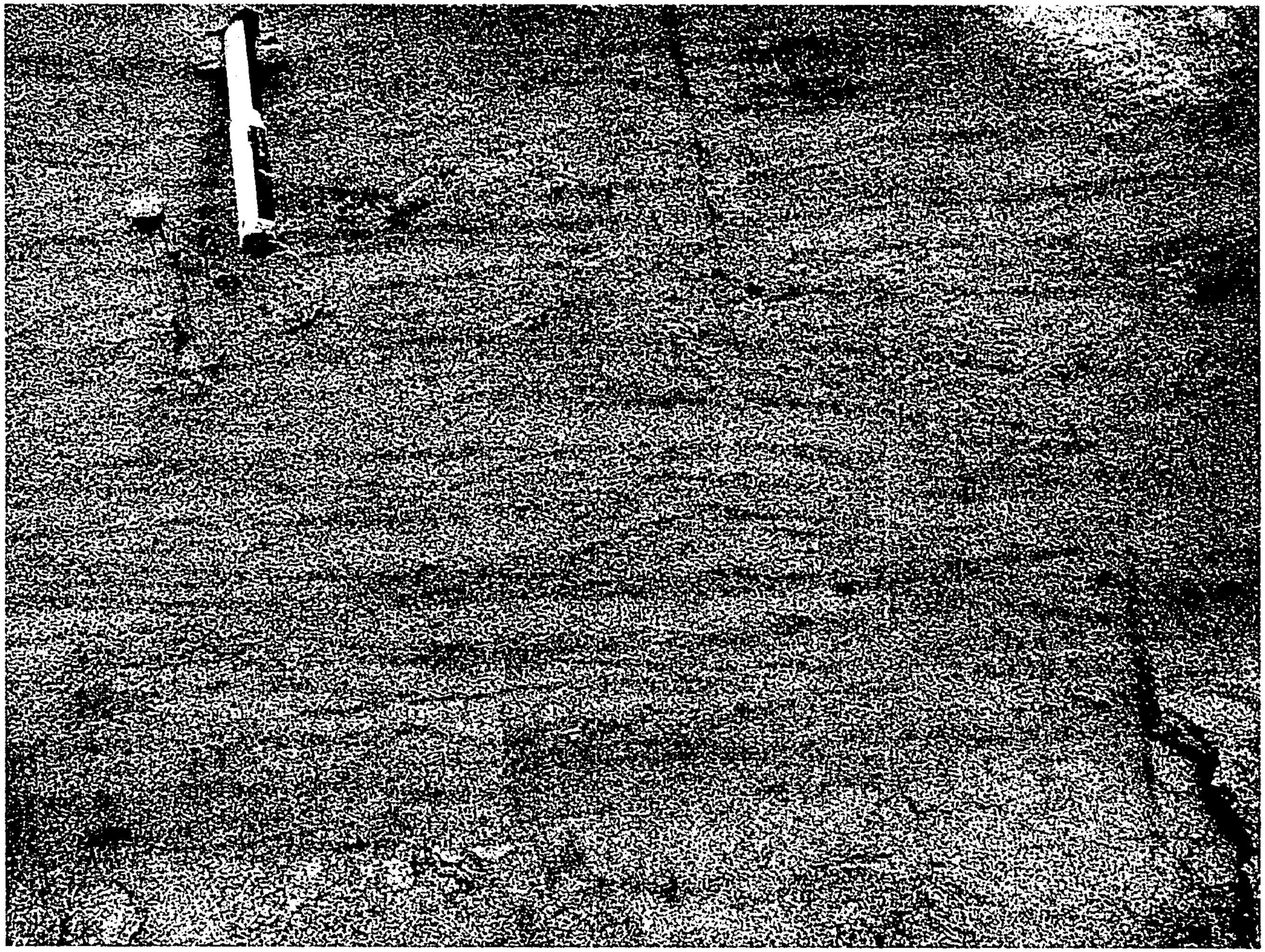


Figure 3-10 Enlargement of sand with silt inter-bedding visible in Figure 3-7 (elevation ~660.5, ~1.3m west of East Wall).

Figure 3-11 Enlargement of ripple formations with coal flakes visible in Figure 3-7 (elevation ~661.5, ~0.5m west of East Wall).

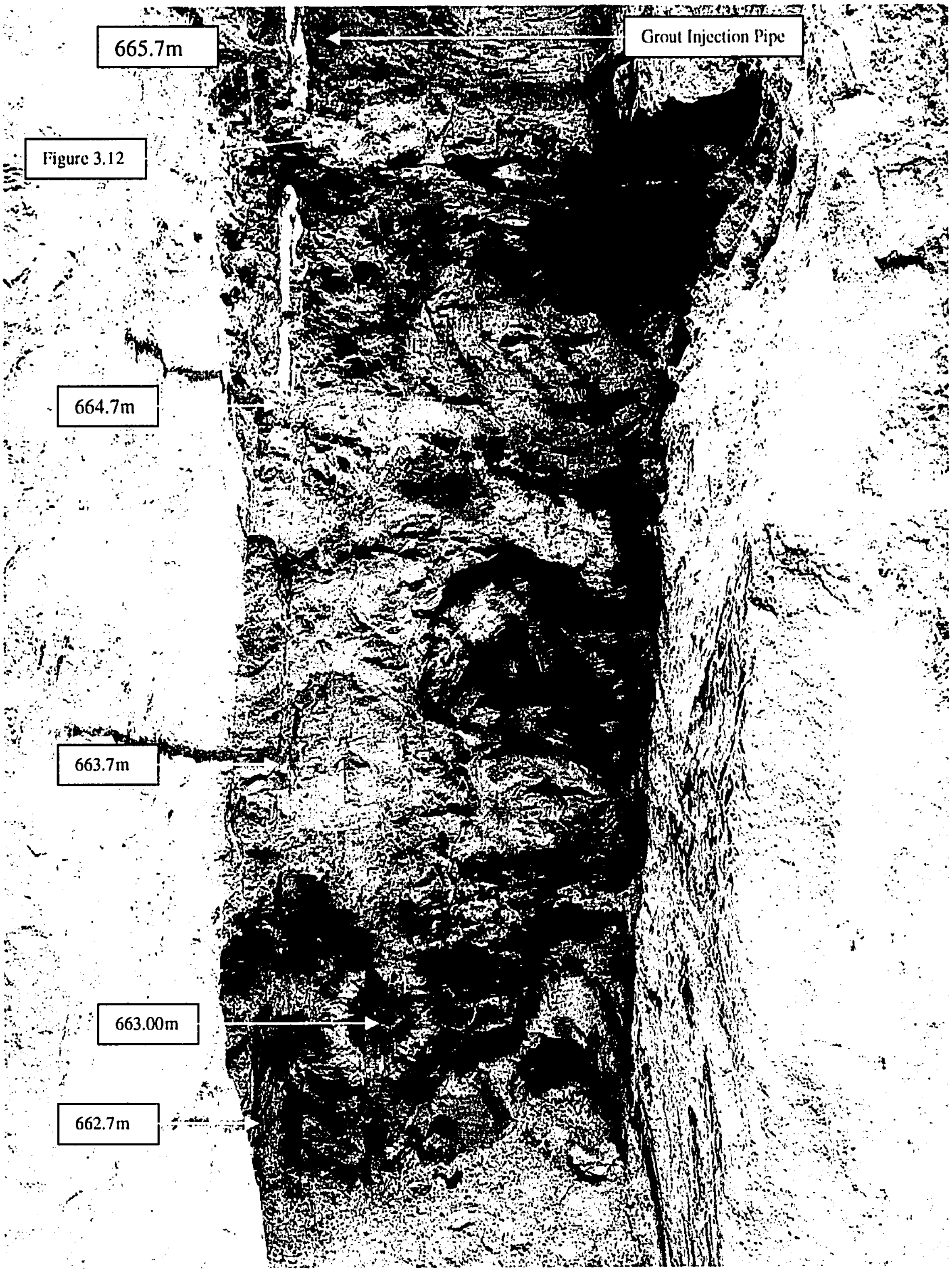


Figure 3-12 Exposed face of chemically grouted soil.

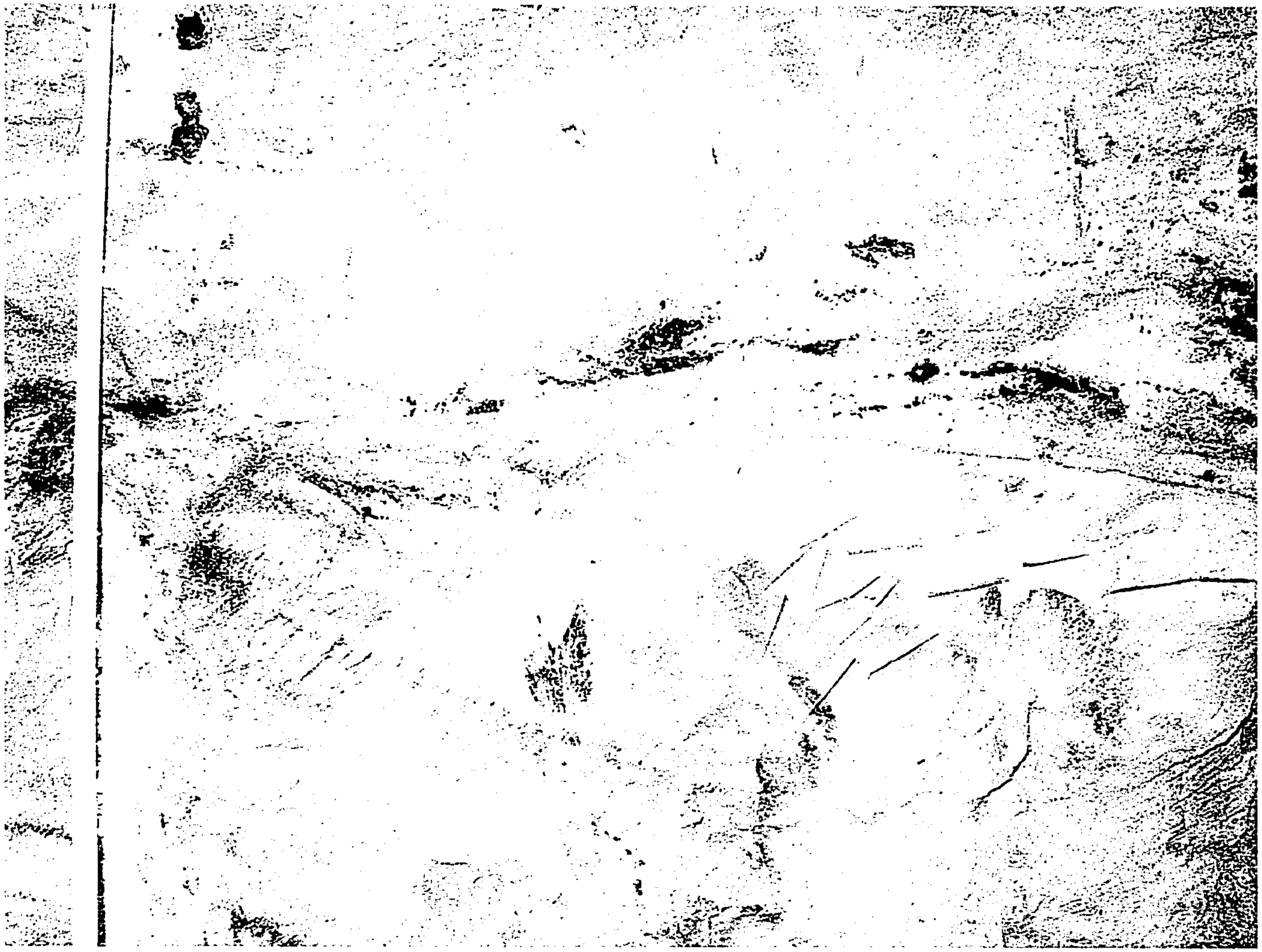


Figure 3-13 Enlarged view of sand-silt transition visible in Figure 3-12 (elev ~665.5m).

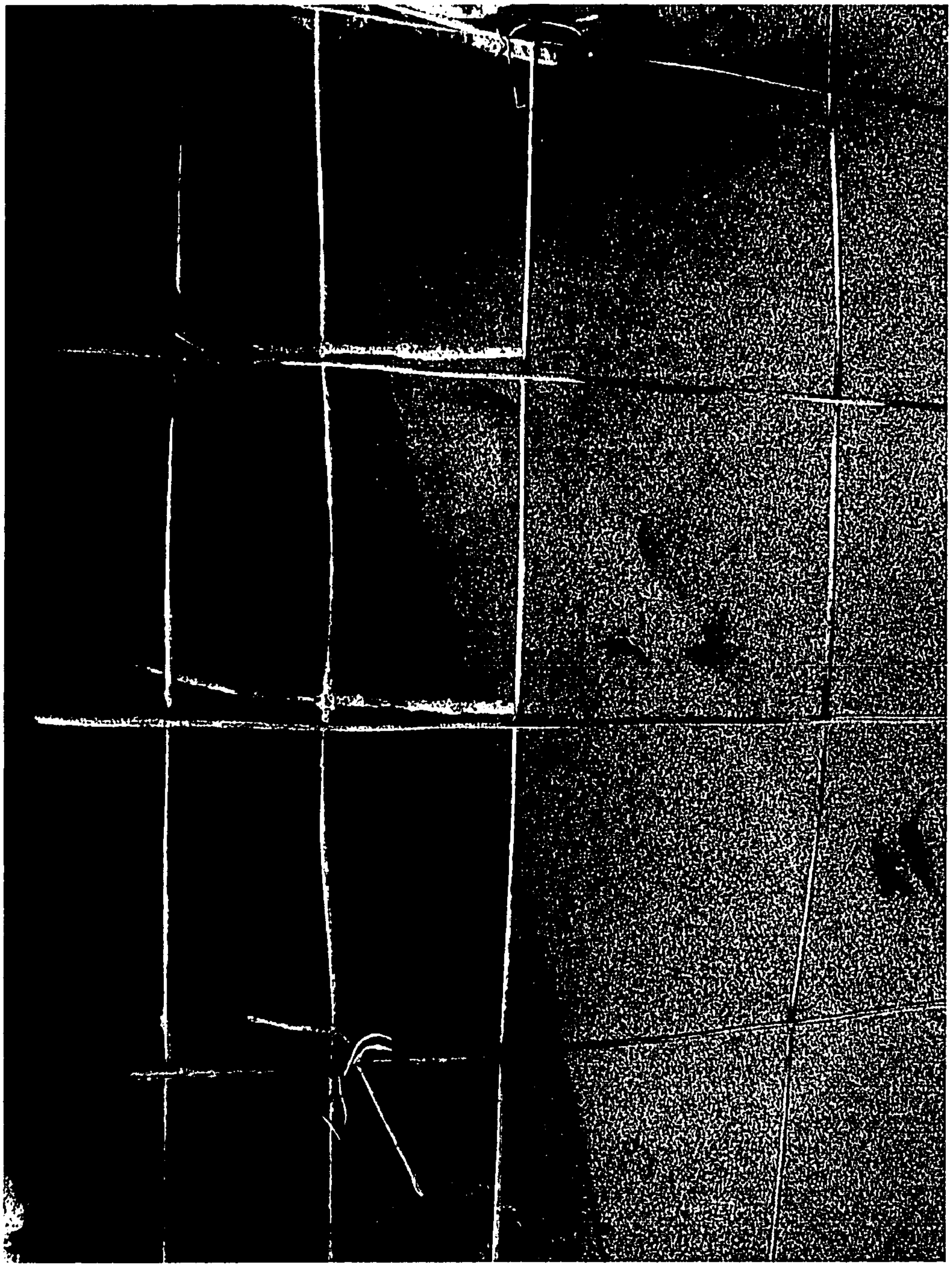


Figure 3-14 Transition from clean sand to sand with silt stringers (elev. 666m).

3.3.2 Characterisation of Outwash Sands and Silts

Contact with the Outwash Sands and Silts is continuous for the first 140m of the advance, from the portal at Stn. +688m until exiting from underneath the Education Car Park. In traversing this distance, the TBM passes through the entire thickness of the Outwash Sands and Silts. Having been hydraulically deposited, the Outwash Sands and Silts range from medium dense to dense, in terms of relative density. The typical range of values for the standard penetration test (SPT'N') values for the Outwash Sands and Silts is in the order of 15 to 45 blows per 300mm (Figure 3-15, p 53).

Much of the Outwash Sands and Silts consist of clean, fine sand with fines contents under 15%. However the range of fines content present within the Outwash Sands and Silts could range from 2% to 40%. This variation is a function of the variability of silt deposits and silt content within the material. In regions of very high silt concentration, fines contents well over 40% can be observed.

These siltier regions of the Outwash Sands and Silts can be grouped into two forms. The first form is that of interbedded sands and silts. Examples of these can be observed on a small scale in Figure 3-14 (p 49), and on a larger scale in Figure 3-7 (p 42). The second form is that of a massive body of silt located within the material. Examples of this form can be seen in Figures 3-12 (p 47) and 3-13 (p 48).

During construction of the portal for the Edmonton SLRT project, excavation through a significant depth of the Outwash Sands and Silts was necessary. As the excavation was progressing, a series of exposed faces were mapped and samples from the faces collected to be tested.

Figure 3-16 (p 53) shows the grain size distributions from samples collected at the faces shown in Figure 3-6 (p 41) and 3-7 (p 42), in addition to the grain size distribution of soil samples collected between elevation 666m and 664m along the east and west walls of the portal. The samples tested, typically had fine contents of less than 8%, the only exception being many of the samples collected from the exposed face shown in Figure 3-7 (p 42).

Figure 3-17 (p 54) shows the aggregate grain size distributions from material collected during the installation of the shallow and deep settlement points along the alignment. The samples tested were obtained from instrumentation being installed beneath the Education Car Park, and in the region between the Education Car Park and St. Joseph's College. The fines contents of the material ranged from 3% to 40% with the majority of samples reporting fine contents of less than 10%. When Figure 3-17 is compared to Figure 3-16 a well defined coarsening of the material can be observed in the fine sand region of the grain size distribution (0.42mm to 0.074mm). The clean sand samples in Figure 3-16 have a 50% passing diameter of ~0.2mm (range of 0.28 to 0.16mm), in Figure 3-17 this value increases to ~0.3mm (range of 0.31 to 0.2mm). In both regions the fines contents of the samples were comparable.

The grain size distributions obtained during AMEC's site investigation for the Outwash Sands and Outwash Silts can be seen in Figure 3-18 (p 54) and 3-19 (p 55) respectively.

The moisture content of the Outwash Sands and Silts varies in response to silt content and proximity to the water table. Moisture contents of the regions of clean sand within the Outwash Sands and Silts typically were in the order of 2 to 8%. In zones of high silt content the moisture content could be as high as 15%. Below the water table, the moisture content of the material varied between 16% and 25% with the variation being driven by the presence of fines in the material. Figure 3-20 (p 55), shows an overall histogram of the measured moisture contents of samples collected during Golders' and AMEC's site investigations.

While geostatistical methods provide an avenue to mathematically characterize this uncertainty in heterogeneity no geostatistical methods were applied to explore the Outwash Sands and Silts in this thesis. In the geotechnical reports issued by Golders and AMEC geostatistical methods were not applied to any significant degree. Those methods applied were restricted to the interpretation of the site investigation data as it applied to creating geologic profiles for the two tunnel alignments. This typically took the form of attempting to define regions of 'sand', 'silt', and 'sandy silt' within the Outwash Sands and Silts. UMA's inferred geologic profile for the tunnel alignment is presented in Figure 3-1 and 3-2.

During the site investigation process, numerous boreholes and CPT tests were conducted along the alignment. Appendix C – Outwash Sands and Silts (p 187), contains a selection of borehole logs and cone penetration tests (CPT) relevant to the Outwash Sands and Silts from the region between the portal and the Education Car Park.

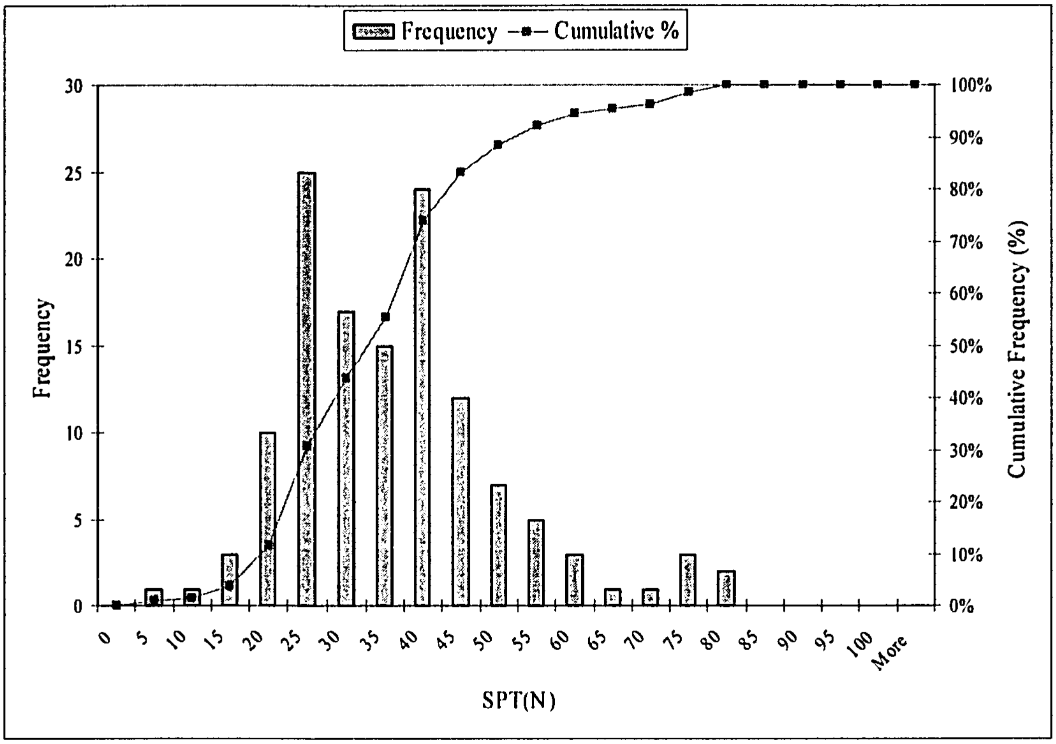


Figure 3-15 Histogram of SPT(N) blows per 300mm in Outwash Sands and Silts.

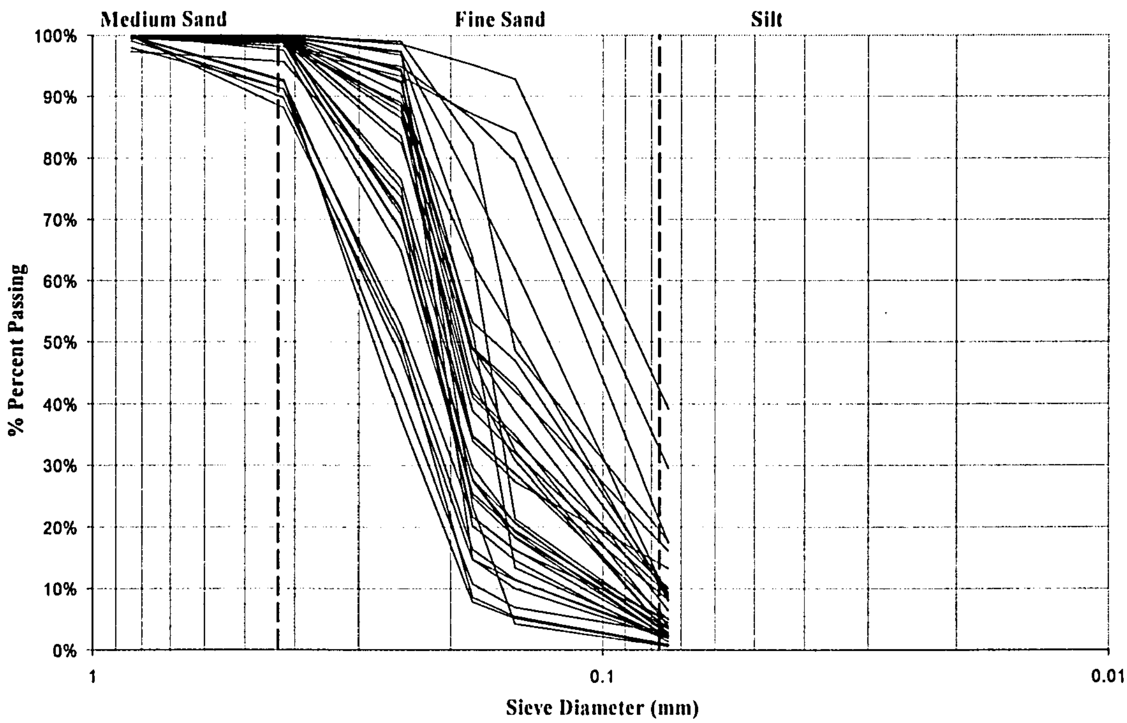


Figure 3-16 Grain size distributions of Outwash Sands and Silts collected during mapping of open cut excavation.

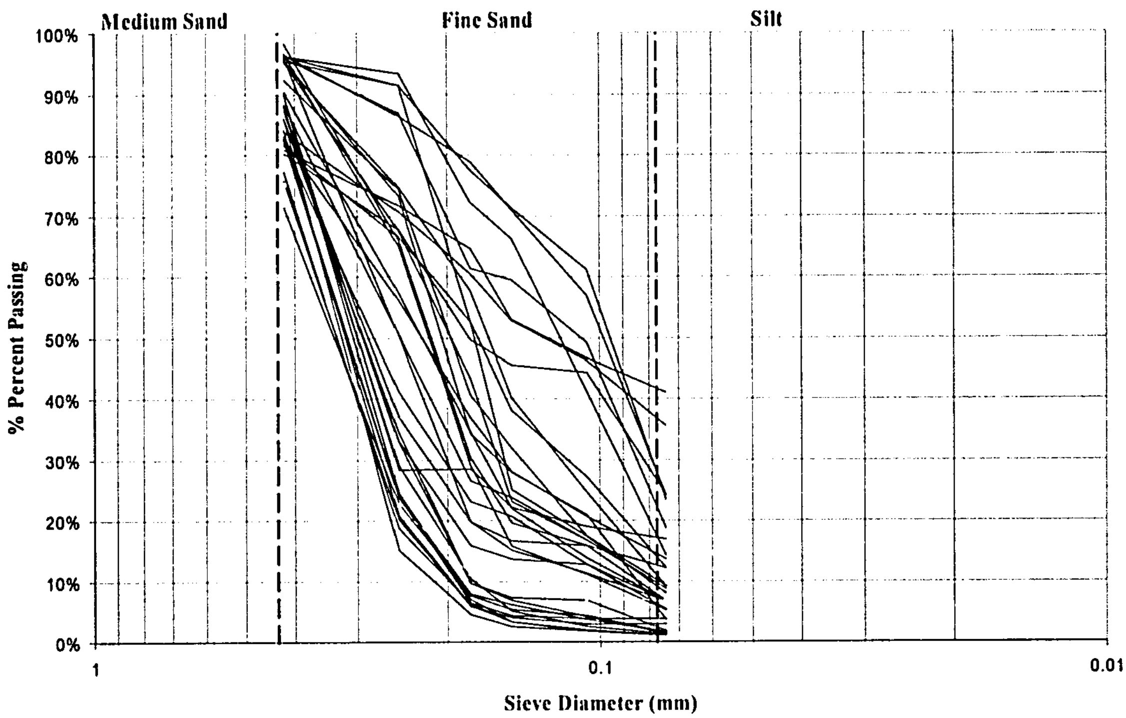


Figure 3-17 Grain size distributions of Outwash Sands and Silts collected from hollow and solid stem augurs during installation of settlement points.

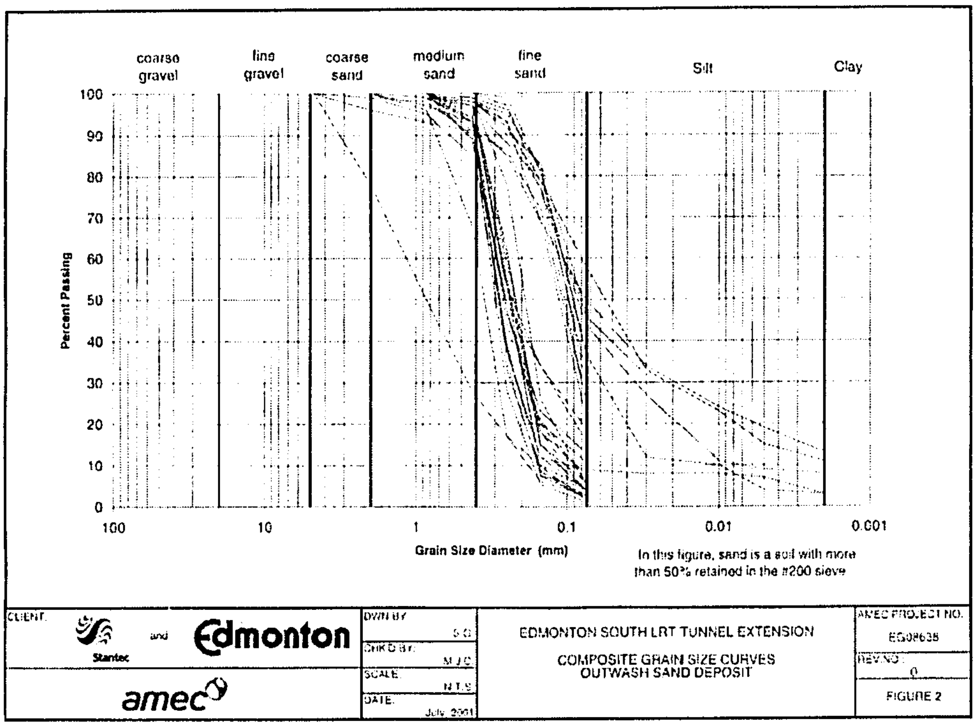


Figure 3-18 Composite grain size distributions for Outwash Sands (AMEC).

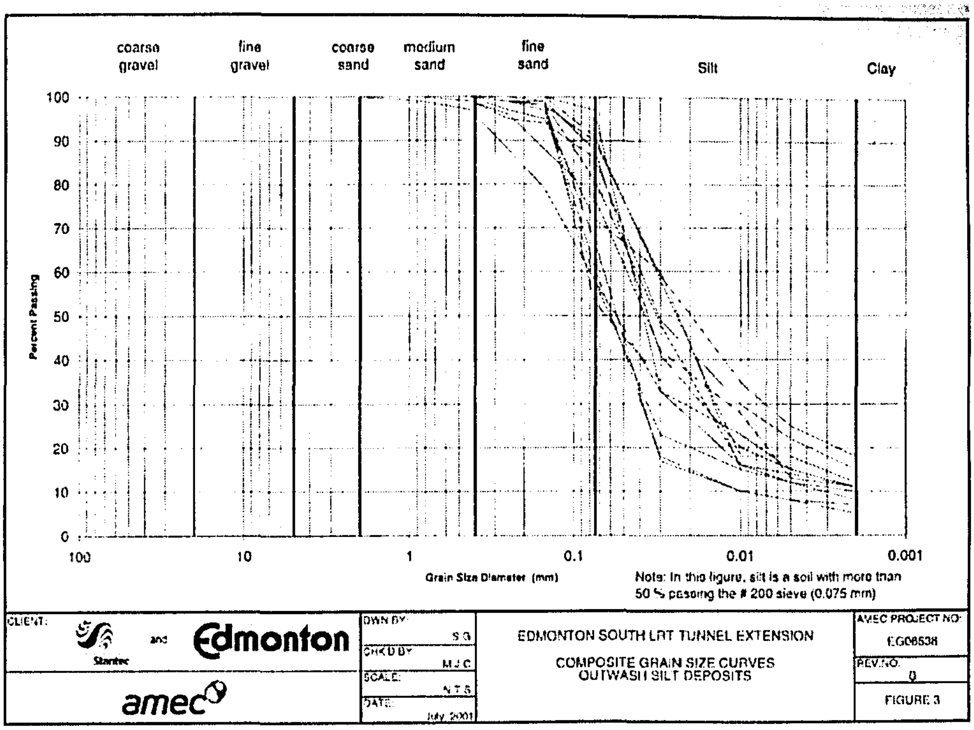


Figure 3-19 Composite grain size distributions for Outwash Silts (AMEC)

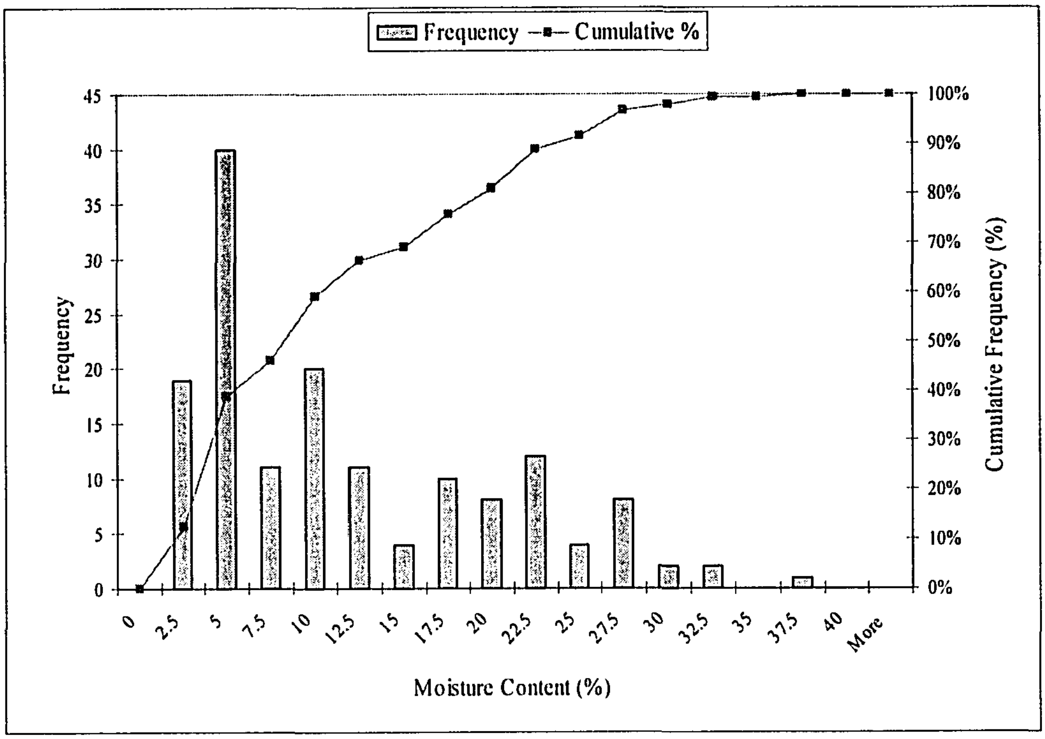


Figure 3-20 Histogram of observed Moisture Contents of the Outwash Sands and Silts.

3.4 Glacial Till

Glacial Till is present throughout the extent of the SLRT project. The Glacial Till underlies the Outwash Sands and Silts present in the project area. Contact with the Glacial Till during construction was anticipated in the region of Stn. +640m, approximately 50 meters from the start of the tunnel. Over the next 100 meters of tunnelling, the Glacial Till would transition from the invert to occupy nearly the entire face by Stn. +640m, based on the stratigraphic interpretation (Figures 3-1, p 30, and 3-2, p 31). Bedrock would begin to transition into the invert in the region of Stn. +525m during construction of the Southbound Tunnel, and in the region of Stn. +475m during Northbound Tunnel construction. Construction after encountering Bedrock occurred in mixed face conditions for the remainder of construction, with Glacial Till and Bedrock occupying between 30% and 70% of the face respectively. As a competent material, the transition from Outwash Sands and Silts to Glacial Till served to minimise the settlement risks imposed by the Outwash Sands and Silts and its susceptibility to over-mining.

3.4.1 Formation Process of Glacial Till

The Glacial Till present on the Edmonton SLRT site was deposited during, and at the end of the last local period of glaciation. Unlike the off-ice post glacial formations of Lake Edmonton Clay and Outwash Sands and Silts, no sorting was involved in the placement of the material. As a result of this deposition, the Glacial Till is a heterogeneous mixture of sands, silts, clays, cobbles, and boulders with no bedding or apparent structure.

An additional source of variability within the Glacial Till is the appearance of intra-till sands. Deposited during the formation of the Glacial Till and by flow within the till, the intra-till sands form a chaotic arrangement of pockets and channels within the till. The intra-till sands are typically water-charged and can show varying degrees of inter-connectedness. An example of the high degree of inter-connectedness came during a series of pump tests in the inter-till sands which showed rapid responses to the pumping, observed over 100m away.

Figure 3-21 (p 58) illustrates a pocket of intra-till sand. Of note is the large quantity of cobbles present within the pocket. Figure 3-22 (p 59) shows two bands of intra-till sand cut in two by a layer of Glacial Till. Limited boulder pavement can be noted at transitions between the Glacial Till and the intra-bedded sands present. The thickness of the most upper intra-bedded sand layer in this Figure is in the order 0.5m. Figure 3-23 (p 60) shows a layer of intra-till sand striking through the tangent pile wall with additional, but less oxidised intra-till sands above it.

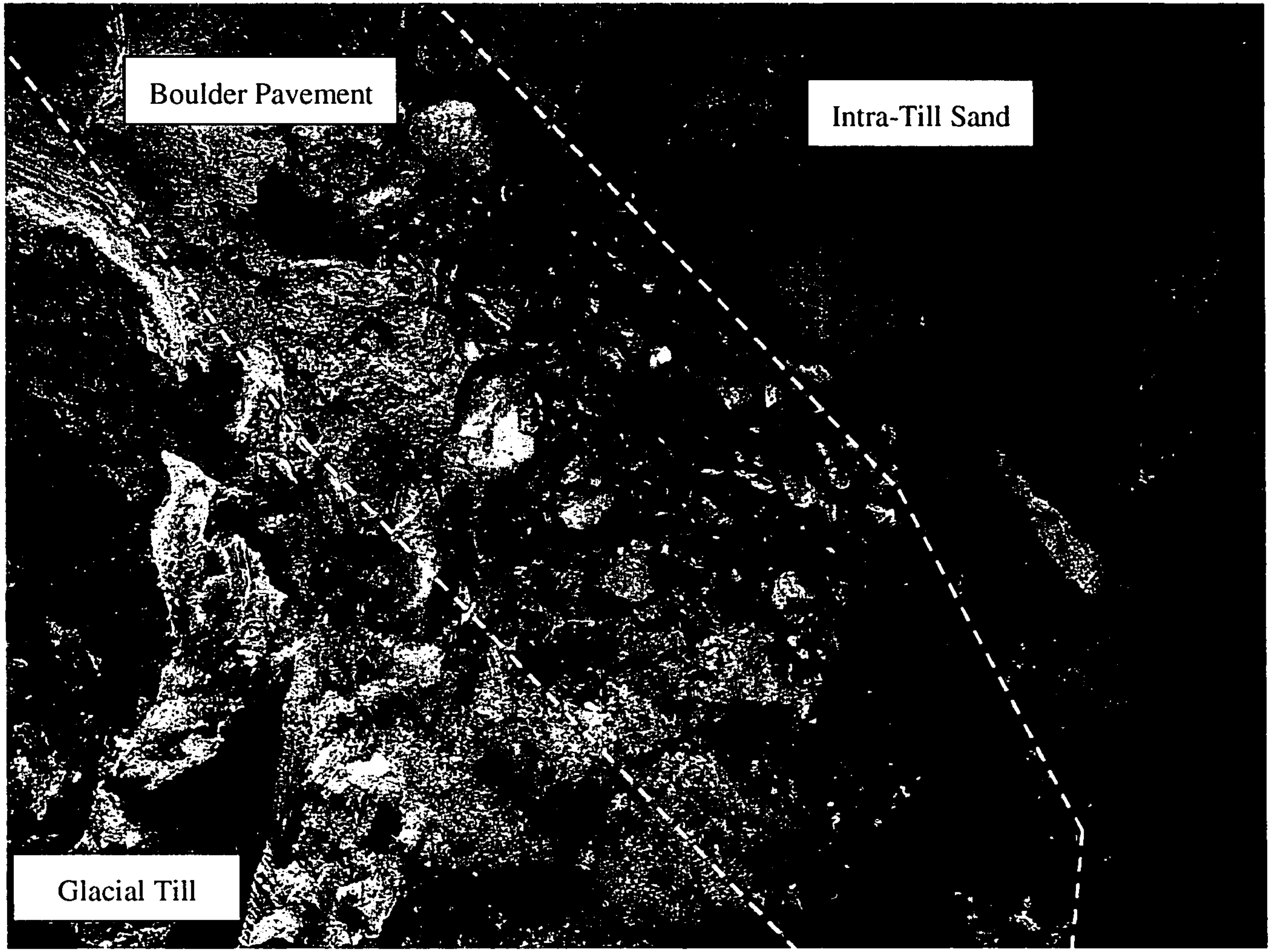


Figure 3-21 Boulder pavement at boundary of Glacial Till and Intra-Till Sands (elev. ~655m).

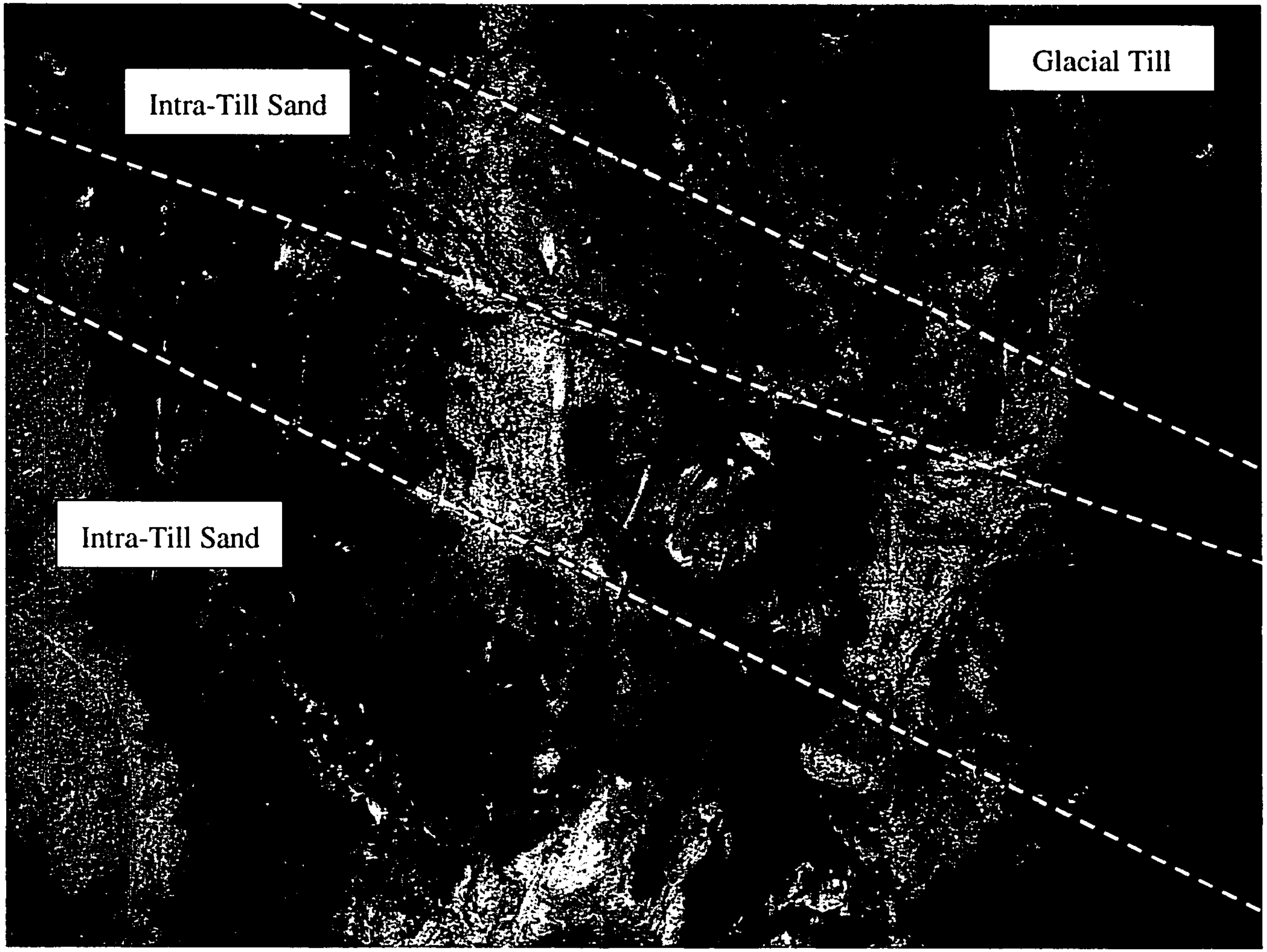


Figure 3-22 Intra-Till Sands with limited boulder pavement (elev. ~655).

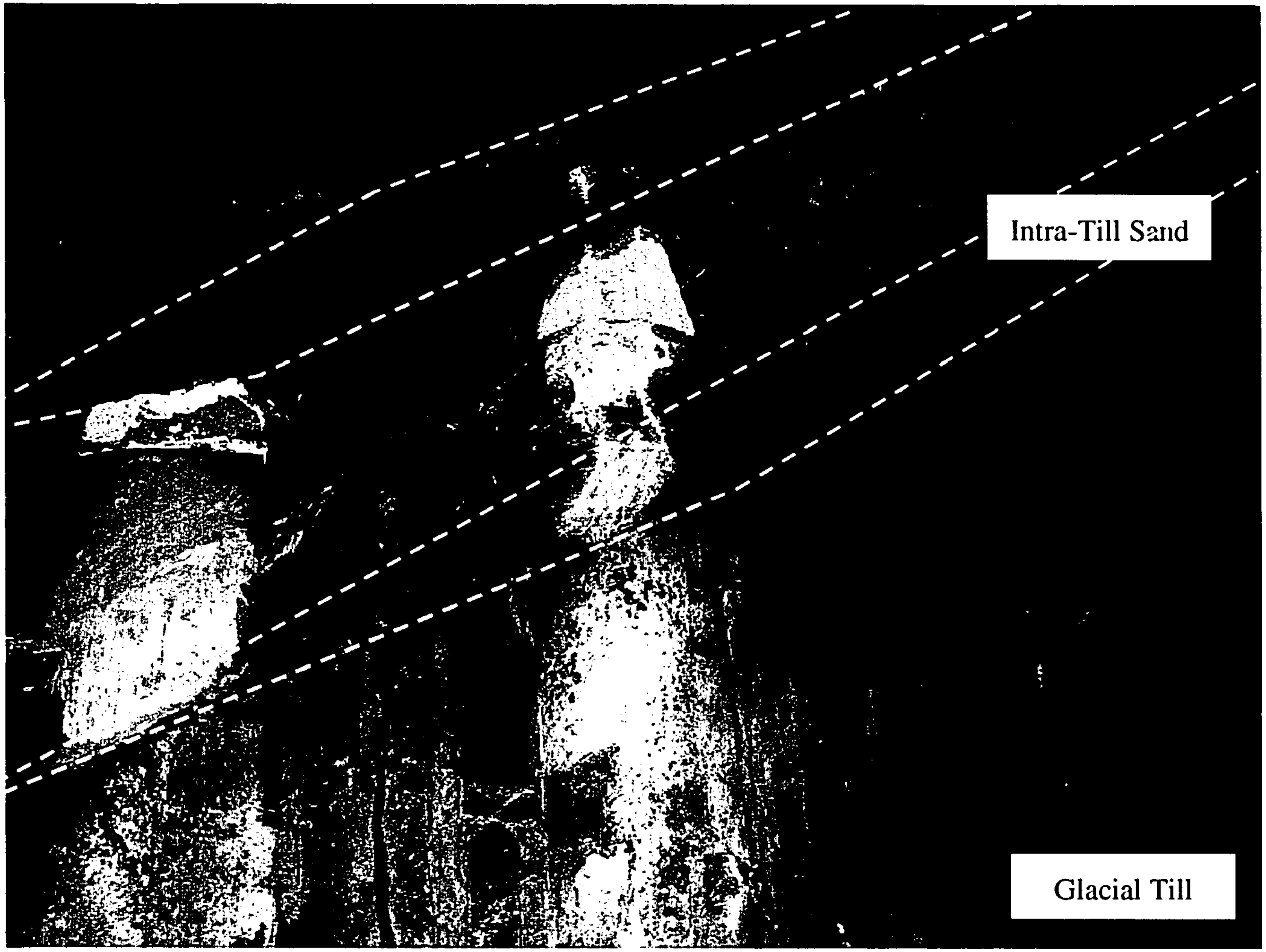


Figure 3-23 Intra-Till Sands striking across tangent pile wall in removal shaft (elev. ~657)

3.4.2 Characterisation of Glacial Till

The Glacial Till present along the tunnel alignment is an over-consolidated heterogeneous mixture of cobbles, sands, silts and clay of hard consistency. The over consolidation of the material, in addition to the stresses applied to the till during its formation ensure that it is heavily fissured. Confined compression tests carried out during AMEC's site investigation showed compressive strengths between 200 and 300 KPa. SPT(N) values for the Glacial Till ranged between 50 blows per 300mm to as high as 50 blows per 50m of penetration. Figures 3-24 and 3-25 (p 63) present histograms of SPT(N) values for blows per 300mm and 150mm respectively. Typical fine contents of the Glacial Till range between 50 and 70% (Figure 3-26, p 64).

Within the Glacial Till, discrete sand formations can be observed. Typically they are less than 2 meters in thickness and exhibit a degree of interconnectedness throughout the till mass. The fines content of the intra-till sand is similar to that of regions of the Outwash Sands and Silts with fines contents of between 7% and 15% with occasional samples possessing fines contents as high as 50% (Figure 3-27, p 64).

Observed moisture contents within the Glacial Till ranged from 10 to 27.5%. A histogram of the recorded till moisture contents can be found in Figure 3-28 (p 65). The observed moisture content of the intra-till sands ranged between 10 and 27.5% (Figure 3-29, p 65).

During the site investigation process, more than 30 boreholes and 10 CPT tests were conducted along the alignment. Appendix D – Glacial Till (p 193), contains a selection of borehole logs (PBH-7, PBH-9) relevant to the Glacial Till from the region between the Education Car Park and St. Joseph's College.

While geostatistical methods provide an avenue to mathematically characterize this uncertainty in heterogeneity no geostatistical methods were applied to explore the Glacial Till in this thesis. In the geotechnical reports issued by Golders and AMEC geostatistical methods were not applied to any significant degree. Those methods applied were restricted to the interpretation of the site investigation data as it applied to creating geologic profiles for the two tunnel alignments. This took the form of attempting to define the regions of intra-till sand within the Glacial Till. UMA's inferred geologic profile for the tunnel alignment is presented in Figure 3-1 and 3-2 (p 30, 31).

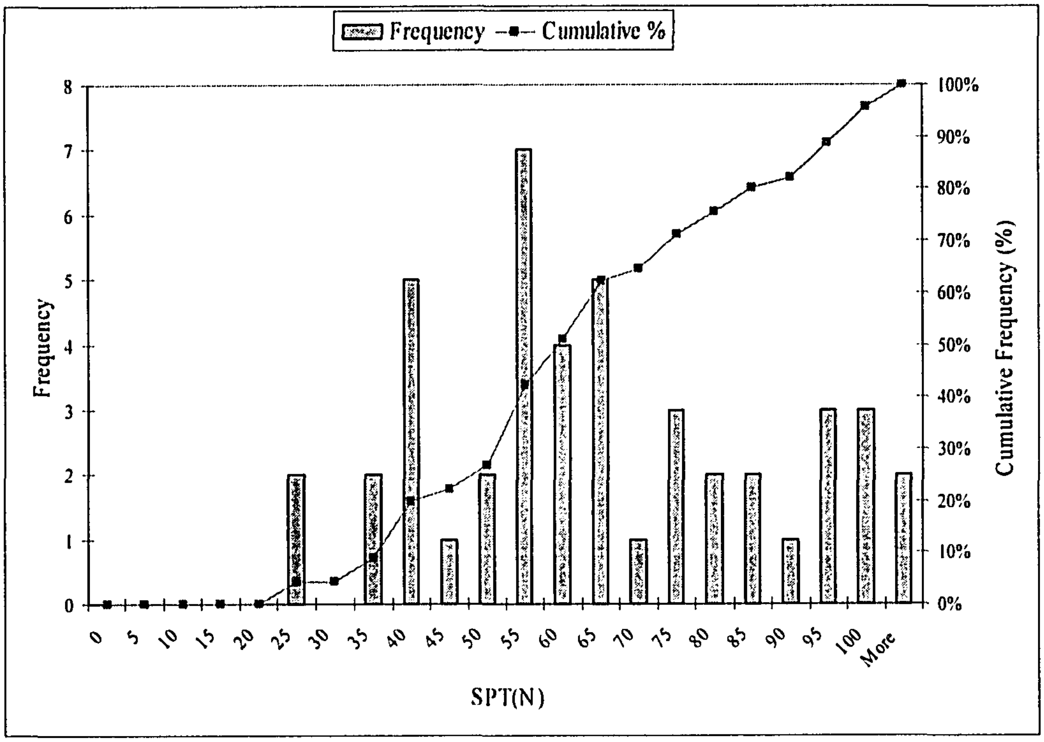


Figure 3-24 Histogram of SPT(N) blows per 300mm in Glacial Till.

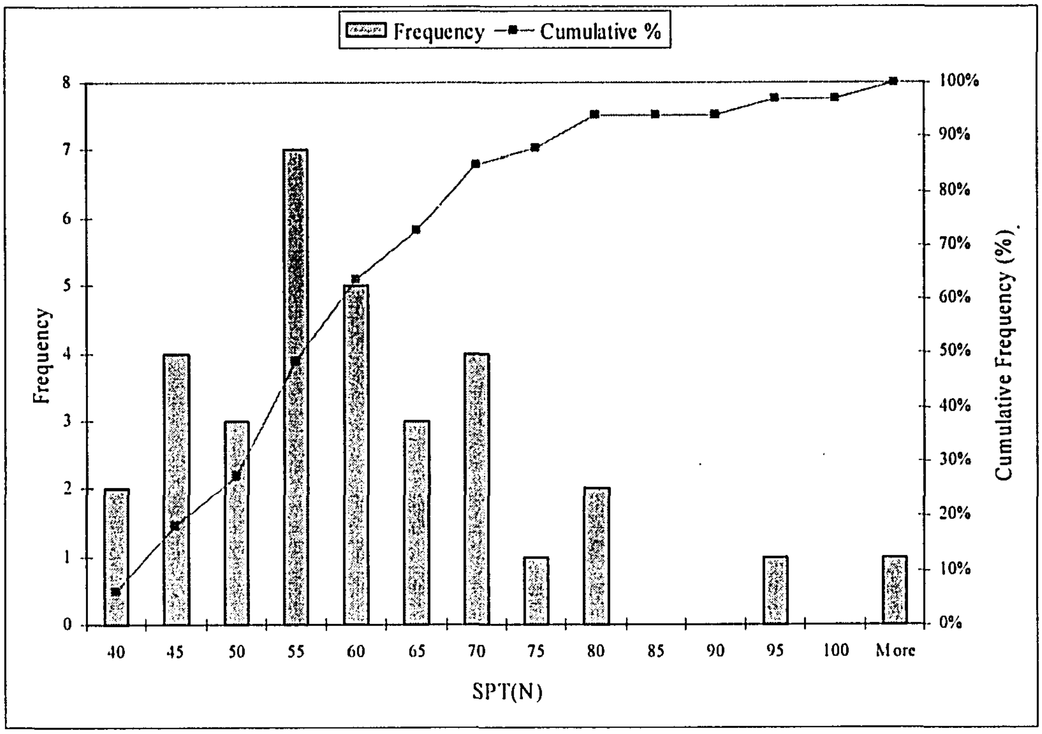


Figure 3-25 Histogram of SPT(N) blows per 150mm in Glacial Till.

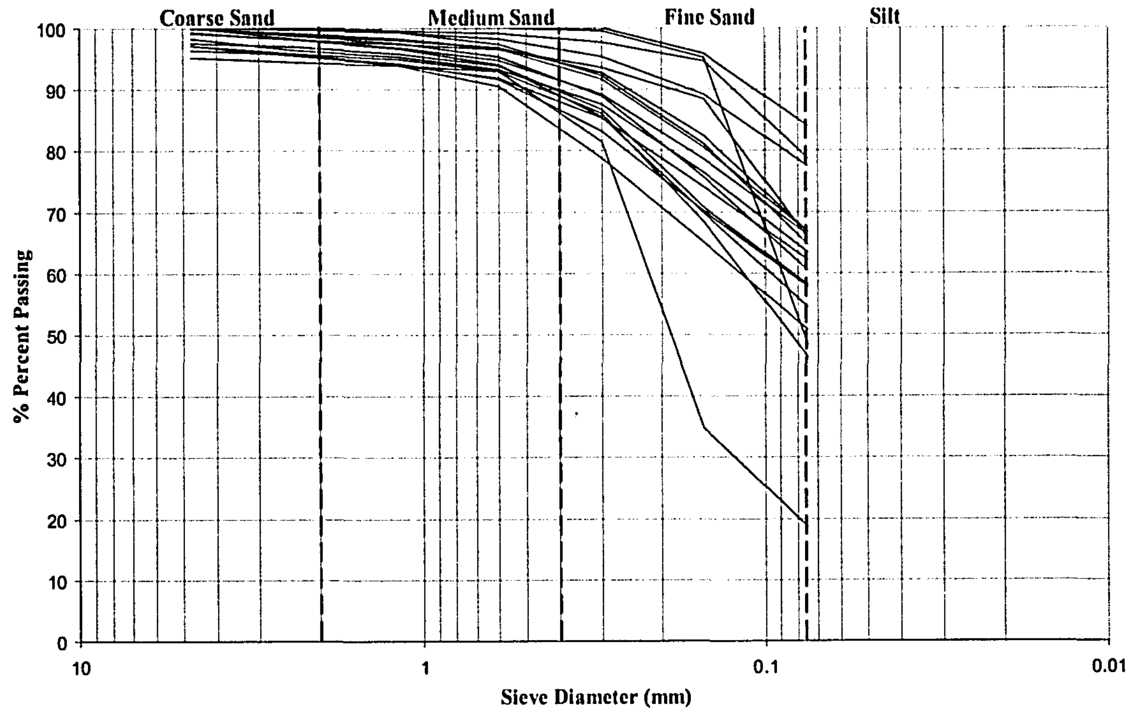


Figure 3-26 Composite grain size distribution of Glacial Till from Golders' site investigation report.

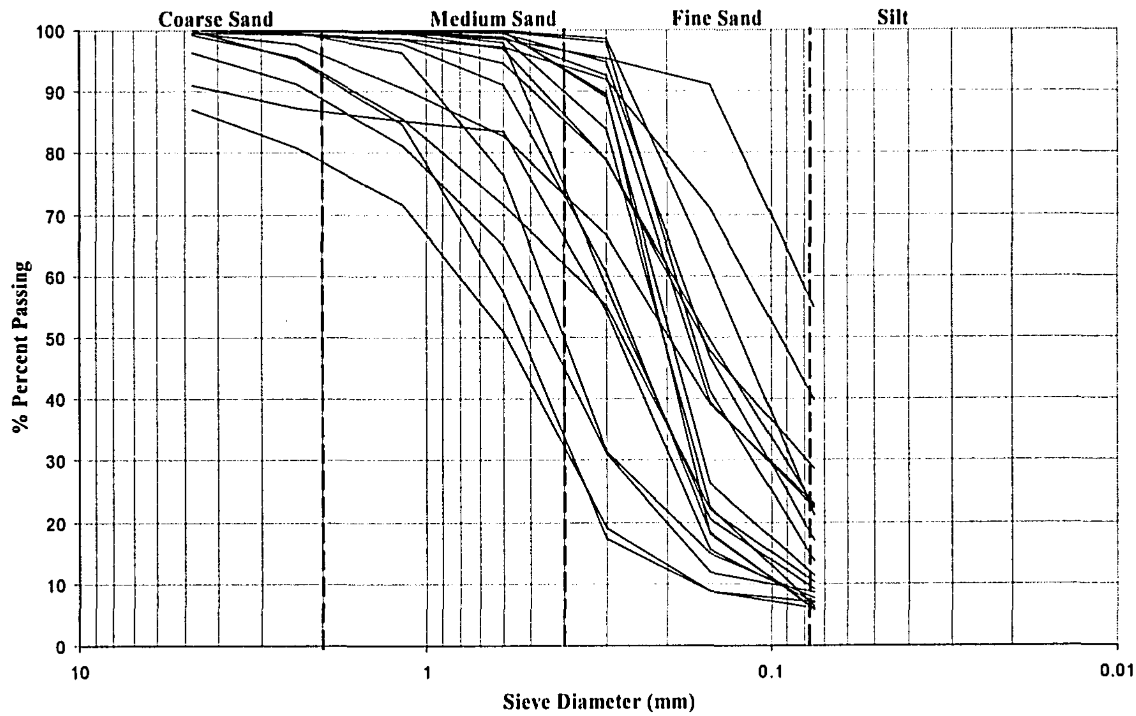


Figure 3-27 Composite grain size distribution of Intra-Till Sands from Golders' site investigation.

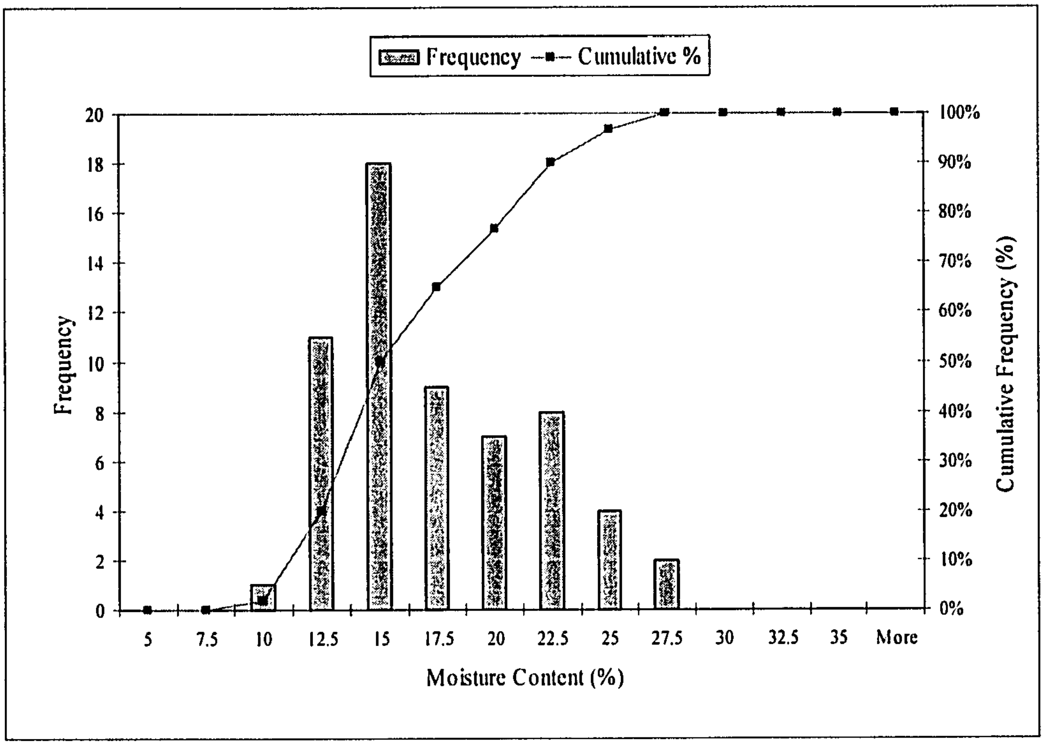


Figure 3-28 Histogram of observed Moisture Contents of the Glacial Till.

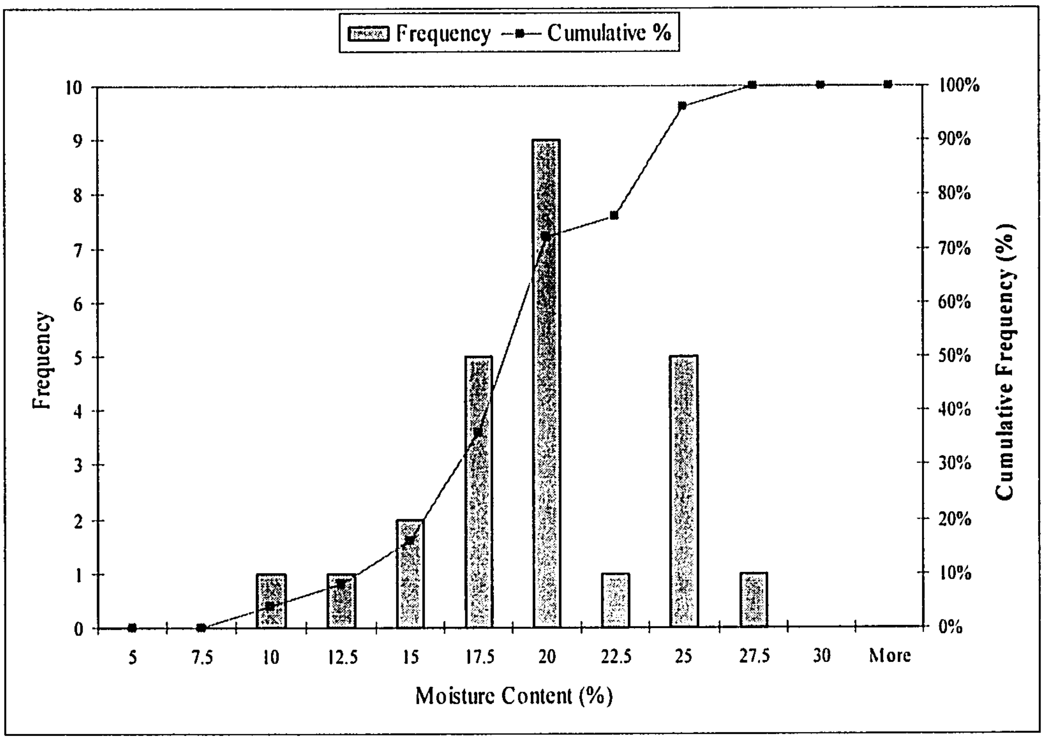


Figure 3-29 Histogram of observed Moisture Contents of the Intra-Till Sands.

3.5 Bedrock

The Bedrock present in the Edmonton SLRT site and much of the Edmonton region belongs to the Edmonton Formation. Like the Lake Edmonton Clay, the Bedrock is only present over a limited extent of the alignment. Based on the geotechnical investigations and interpretations contact with the Bedrock varies greatly between tunnels: in the vicinity of Stn. +525m in the Southbound Tunnel, and Stn. +475m in Northbound Tunnel. After these points, the Bedrock gradually assumes a larger portion of the face area. As the desired depth of the tunnel was nearly reached when contact with the Bedrock was made, full face tunnelling within the Bedrock did not take place. Construction after Stn. +525m and +475m took place in mixed face conditions, with Bedrock occupying the invert and Glacial Till the crown.

3.5.1 Formation Process of Bedrock

Formed during the late Cretaceous period, during the formation and erosion of the Cordillera and Rocky Mountains, the Edmonton Formation Bedrock consists of a number of elements; claystone, sandstone, siltstone, coal and the occasional bentonitic clay seam. In the region occupied by the SLRT alignment, the bedrock consists largely of claystone with limited quantities of sandstone. During the site investigation limited amounts of sandstone were identified along the Northbound Tunnel alignment. The sandstone identified during the site investigation was likely emplaced by glacial action, and might represent a localised stratigraphic layer.

During the glaciation prior to the deposition of the Glacial Till, the surface of the bedrock underwent extensive excavation and reworking. As a result, the zone of clay shale encountered in tunnelling is heavily slickensided as a result of the stresses placed on the bedrock by the movement of the glaciers. Figure 3-30 (p 67), taken during the construction of the removal shaft, shows the relatively clear transition between the light grey bedrock and the overlying glacial till. Figure 3-31 (p 68) taken at the same time as Figure 3-30 also illustrates the transition between bedrock and glacial till.

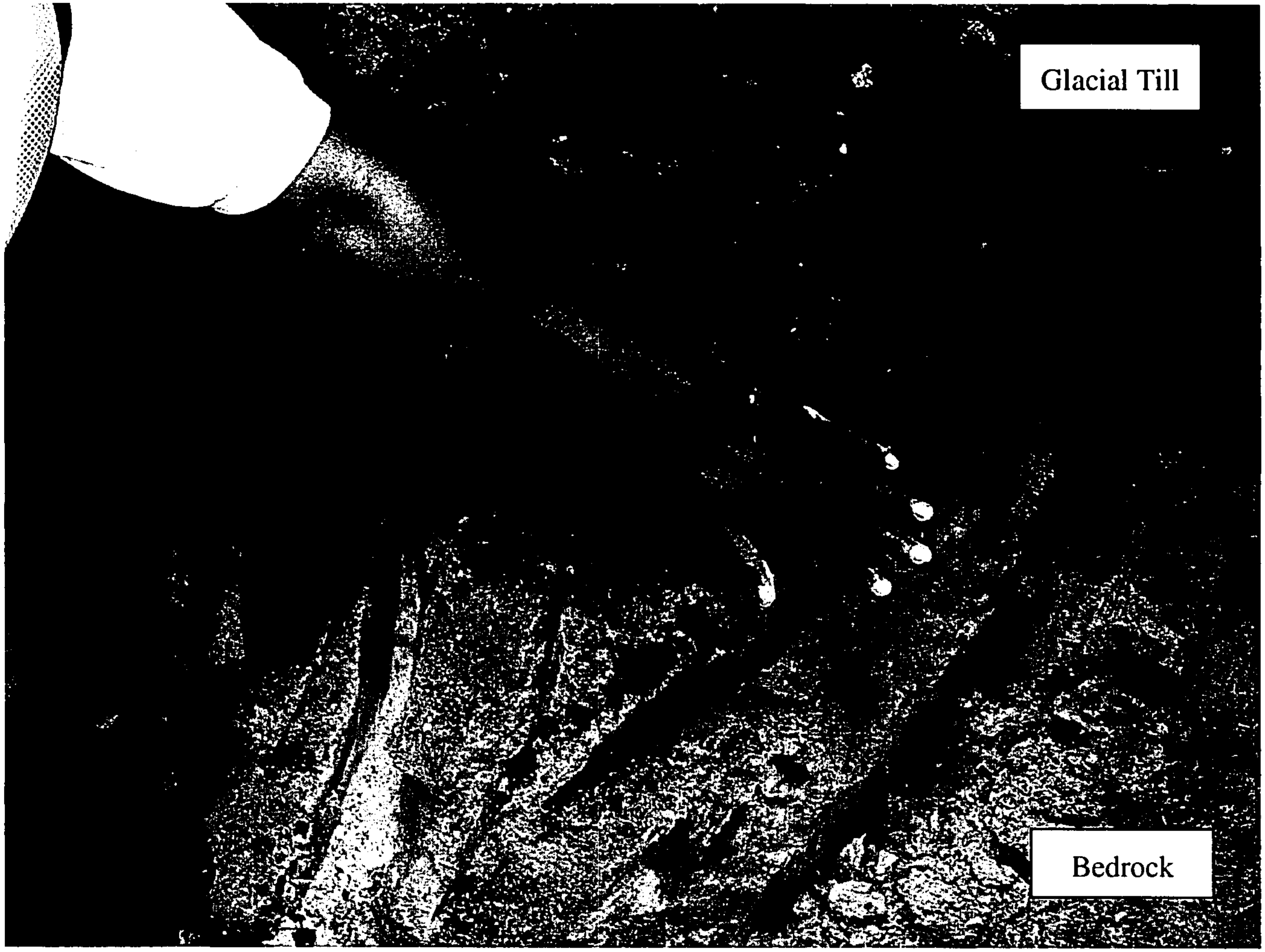


Figure 3-30 Transition from Bedrock to overlying Glacial Till (elev. ~652m).

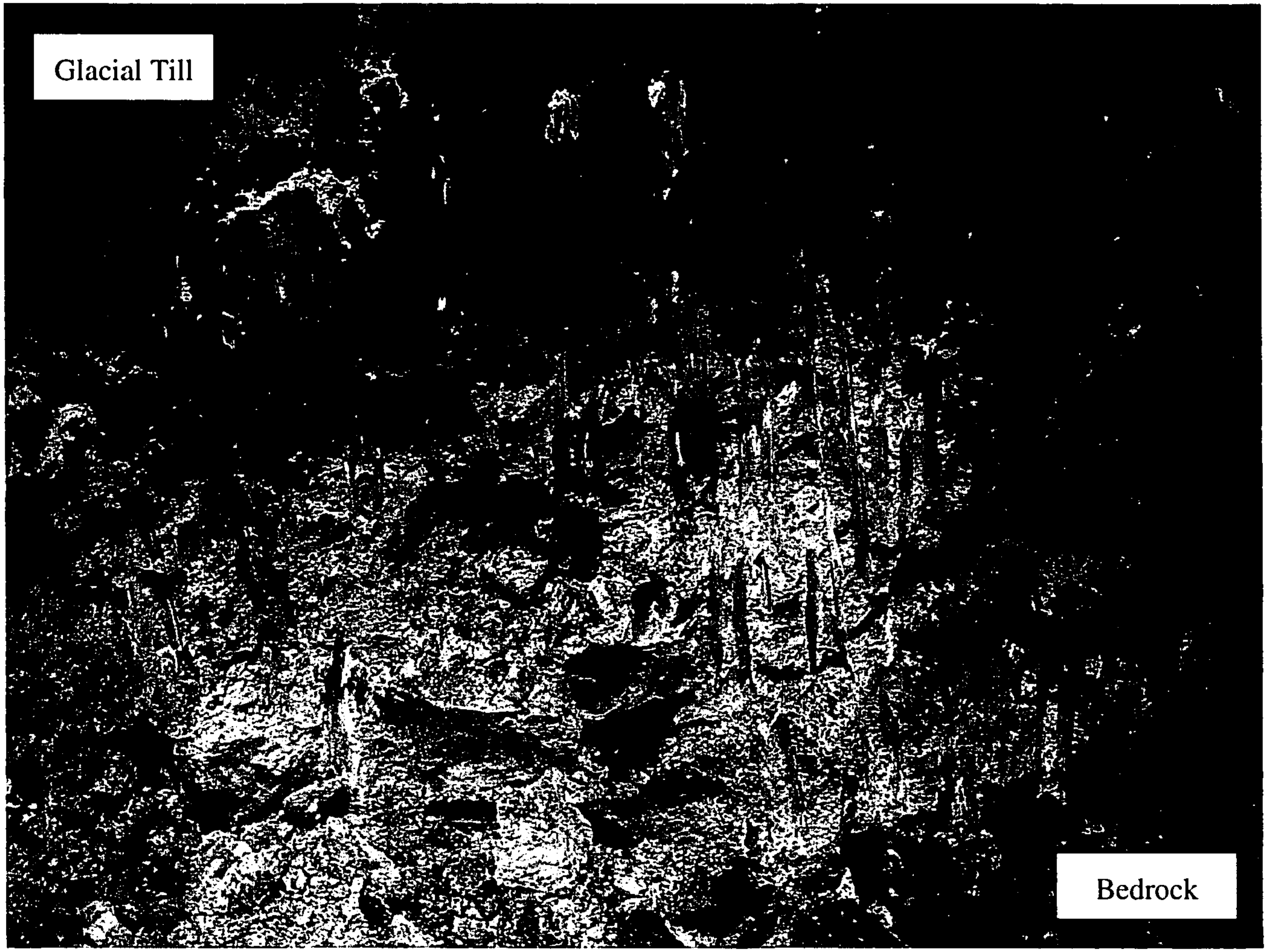


Figure 3-31 Transition from Bedrock to overlying Glacial Till (elev. ~653m).

3.5.2 Characterization of Bedrock

The Edmonton Formation Bedrock present throughout the tunnel alignment consists primarily of deformed, weathered claystone with occasional deposits of sandstone. The AMEC site investigation recorded SPT(N) values of between 48 blows for 300mm of penetration, to 50 blows per 25mm of penetration within the claystone. Within the sandstone, SPT(N) values between 57 blows for 300mm of penetration to 50 blows per 25mm of penetration were obtained. Figure 3-32 (p 70) presents the histogram for SPT(N) values in the Bedrock achieving penetration of 300mm.

Moisture contents within the Bedrock ranged between 15 to 25% using a probability interval of 80%. However the extreme moisture contents of the bedrock ranged between 7.5 to in excess of 30%. Figure 3-33 (p 70) presents a histogram of the moisture content obtained during the site investigations carried out by Golders and AMEC.

One characteristic of the claystone present is its rapid degradation when exposed to air and water. This behaviour can be observed in sections of the North Saskatchewan River valley where the claystone is exposed. When exposed to large amounts of water, the claystone degrades into a light grey paste with little strength.

Three boreholes were selected, PBH-1, PBH-2, and 00-01 with the relevant portions of the borehole logs located in 'Appendix E – Bedrock' (p 198). These boreholes were part of the site investigation process concentrated in the region from St. Joseph's College to the removal shaft at the end of the tunnel.

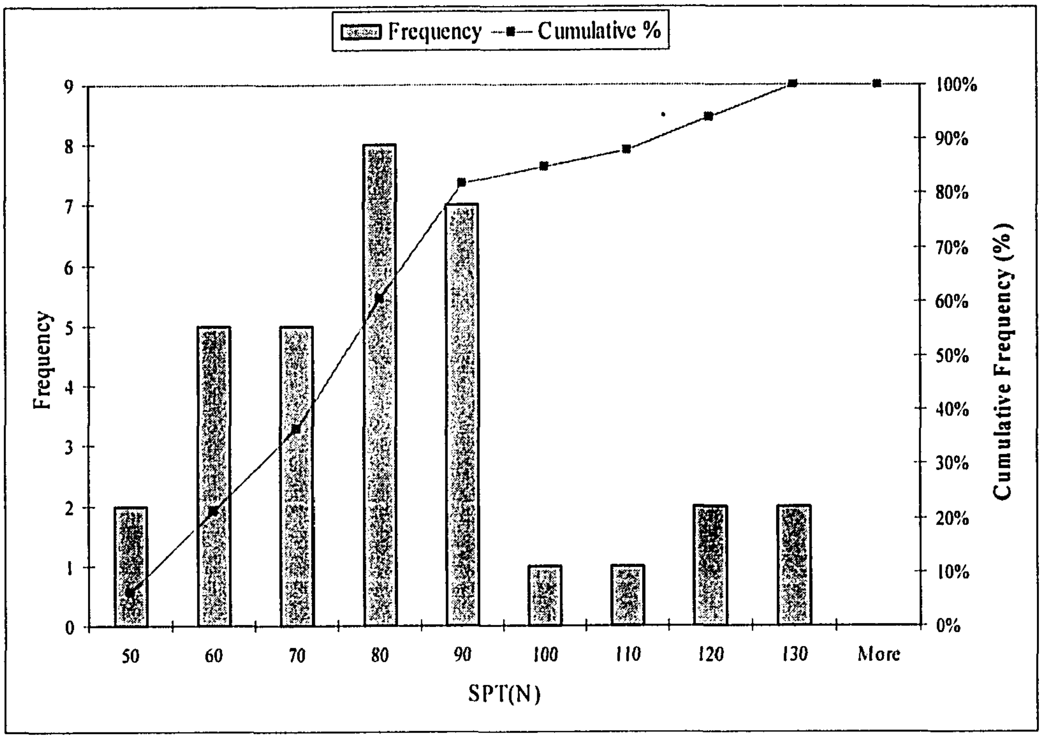


Figure 3-32 Histogram of SPT(N) values per 300mm in Bedrock.

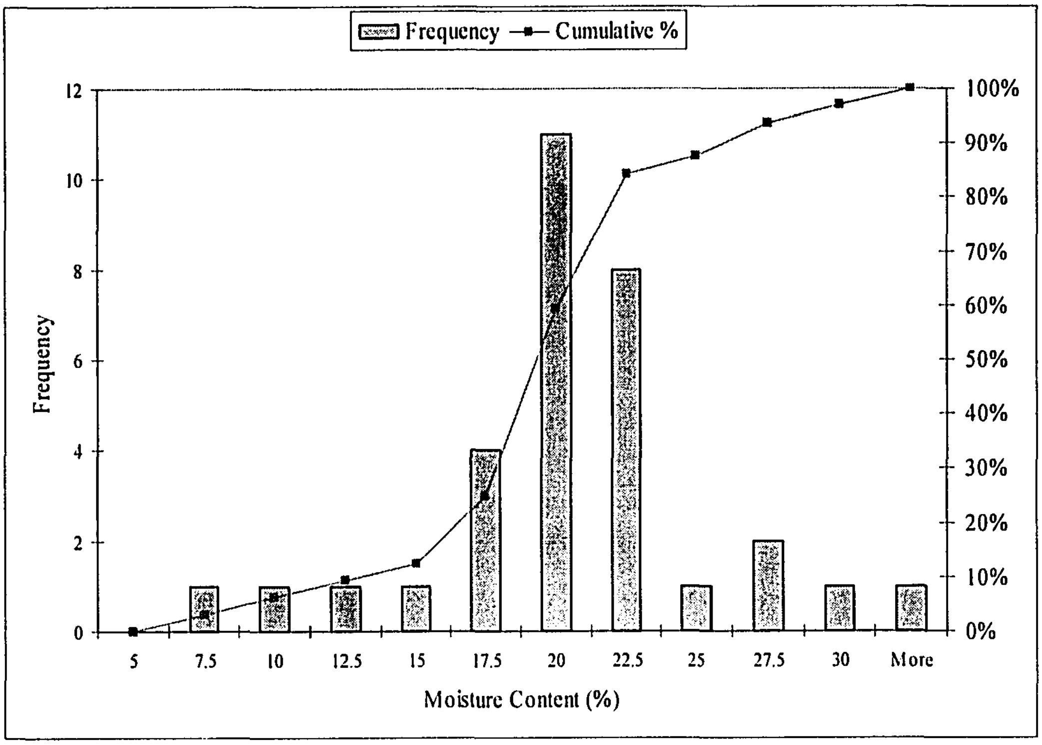


Figure 3-33 Histogram of observed Moisture Contents of the Bedrock.

4.0 Field Measurements Collected During Construction of Southbound and Northbound Tunnels

This Section presents the information obtained during the construction of the Southbound and Northbound Tunnels. The Section presents the characteristics of the excavated soil, the information logged manually and by the TBM computer during tunnelling, the observed settlements induced by tunnelling, and the strength development of the grout used for tail shield grout injection.

4.1 Collection of TBM Spoil

In the detailed discussion of local geology in Section 3.0 (p 29) it was stated that the main materials of interest were the Outwash Sands and Silts, Glacial Till with intra-till sands, and Bedrock. All samples collected during construction of the Edmonton SLRT were obtained from the surficial muck bin. The muck bin was the storage pit for the excavated material prior to its being loaded onto trucks and removed from site. Figure 4-1 (p 76) shows the muck bin and one of the train cars used to transport the excavated soil from the tunnel as the muck is being dumped into the muck bin. Material was collected from the muck bin using a long staff with a scoop attached. If material was dumped near to the sides of the muck bin, it could be collected from the edge of the bin. If material was dumped away from the sides of the muck bin, an excavator was used to collect material from the center of the muck bin (the assistance of the excavator operator was appreciated).

After collection, material was placed in a numbered series of pails and the sample information recorded (date, ring #, sample #). During construction of the Southbound Tunnel samples were collected every 2 to 3 rings, 2.4m to 3.6m, and in the case of the Northbound Tunnel, every 3 to 5 rings, 3.6m to 6m.

Sample sizes collected for each ring were in the order of 5 to 8kg. This material was used to determine the bulk moisture content of the spoil, and the grain size distribution. Moisture contents were determined by drying the entire 5 to 8 kg sample. Figure 4-2 (p 76) shows the soil in a wet state when collected, and after drying.

After drying, the soil mass was broken up and between 0.8 and 1.5kg of dried soil was collected. Sample sizes depended on the character of the spoil. Spoil consisting of primarily Outwash Sands and Silts with some degree of Glacial Till were tested at the lower end of the sample size range. Whereas Glacial Till and Bedrock samples were tested using sample sizes at the higher end of the scale. Each sample was then washed through a #200 sieve to remove as much of the fines content of the sample as possible. This step was found to be necessary to provide accurate measurement of grain size distributions. Without washing, samples collected in the siltier regions of the Outwash Sands and Silts would appear to be upwards of 20% coarser. As a result, all samples collected during construction were washed manually through a #200 sieve to ensure accurate data.

Figures 4-3 (p 77) and 4-4 (p 78) illustrate the moisture contents of the soil collected during tunnelling and the inferred moisture content of the in-situ soil for the Southbound and Northbound Tunnels respectively. The inferred moisture content was obtained by using the geologic profile issued in the Geotechnical Baseline Report and then calculating the average moisture content of the material present at the face of the TBM. The smoothness of the inferred curve is a result of the smoothness of the geology at the face of the TBM as presented in the Geotechnical Baseline Report. Marked in each Figure is the extent of the tunnel alignment that underlies the Education Car Park and St. Joseph's College. Also presented in each Figure is the percent area of face occupied by the three major stratigraphic layers, Outwash Sands and Silts, Glacial Till, and Bedrock. These two Figures show the amount of water introduced to the soil through the conditioning process. It can be noted that during the construction of the Southbound Tunnel, the moisture content of the TBM spoil was consistently increased by approximately 8%, from 11% to 16% in-situ to 18% to 22% spoil, from the start of construction until Stn. +510m, was reached. Stn. +510m corresponds to the region where Bedrock was first encountered. After Stn. +510m, the change between inferred in-situ and spoil moisture content showed a more erratic trend. However, the typical increase in moisture content between in-situ and the spoil remained on the order of 8%. In the construction of the Northbound Tunnel, it was observed from the start of construction to Stn. +580m, the spoil moisture content follows a similar trend as in the Southbound Tunnel, showing wetting in the order of 8%. After Stn. +550m, the soil conditioning became far drier than during the construction of the Southbound Tunnel with the typical wetting being on the order of 4%. The observed spikes in moisture content, positive and negative, observed during Southbound construction and associated with operating in the Glacial Till and Bedrock, are present but not of the same magnitude as during Northbound construction.

Figures 4-5 (p 79) and 4-6 (p 80) present the grain size distributions from the Southbound and Northbound Tunnel spoil, respectively. Also included in these two Figures is the moisture content of the spoil. Marked in each Figure is the extent of the tunnel alignment that underlies the Education Car Park and St. Joseph's College.

In constructing the Southbound LRT Tunnel the site investigations indicated that early construction would take place, full face in the Outwash Sands and Silts with Glacial Till being intercepted in the region of Stn. +640m. It can be seen in the grain size distribution in Figure 4-5, that at approximately Stn. +625m a material with silt contents much greater than 25% had been encountered, suggesting contact with the Glacial Till. Prior to this point, the material obtained was characteristic of the Outwash Sand and Silts, the fines fraction being in the order of 22%.

From this stage of construction, Stn. +625m until Stn. +510m, Glacial Till transitioned from being on the invert to occupying the full face of the TBM. Along the alignment at Stn. +450m, +510m, and +570m, coarsening trends in the TBM spoil were observed, suggesting contact with large quantities of intra-till sands in those regions. In the regions within the Glacial Till, with moderate quantities of intra-till sands, the fines content of the material was in the order of 45% with a 5% margin. In regions with extensive deposits of intra-till sands, fines contents in the order of 30% to 32% were observed.

Contact with the Edmonton Formation Bedrock was anticipated as early as Stn. +525m during the construction of the Southbound Tunnel based on the site investigations carried out. However, the appearance of significant quantities of Bedrock was not observed until after Stn. +500m, although flakes of bedrock in the spoil had been observed in the muck collected for sampling prior to that time. Construction after Stn. +500m took place with a mixture of Bedrock, intra-till sands and Glacial Till at the TBM face. Between Stn. +500m and +470m, the mix of Glacial Till and Bedrock produced a spoil with fines contents between 52% and 64%. In the region of Stn. +510m and +450m spoil, fines contents were in the order of 35% and 45% respectively, suggesting contact with large quantities of intra-till sand. After passing the intra-till pocket located at Stn. +450m, a bedrock rich region was encountered producing fines contents as high as 70% at Stn. +435m. After reaching a maximum of 70% at Stn. +450m, the fines content lessened to being in the order of 55% by Stn. +425m.

In constructing the Northbound Tunnel the resultant grain size distribution (Figure 4-6, p 80), while similar in form to that of the Southbound Tunnel (Figure 4-5, p 79), reflects distinct and different conditions even though the two tunnels were separated only by a short distance. Early in the construction of the Northbound Tunnel, a large zone of fines was encountered in the region of Stn. +650m, with fines contents as high a 50%, suggesting a large silt deposit within the Outwash Sands and Silts. After Stn. +650m, the fines contents of the TBM spoil were in the order of 30% by Stn. +640m. The next marked change in character of the TBM spoil was noted at Stn. +600m, which indicated the coming into contact with either significant quantities of till, or a combination of till and silt deposits within the Outwash Sands and Silts. Between Stn. +600m and +575m, fines contents in the order of 42% were observed in the TBM spoil. Between Stn. +575m and Stn. +525m the fines fraction of the TBM spoil dropped to approximately 38%. This lower percentage suggested the presence of large quantities of intra-till sands within the Glacial Till, as the TBM face was likely to be near full face within the Glacial Till by Stn.

+575m. Between Stn. +510m and Stn. +480m, the observed fines content increased from 40% to 60%. This region corresponds to the anticipated first contact with Bedrock, in the vicinity of Stn. +500m. As in the Southbound Tunnel, a large region of intra-till sands was then encountered between Stn. +475m and Stn. +425m, with fines contents reduced to approximately 35%. After Stn. +425m, increasing bedrock content was noted and the fines content increased from 35% to 70% by the time the tunnel had reached the removal shaft.

In both Figures 4-5 (p 79) and 4-6 (p 80) the moisture contents of the TBM spoil were included. The general character of the spoil moisture content appeared as a subdued reflection of the fines content of the material being excavated. Aside from upsets in process control, where excessive quantities of foam were injected, examples being significant spikes in spoil moisture content at Stn. +625m, +530m, and +460m during the construction of the Southbound Tunnel, changes in the fines content of the in-situ material larger than 10% were reflected in the spoil moisture content. The more dramatic the change in fines content, the more dramatic the change in spoil moisture content.

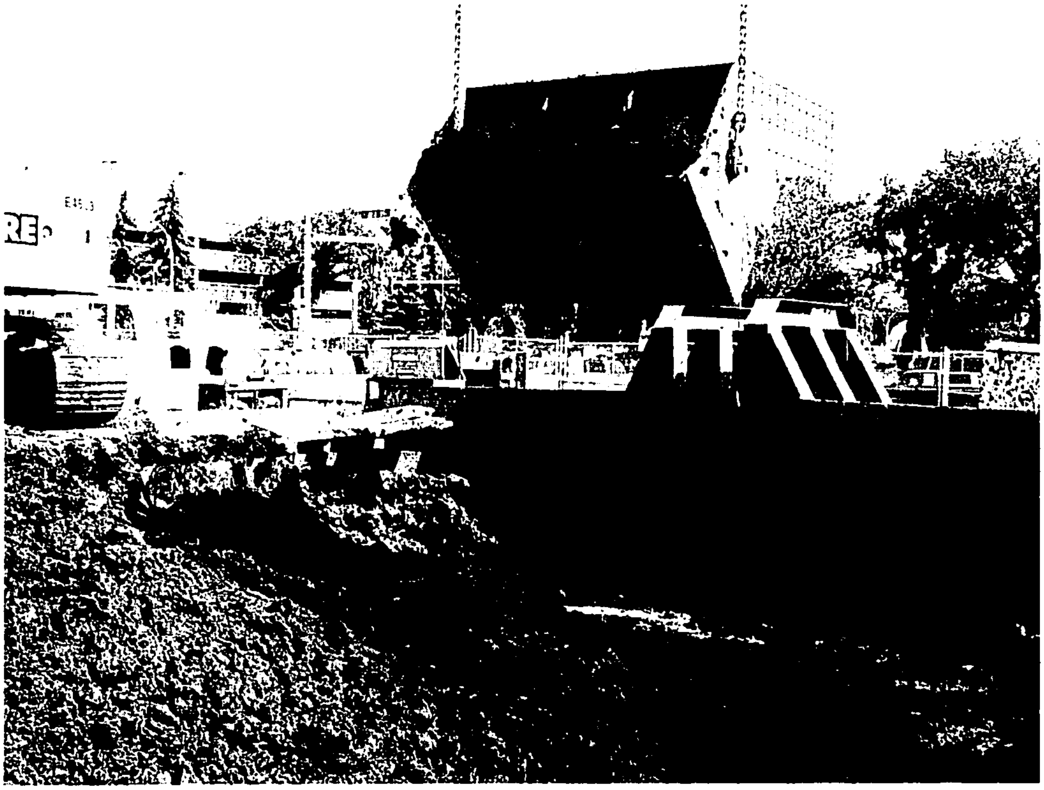


Figure 4-1 TBM spoil being dumped into muck bin.

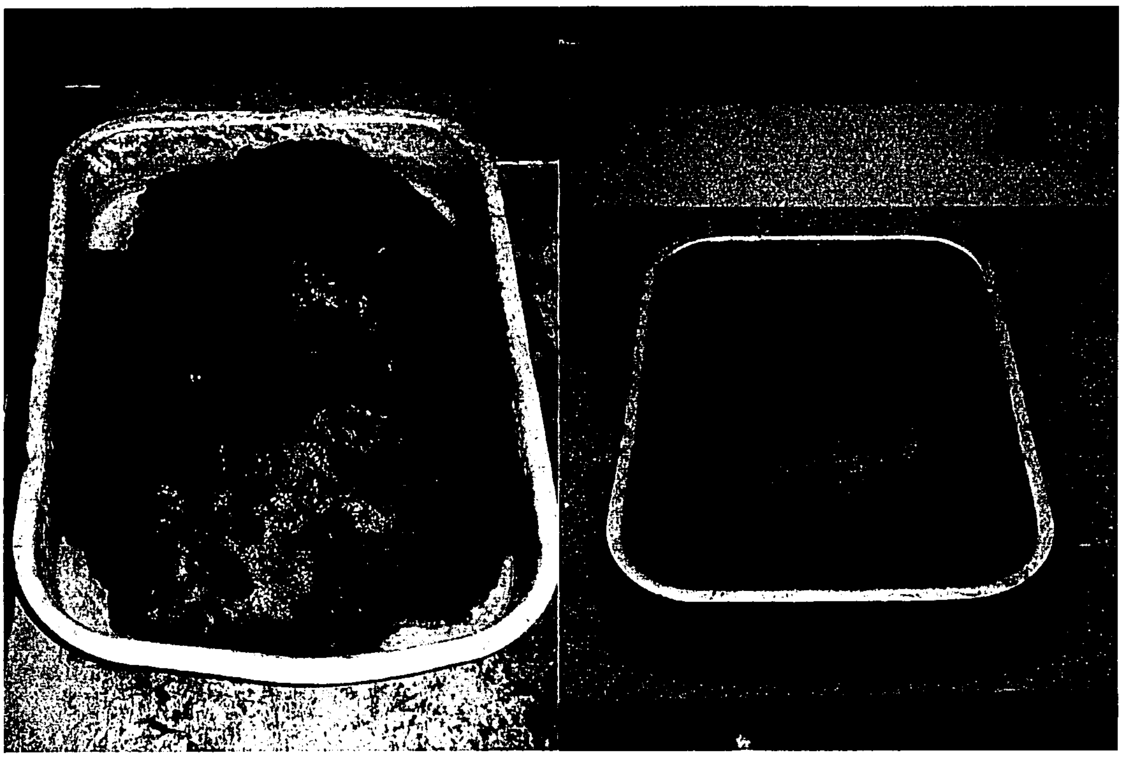


Figure 4-2 TBM spoil sample used in testing, wet on left, dry on right.

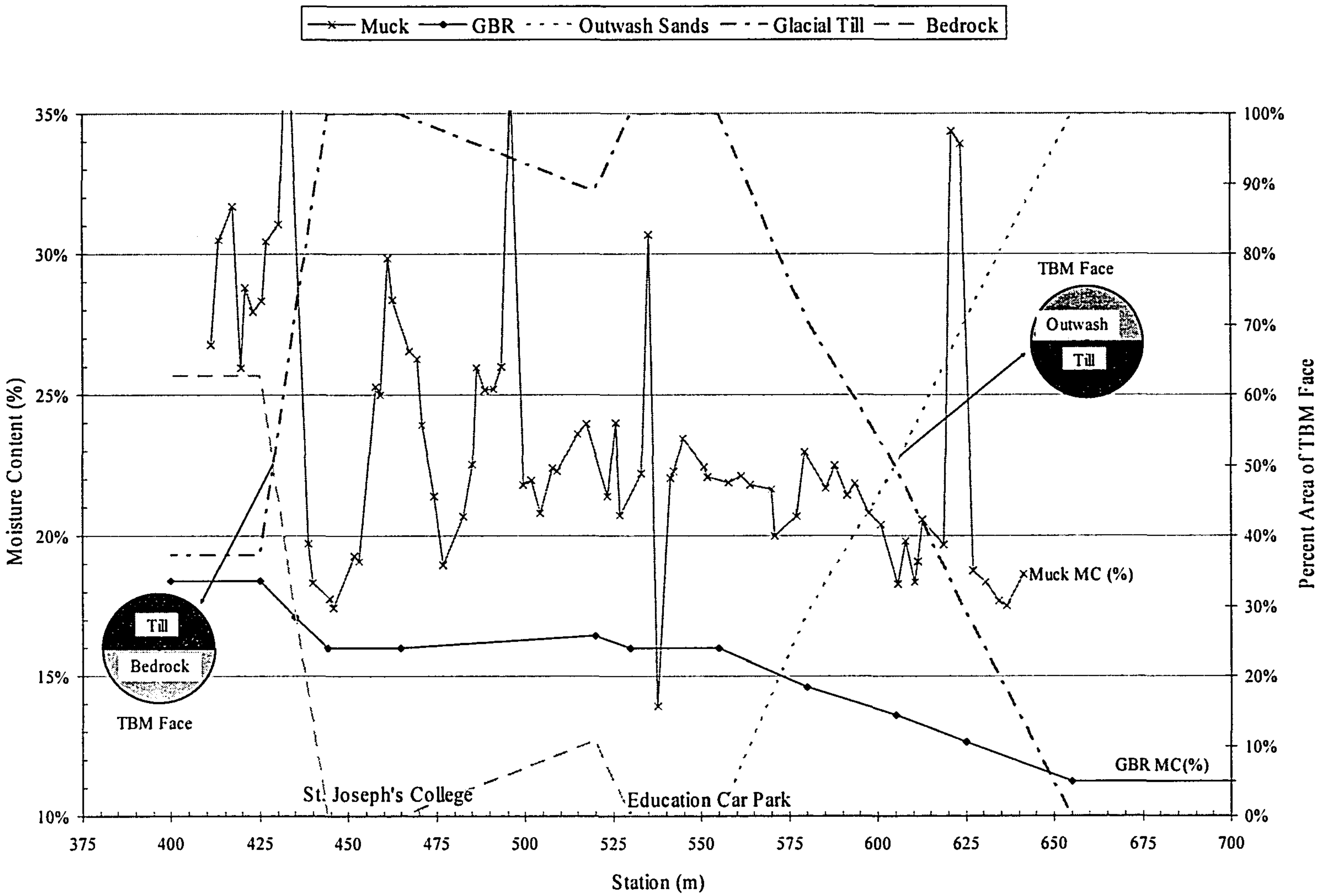


Figure 4-3 Moisture content of material excavated during construction of the Southbound LRT Tunnel and the inferred in-situ moisture content of the soil.

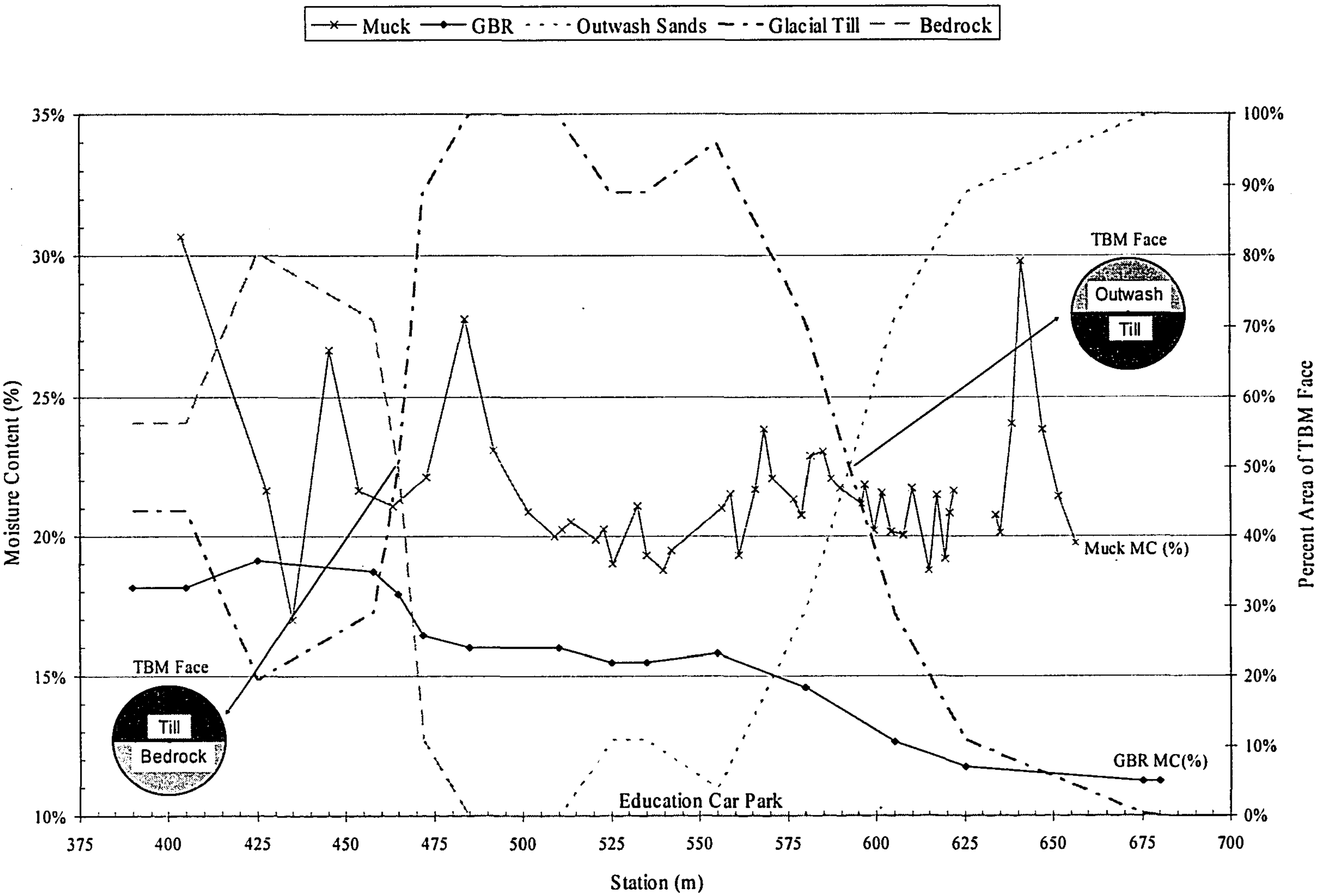


Figure 4-4 Moisture content of material excavated during construction of the Northbound LRT Tunnel and the inferred in-situ moisture content of the soil.

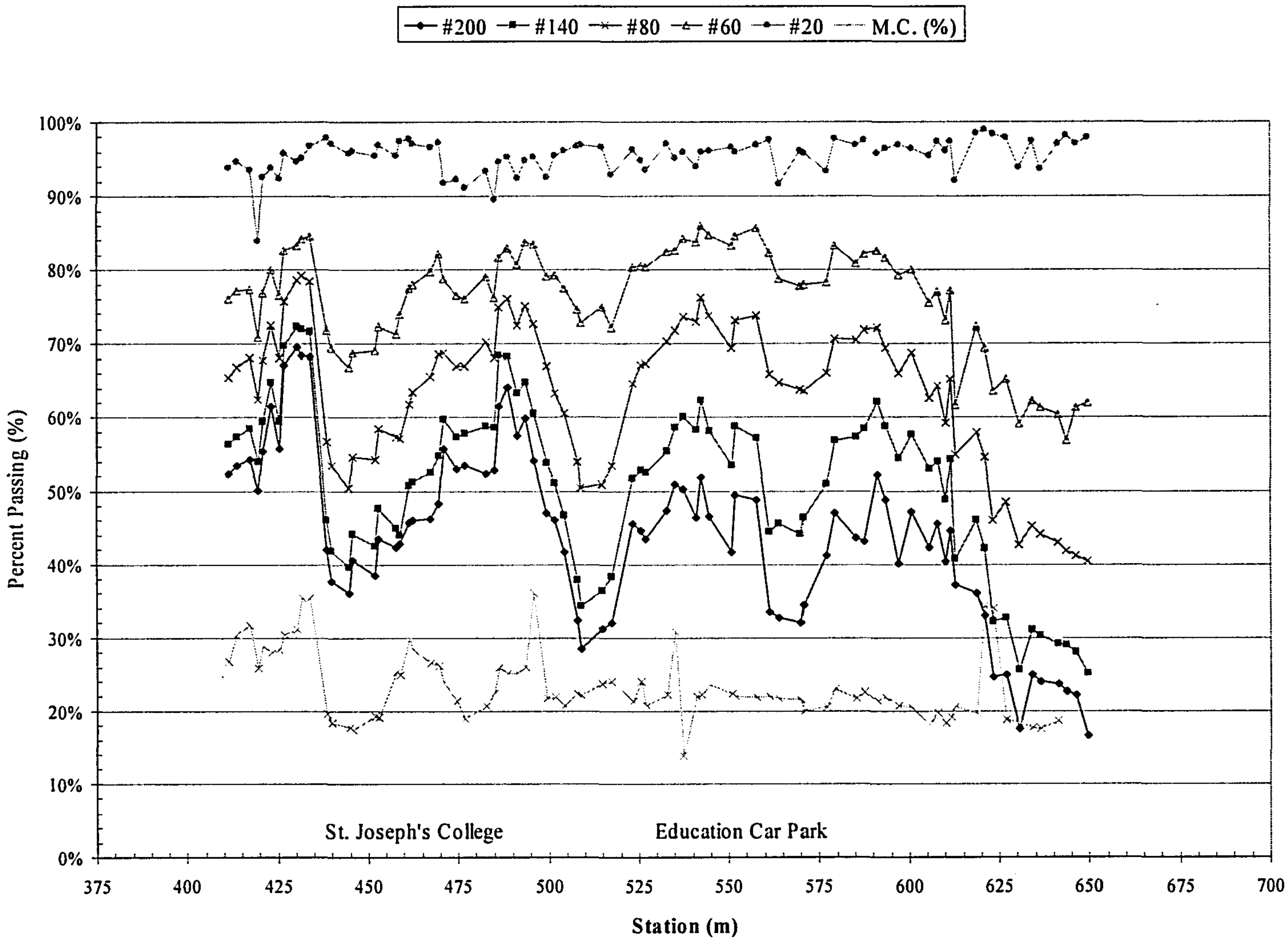


Figure 4-5 Grain size distribution and moisture content of material excavated during construction of Southbound LRT Tunnel.

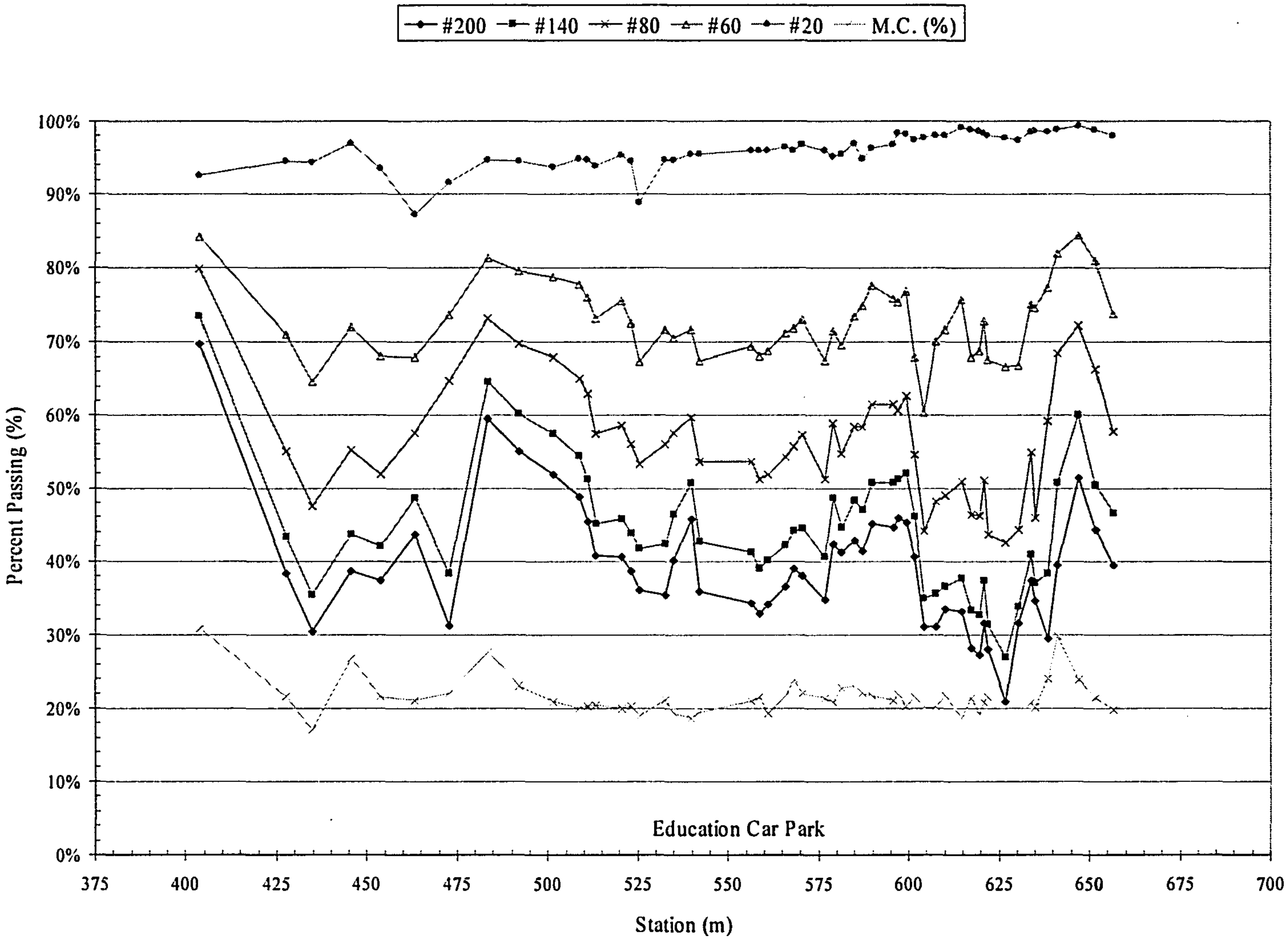


Figure 4-6 Grain size distribution and moisture content of material excavated during construction of Northbound LRT Tunnel.

4.2 Information collected from Construction Logs and TBM Data Logger

During construction of the North and Southbound Tunnels extensive data logging was undertaken by the tunnelling contractors, Aecon-McNally, in both manual and automatic form. This extensive set of records allowed for the reconstruction of the operation of the TBM using a variety of parameters (construction of the Southbound Tunnel produced upwards of 12,000 computer log files). During the construction of the North and Southbound Tunnels, the computer logging systems operated 24 hours a day logging parameters at ten-second intervals.

4.2.1 Operational torque experience by TBM during Southbound Tunnel construction.

Figure 4-7 (p 83) presents the operational torque of the cutting head during the construction of the Southbound LRT Tunnel. Shown on the plot are the mean and the standard deviation of the torque applied during excavation. The mean torque was determined by identifying all log entries made while the TBM was advancing along the alignment. These values were then transferred to a separate spread sheet where they were averaged, and the standard deviation calculated for each tunnel ring number.

Tunnelling within the Outwash Sands and Silts generated cutting head torques between 3100 KN*m and 3400 KN*m prior to Stn. +625m. As Glacial Till began to occupy a greater portion of the face area after Stn. +625m, the observed torque gradually increased from ~3400 KN*m to ~3900 KN*m by Stn. +510m. It was in this region, Stn. +510m, that contact with the Bedrock was made. From this point in construction, operating in mixed face conditions of Glacial Till and Bedrock, the observed torques were in the range of 4000 KN*m to 4100 KN*m. However in the region of Stn. +475m to +450m, where a large pocket of intra-till sands was encountered, the observed torque decreased to ~3800 KN*m.

The standard deviation of torque observed when tunnelling within the Outwash Sands and Silts was initially in the order of 700 KN*m in the region of Stn. +650m. The anomaly of zero torque at Stn. +650 was the result of mechanical break down and technical difficulties with the TBM.7 By the time contact was made with the Glacial Till at Stn. +625m, the standard deviation had decreased to ~550 KN*m. The standard deviation remains at or near this value until Stn. +600m. After Stn. +600m, the standard deviation decreased to ~500 KN*m. It remained at this order for the remainder of construction.

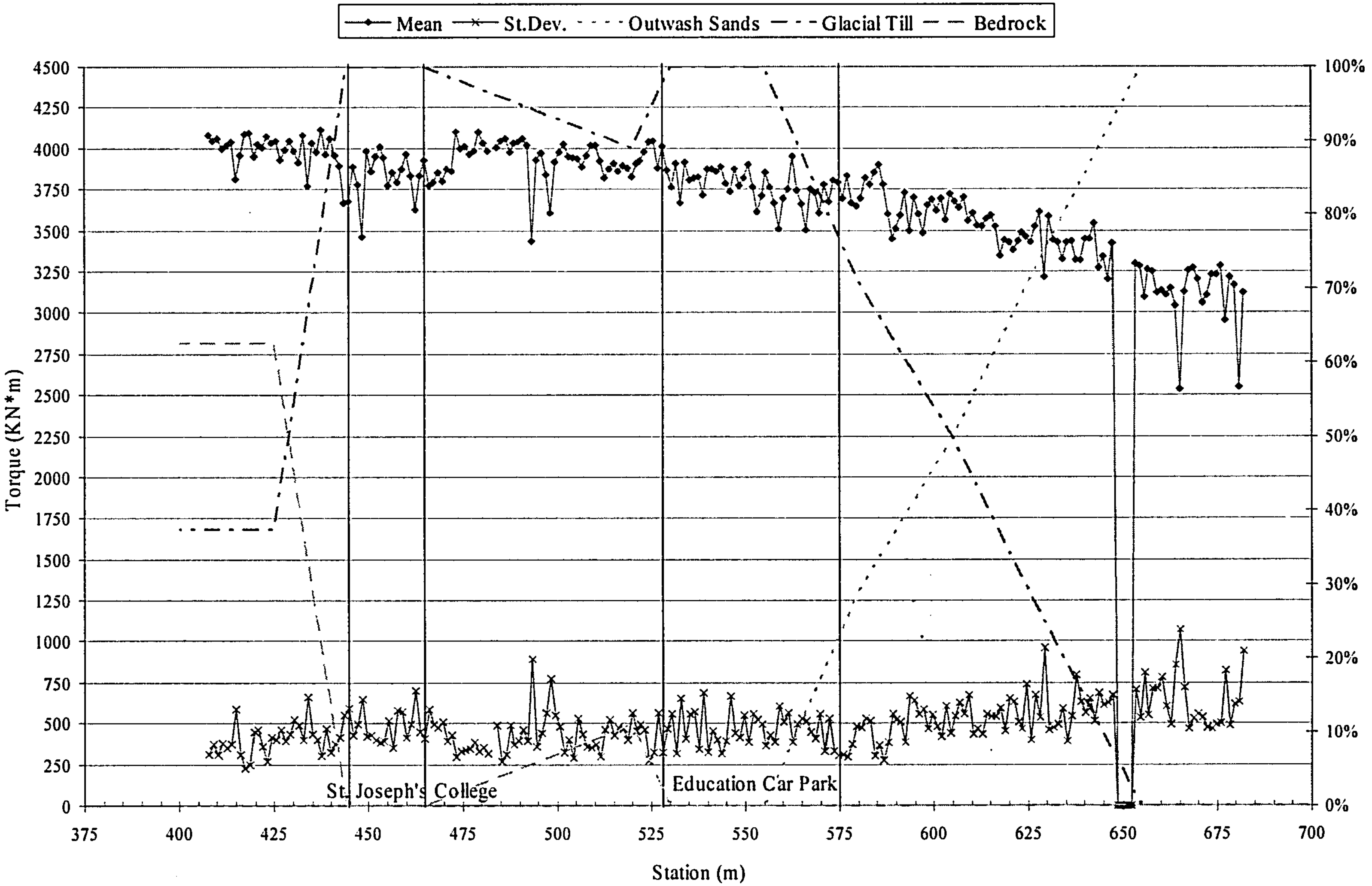


Figure 4-7 Cutting-head torque recorded during advance of TBM, Southbound Tunnel.

4.2.2 Observed face pressures during construction of Southbound Tunnel.

Figure 4-8 (p 85) presents the face pressures logged during construction of the Southbound Tunnel. The face pressure data was obtained from the mid-level pressure sensors, located roughly halfway down the face within the excavation chamber. The value for the mean face pressure was determined by using the data entries for face pressure over the entire installation of any given ring.

Tunnelling within the Outwash Sands and Silts between Stn. +675m and +625m, the observed mean face pressure was in the order of 1.6 bar. After making contact with the Glacial Till in the region of Stn. +625m, the mean face pressure from this point to Stn. +600m decreased to 1.3 bar. After Stn. +600m, the mean face pressure increased to a range of 1.9 to 2.1 bar from Stn. +600m to Stn. +525m. Stn. +525m marked the end of the passage underneath the Education Car Park and by this stage of tunnelling, the TBM face was full-face in the Glacial Till. From Stn. +525m to +450m, where operation was within a mixed face of Glacial Till and Bedrock, the mean face pressure decreased from 1.5 bar to 0.75 bar. The face pressures maintained after Stn, +450m to the end of construction varied greatly with most values falling between 0.25 bar and 0.75 bar.

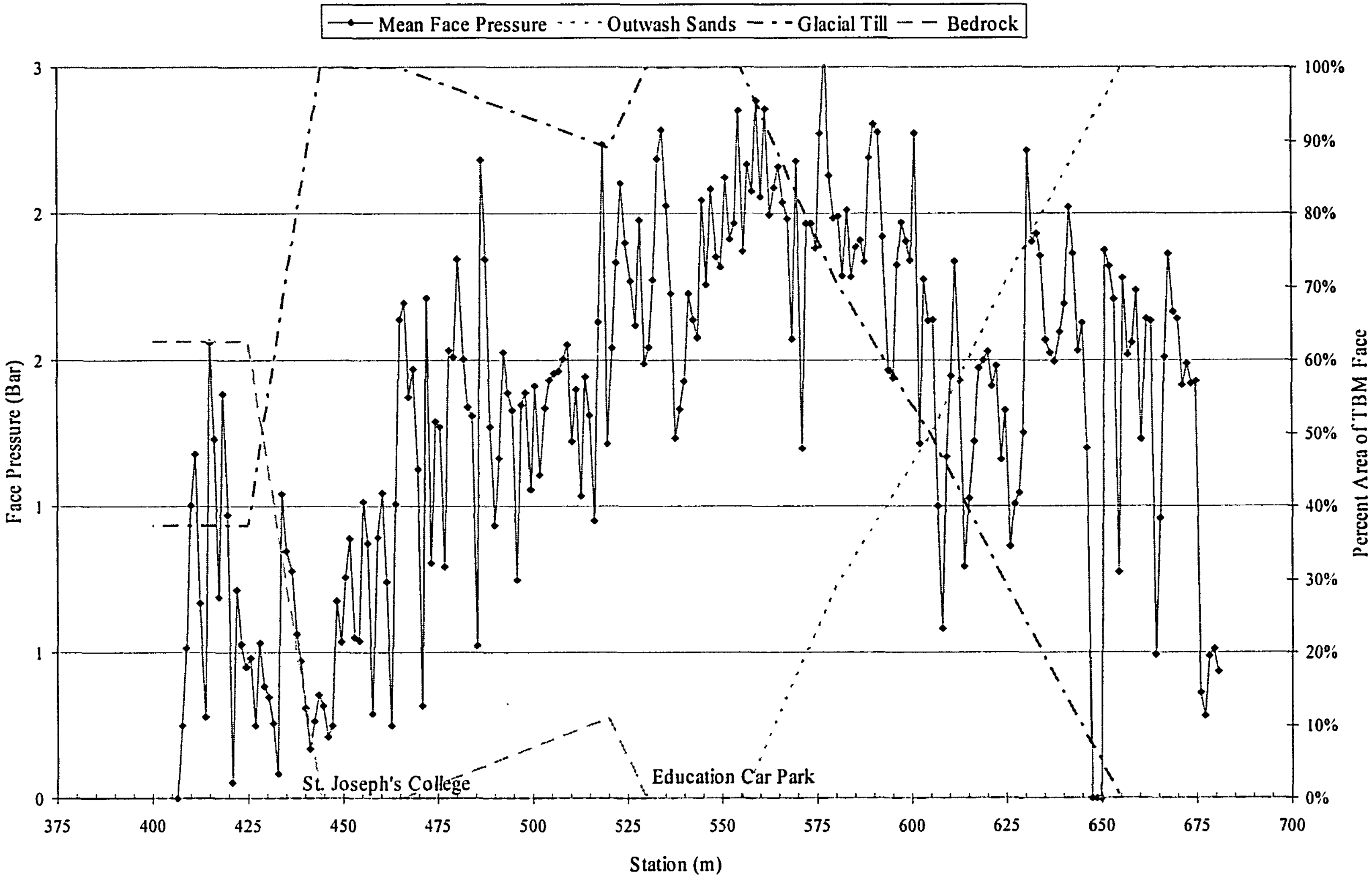


Figure 4-8 Face Pressure recorded during advance, Southbound Tunnel.

4.2.3 Excavation time for Southbound and Northbound construction.

Figure 4-9 (p 88) illustrates the time needed for the excavation of the soil for any given advance (1 ring = 1.2m) during the construction of the Southbound Tunnel. The information presented in this plot was obtained from the manual site logs, and references the start and finish times for each advance during construction. Also included in this Figure is the inferred geology as shown in Figure 3-1 (p 30). A moving average trend line with a spread of three was used to present a smoothed value for the time needed to excavate each ring. The spread refers to the number of samples to read ahead and behind of the current location to determine the average for any given position. Most apparent in this Figure are the numerous peaks observed over the length of construction. The observed peaks were in large part due to start-up technical issues with the TBM, the constraints of operating with a limited muck train for the initial burial of the TBM (first 60m of construction), and operational experience with the local geology. In attempting to gain a reasonable picture of typical excavation times the Northbound Tunnel offers a better baseline as the start-up technical issues, and operational experience issues present for the Southbound Tunnel were much reduced for the construction of the Northbound Tunnel.

Figure 4-10 (p 89) illustrates the excavation time per ring for the Northbound Tunnel. It also presents the mining times as a trend line using a moving average of three. In observing this plot, the distinct stratigraphic layers present are reflected in the mine times observed. Excavation within the Outwash Sands and Silts prior to Stn. +625m was in the order of 50 minutes per advance. As Glacial Till began to occupy a larger proportion of the face of the TBM, the mine times reduced from 50 minutes to an average of 20 minutes per ring between Stn. +600m and Stn. +450m. As Bedrock began to occupy a greater area of the face, the average mine times increased to some 45 minutes by Stn. +430m. In a region of intra-till sands between Stn. +430m and Stn. +415m, the average mine time dropped to 25 minutes. As construction of the Northbound Tunnel was not plagued by the same degree of technical and start-up issues as the Southbound Tunnel, the mine times recorded provide a more accurate picture of the performance of the TBM.

Figures 4-11 and 4-12 (p 90) present the progress of Southbound Tunnel construction in terms of rings constructed per day. Figure 4-11 is laid out by specific dates to show the progress on any specific date. Figure 4-12, by comparison, shows the same information as Figure 4-11 by days in production. The information for Northbound Tunnel construction can be found in Figures 4-13 and 4-14 (p 91) which detail the weeks and days-in-production respectively.

The Southbound and Northbound Tunnels were completed in 46 and 41 days respectively, with the Southbound Tunnel being some 28m, or 24 rings, longer than the Northbound Tunnel. Productivity early in construction was hindered by the need to run the muck train at a reduced size due to the limited worksite space which prevent a full muck train from being used. As a result, the full productivity of the TBM was not reached until the TBM had advanced sufficiently to allow the full assembly of the gantry, and the operation of a full train. Maximum productivity in the Southbound Tunnel was between 6 and 8 rings per day and was maintained between Stn. +600m and +500m. After Stn. +500m, until the end of construction, daily productivity varied between 5 and 7 rings per day. During construction of the Northbound Tunnel, maximum daily production was reached after Stn. +590m, and varied between 7 and 9 rings per day. This rate of construction was maintained for the rest of the construction of the tunnel save for the final 40m, where productivity was reduced to between 6 and 7 rings per day.

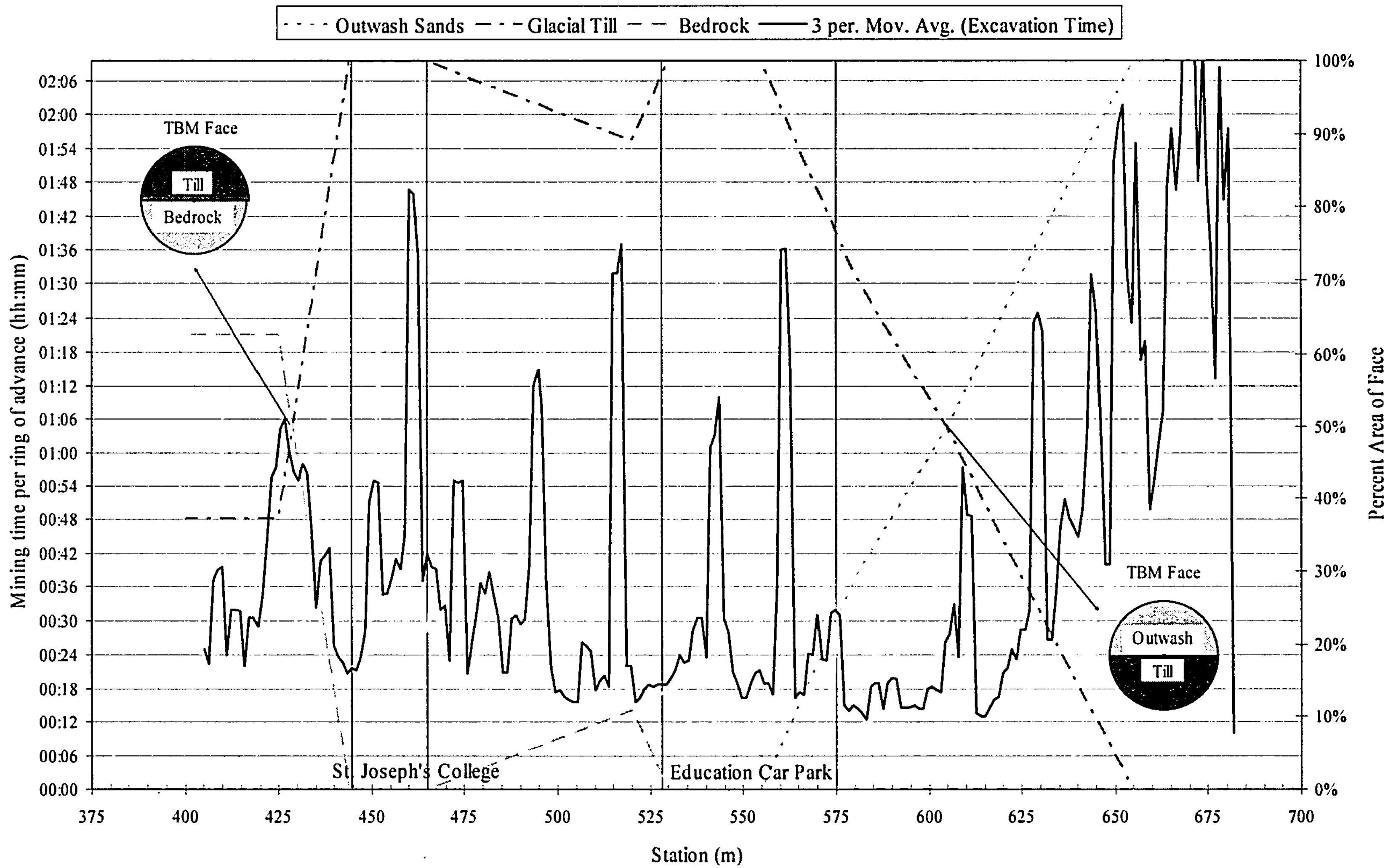


Figure 4-9 Time needed to excavate one ring during construction of Southbound LRT Tunnel.

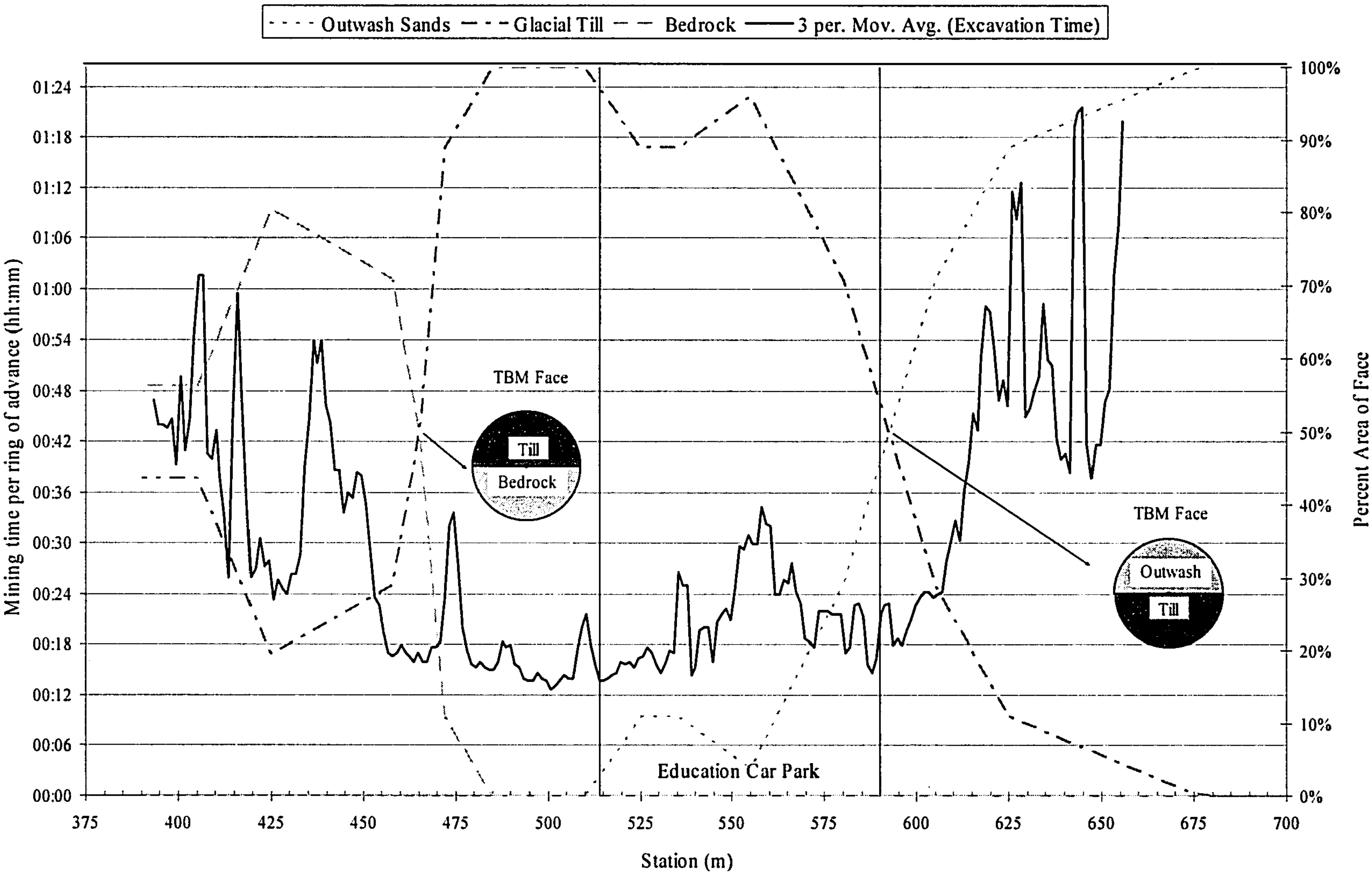


Figure 4-10 Time needed to excavate one ring during construction of Northbound LRT Tunnel.

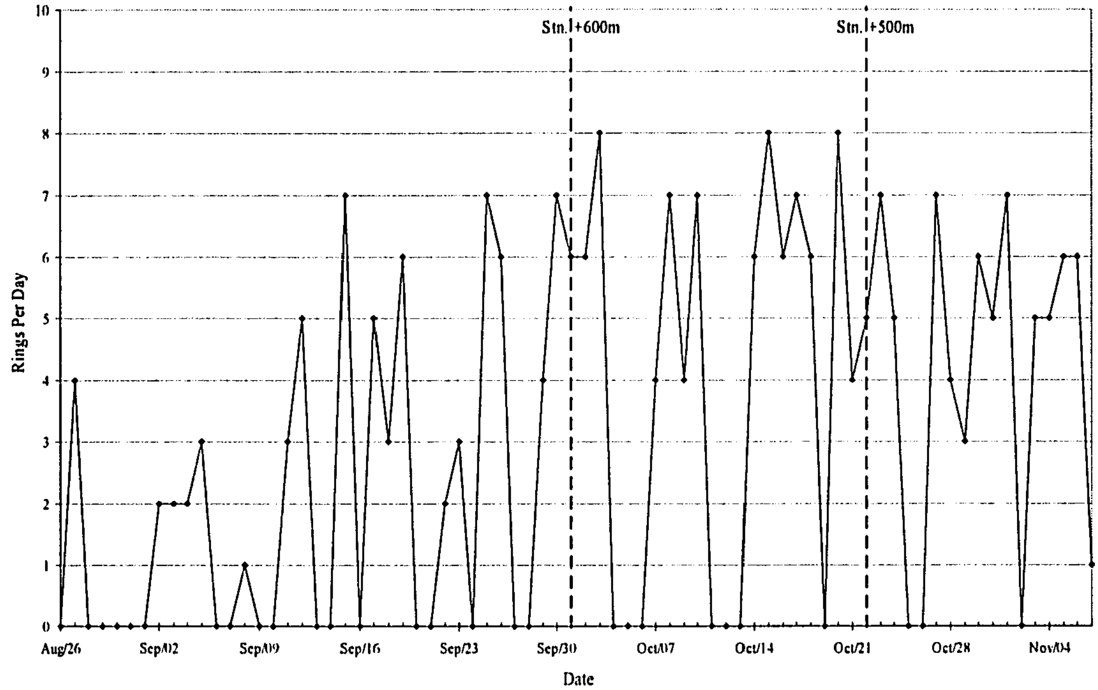


Figure 4-11 Rings built per day during construction of Southbound LRT Tunnel (2004).

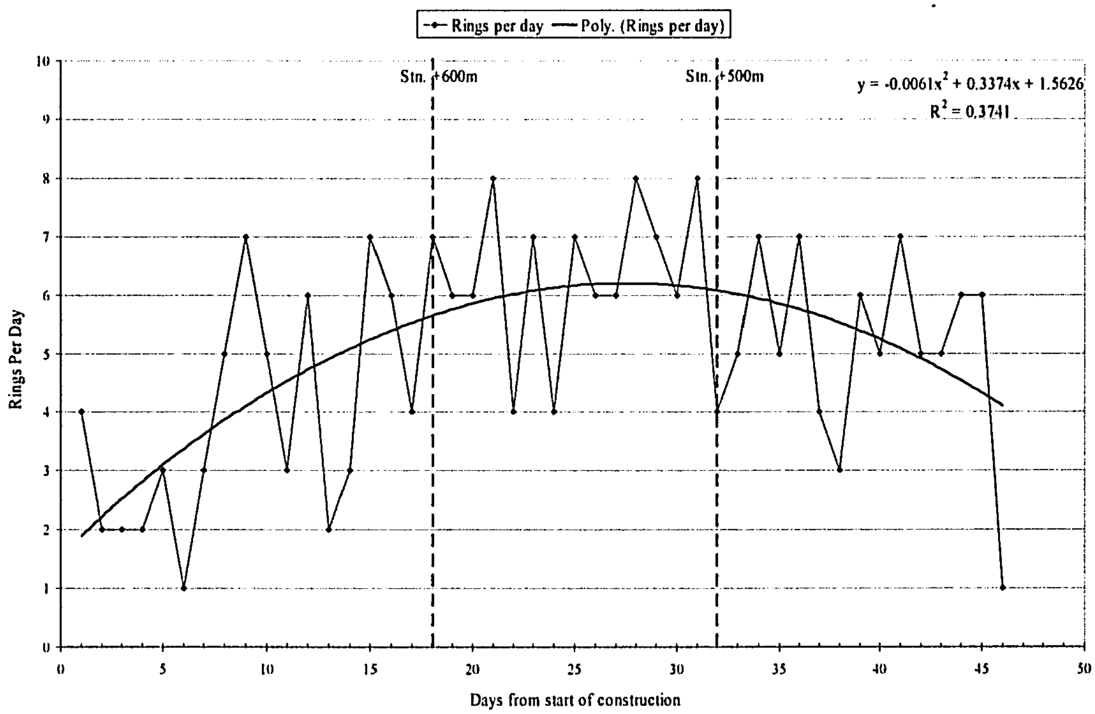


Figure 4-12 Rings built per day during construction of Southbound LRT Tunnel (2004).

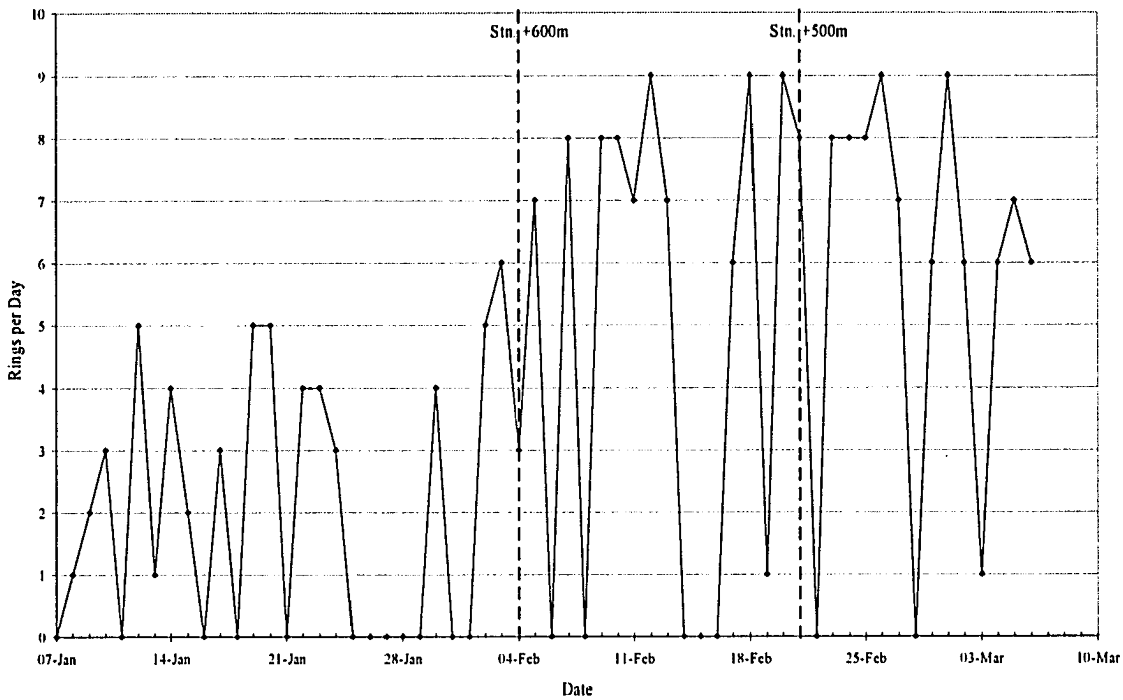


Figure 4-13 Rings built per day during construction of Northbound LRT Tunnel (2005).

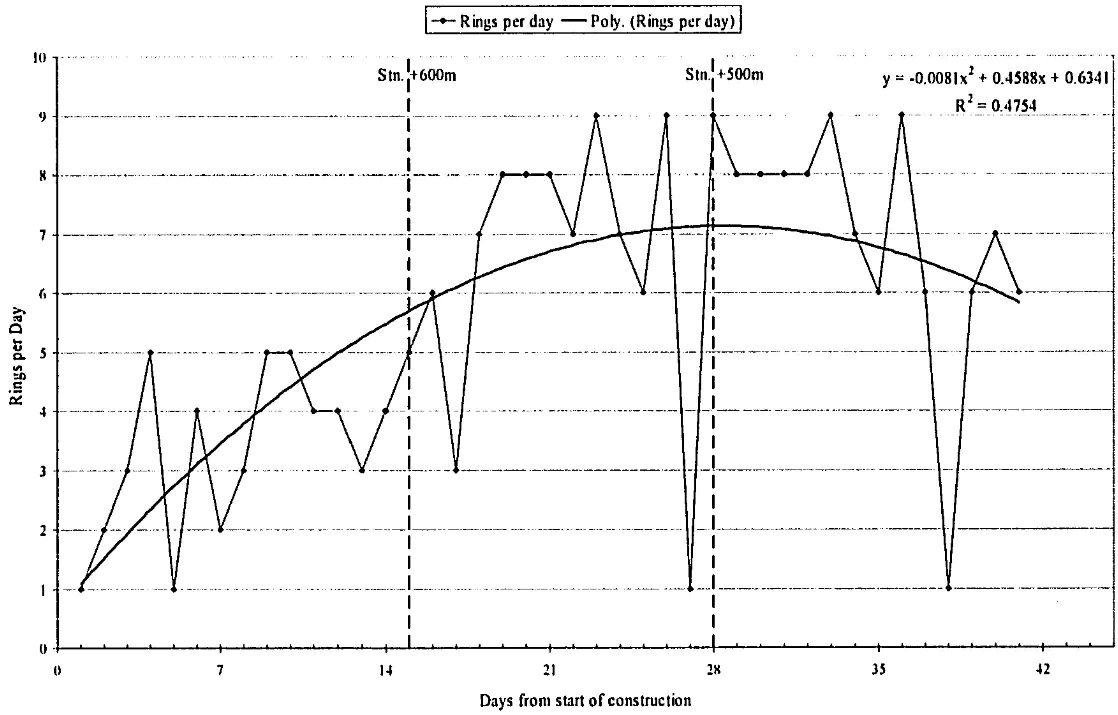


Figure 4-14 Rings built per day during construction of Northbound LRT Tunnel (2005).

4.2.4 Volumes of injected grout for Southbound and Northbound construction.

Figures 4-15 (p 93) and 4-16 (p 94) present the volumes of injected grout per ring of advance for the Southbound and Northbound Tunnel construction respectively. The grout injection volumes were obtained from the manual site logs, which were based on measurements of the train car that the grout was pumped from. Figures 4-17 and 4-18 (p 95) present the grout injection pressures used during the construction of the Southbound and Northbound Tunnels respectively.

The theoretical grout volume required, based on the difference between the over-cut diameter and that of the assembled segmental lining was approximately 3.2m^3. At no point during construction was the grout volume injected at or near this level when concerns over settlement were present.

During the construction of the Southbound Tunnel, prior to passing under the Education Car Park at Stn. +575m, the injected grout volume per ring of advance (1.2m) was in the order of 3.25m^3. The injected grout pressure in this region increased from 30 to 40 PSI at Stn. +625m, to between 40 and 45 PSI at Stn. +575m. Construction after Stn. +575m showed an increase in the volume of grout injected, with the average volume per ring in the order of 4.2m^3. This level of grout injection was maintained for the rest of construction, decreasing to 4.0m^3 after Stn. +435m. The injected grout pressure remained in the order of 40 to 45 PSI from Stn. +575m to the end of construction.

During construction of the Northbound Tunnel, the average injected grout volume per ring was maintained in the region of 4.3m^3. This level of grout injection was maintained from the time the TBM was sufficiently buried to allow for grouting to Stn. +415m, where the injected volume reduced to 3.25m^3 per ring. The initial grout injection pressures from the start of grouting to Stn. +615m, were between 40 to 50 PSI. After Stn. +615m to Stn. +450m, the injection pressure was maintained between 35 and 45 PSI. After Stn. +450m, the injection pressure fell from this average of 35 to 45 PSI to approximately 35 PSI by the end of tunnel construction.

The grouting procedures specified for the Edmonton SLRT can be found in Appendix F – Grouting Procedures for Edmonton SLRT Construction (p 202).

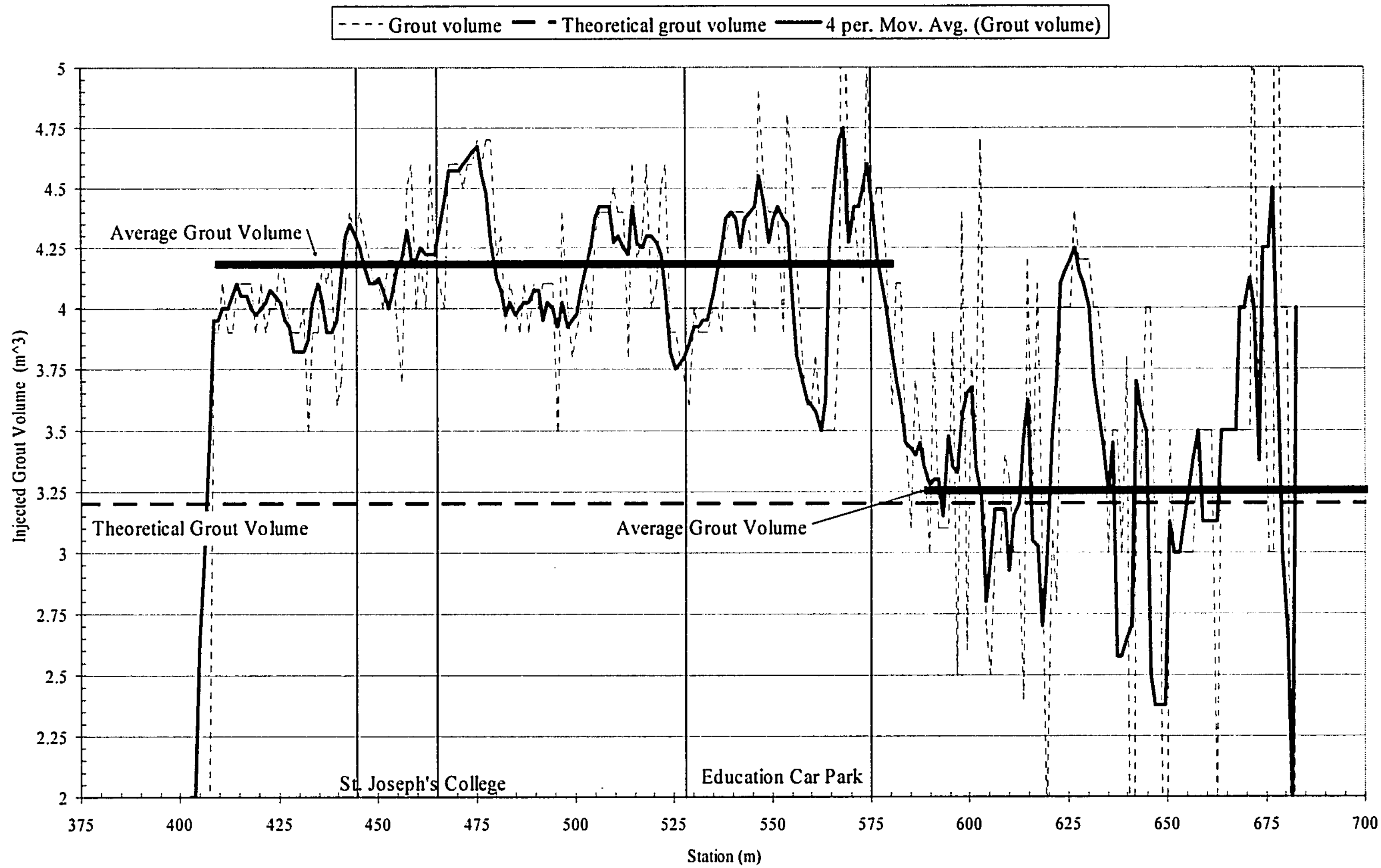


Figure 4-15 Injected grout volume per ring of excavation during construction of Southbound LRT Tunnel.

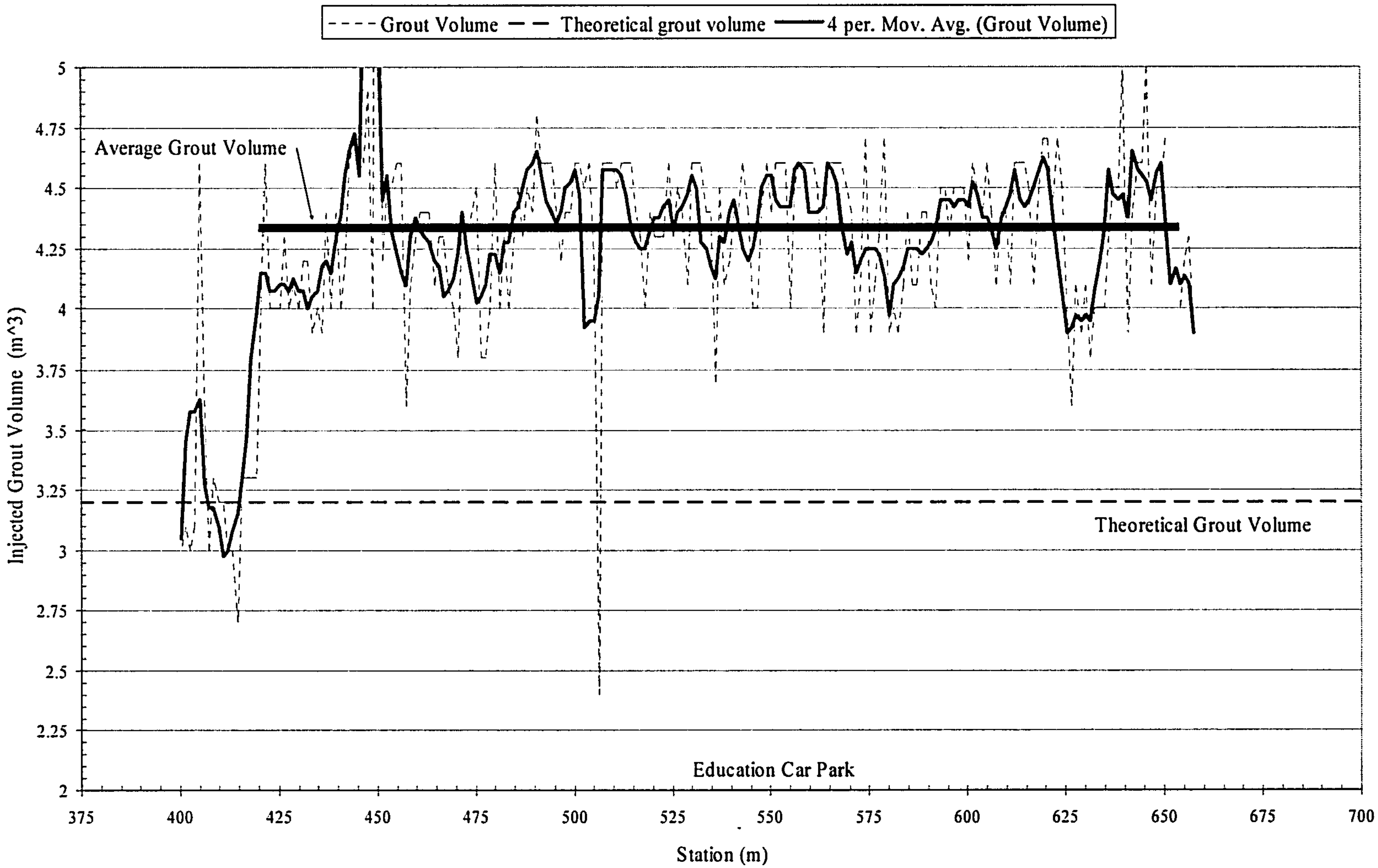


Figure 4-16 Injected grout volume per ring of excavation during construction of Northbound LRT Tunnel.

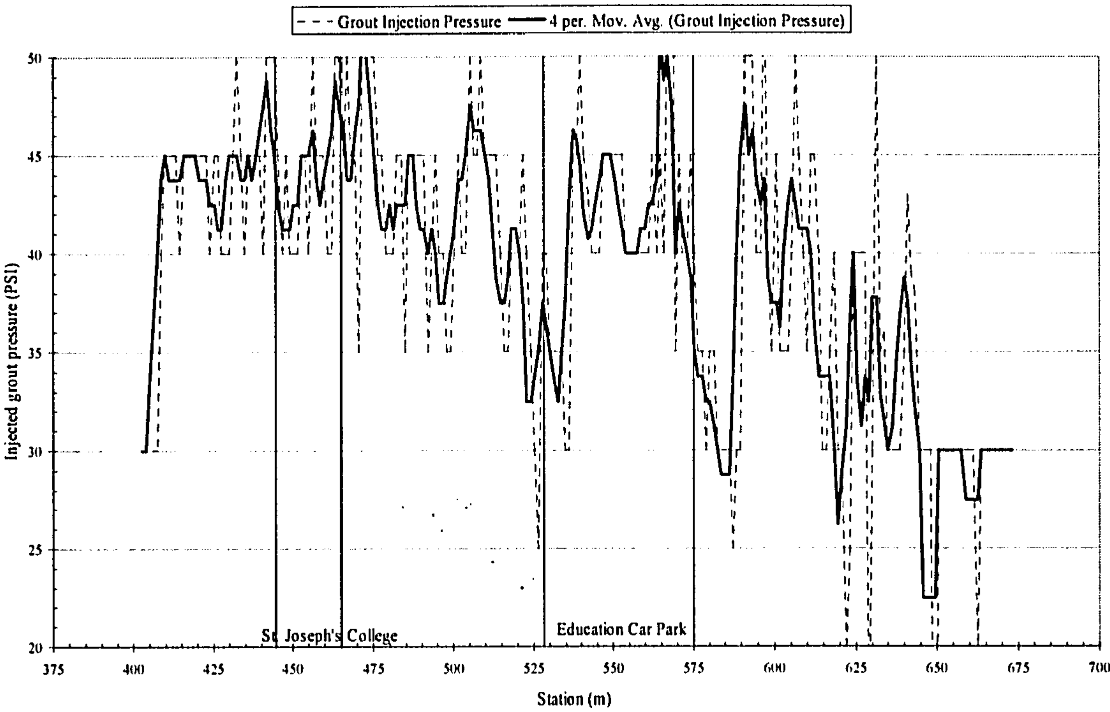


Figure 4-17 Injected grout pressure during Southbound construction.

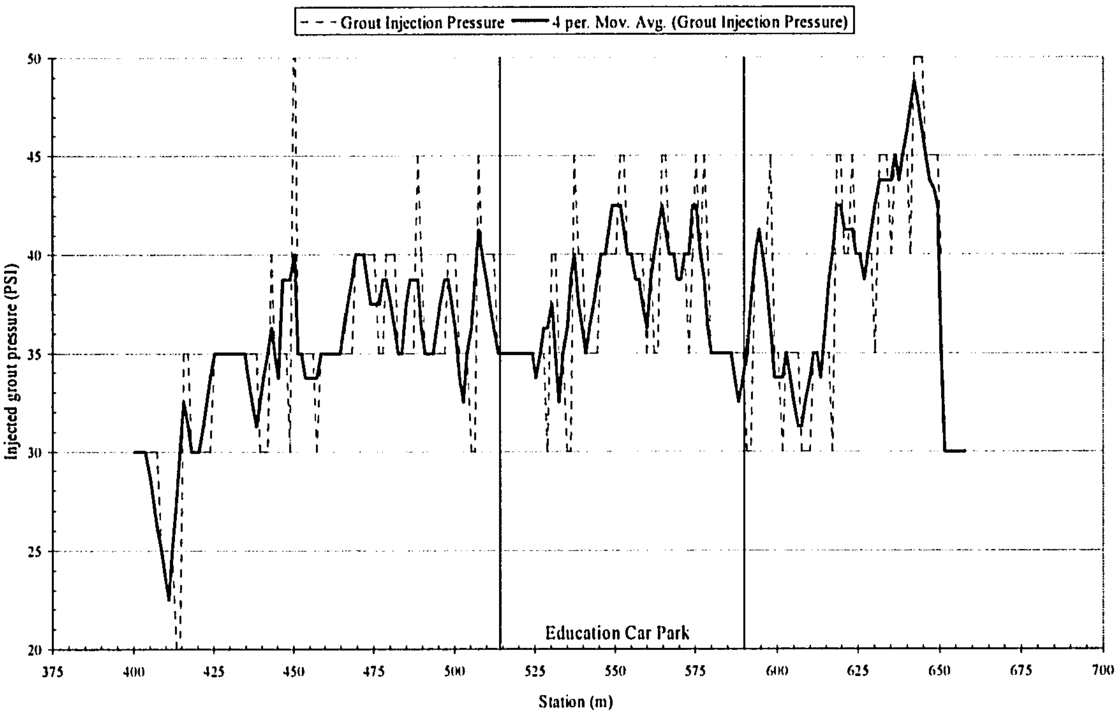


Figure 4-18 Injected grout pressure during Northbound construction.

4.2.5 Bulked excavation volumes.

Figures 4-19 (p 98) and 4-20 (p 99) present the muck skips used for the excavation of each ring for the construction of the Southbound and Northbound Tunnels respectively. Excavation within the Outwash Sands and Silts demanded on average 4.5 to 5 muck cars for the excavation of the Southbound Tunnel, and 5 cars for the Northbound. Within the region of Glacial Till, construction for both the North and Southbound Tunnels required 5 cars on average. However, as the bedrock content of the face of the TBM increased, the cars required for the construction of the North and Southbound Tunnels increased to between 6 and 7 cars.

With a cut diameter of 6.564m, the in-situ volume excavated per ring (1.2m) of advance was 40.6 m^3. The muck cars in service on the train possessed a volume of 8 m^3; as such one ring of advance would theoretically require 5 muck cars. Construction of the Northbound and Southbound Tunnels required between 4 and 5 muck cars per ring of advance within the Outwash Sands and Silts. This value indicates a shrinkage factor between 1.0 and 0.9 for the wetted sand and silt. In the case of construction at near full face conditions within the Glacial Till, Southbound construction produced in the order of 6 cars per ring of construction, with the Northbound Tunnel producing between 5 and 6 cars. This variation between tunnels resulted from the greater content of intra-till sands encountered during the construction of the Northbound Tunnel. These muck volumes correspond to a bulking factor between 1.2 and 1.05. This range of values is lower than that of generalised clay which is 1.3 to 1.4. This is because the generalised value anticipates dry excavation using a backhoe or similar equipment. The soil is then dry dumped into some container for removal. In the case of tunnelling within the Glacial Till the presence of significant quantities of intra-till sands, and the mixing and conditioning of the spoil during the mining process would serve to reduce the observed bulking factor from the generalised value as the voids present in the movement of dry soil blocks are not present..

At no point did construction take place full face within the Bedrock. As a result, the bulking factors observed for the latter part of construction (after Stn. +475m Southbound, Stn. +500m Northbound) are valid for mixed face condition of Glacial Till and Bedrock. Operating in these mixed face conditions, construction of both tunnels produced between 6 and 7 muck cars per ring of advance. After Stn +425m, during Northbound construction, the number of muck cars increased to between 7 and 8. This region of Northbound construction also produced the highest fines content in the spoil suggesting a much greater fraction of bedrock at the tunnel face. These muck volumes correspond to a bulking factor of between 1.2 and 1.4 for tunnelling with mixed face conditions of Glacial Till and Bedrock with Glacial Till being the predominant material at the face. With Bedrock the predominant material at the face, the observed bulking factor increased to between 1.4 and 1.6. The bulking factor obtained approximates the anticipated dry bulking factor of between 1.3 and 1.4 for clay and 1.5 for shale. In this case of mixed face within the Bedrock and Glacial Till, the increase in bulking factor is largely a function of Bedrock content. Glacial Till would be effectively broken down during the mining process. In contrast, Bedrock would not be fully broken down by the mining process and the spoil would be a mixture of water, Glacial Till and Bedrock paste with intact cobbles of Bedrock.

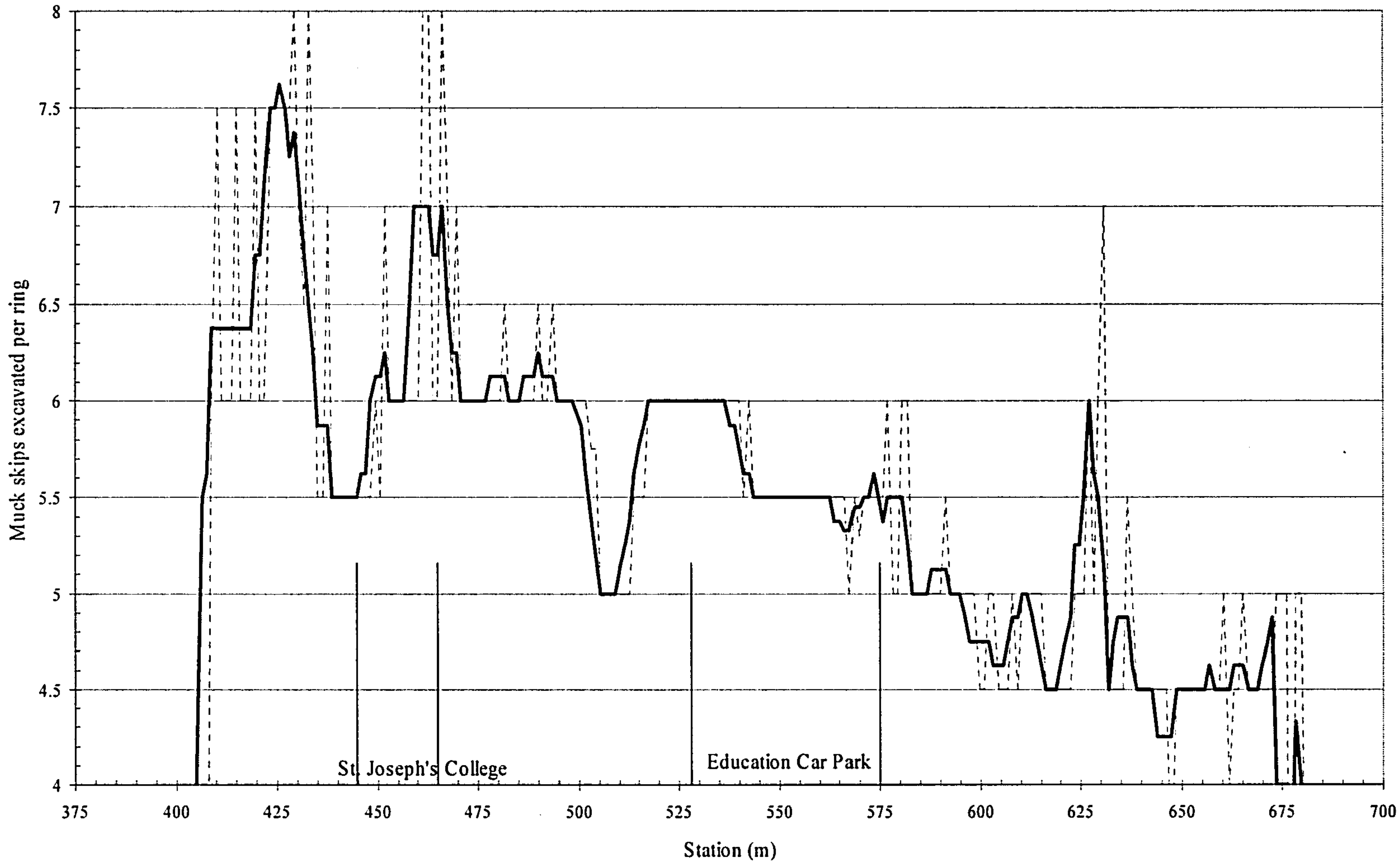


Figure 4-19 Muck cars needed per ring of excavation during construction of Southbound LRT Tunnel.

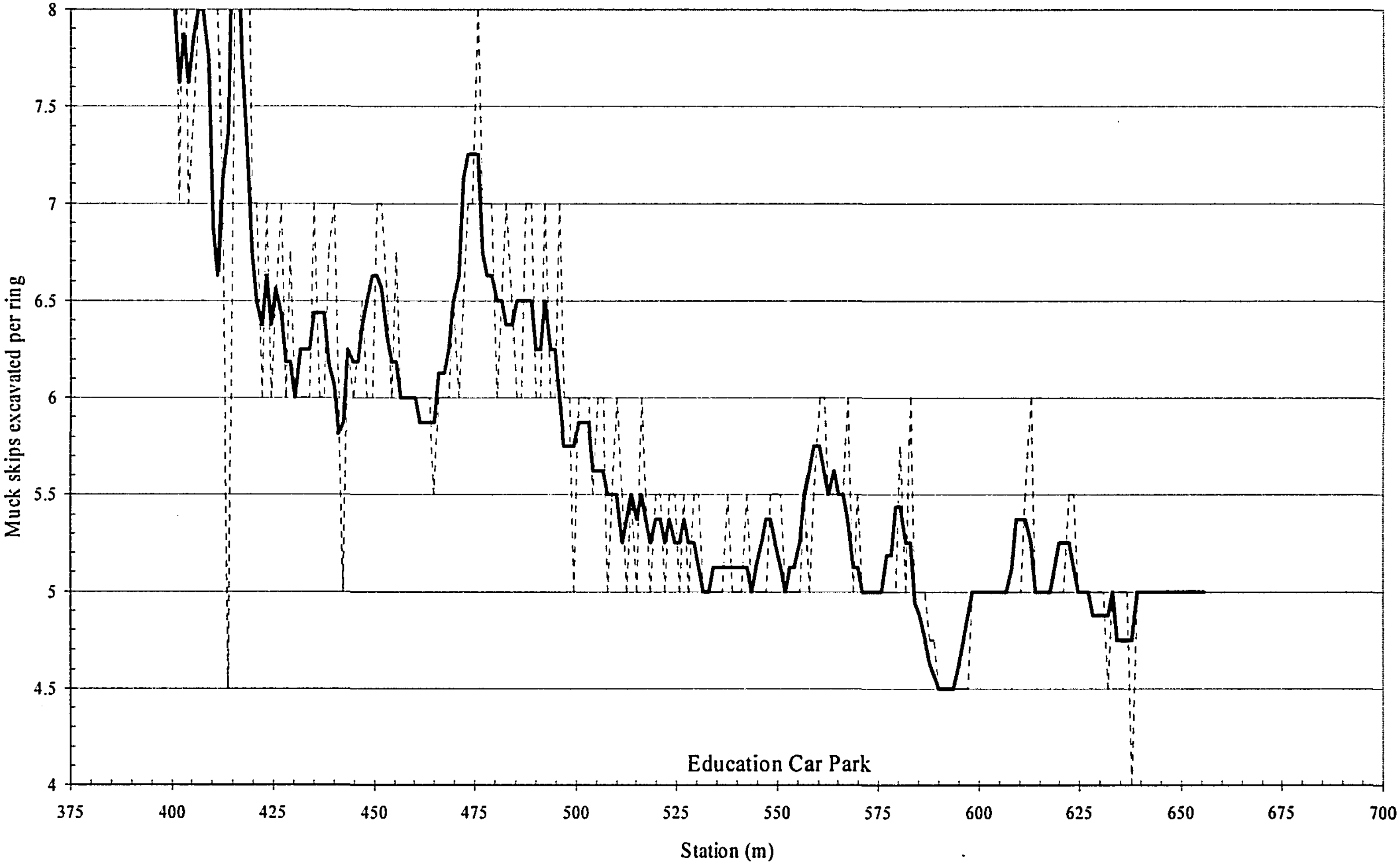


Figure 4-20 Muck cars needed per ring of excavation during construction of Northbound LRT Tunnel.

4.3 Settlement Data

Section 2.1 (p 15) discussed the control of ground movement associated with EPB-TBMs. In that discussion, two elements stood out as those most easily controlled by the operator: ensuring adequate face pressure to reduce ground loss, and ensuring adequate grouting to fill the void between the excavation diameter and the diameter of the installed segmental lining. This Section will present the settlements observed during the construction of the Southbound and Northbound SLRT Tunnels.

The observed deep settlements will be discussed within this section. However the Figures detailing the deep settlement discussion can be found in 'Appendix G – Deep Proximity, and Time, Settlements' (p 208) and 'Appendix H – Deep Array Settlement' (p 219). In the discussion, the Figures in Appendix G will be referenced directly in this Section and abbreviated as Figure G-#.

A series of six Figures detail the location of the instrumentation used to monitor the tunnelling induced settlement. Figure 2-21 presents the Edmonton SLRT Site Plan as presented in AMEC's site investigation. Figure 4-22 (p 104) details the instrumentation from the portal to the approaches to the Education Car Park. Figure 4-23 (p 105) details the instrumentation present in the Education Car Park and towards St. Joseph's College. Figure 4-24 (p 106) details the instrumentation in the region of St. Joseph's College to the end of the tunnel alignment. Figure 4-25 (p 107) details the survey points affixed to the structure of the Education Car Park. Figure 4-26 (p 108) details the survey points affixed to the Utility Corridor, located on the approaches to the Education Car Park.

The primary source of information presented in this Section is the deep and shallow settlement points installed along the alignment. Secondary instrumentation was present in the form of survey points affixed to the Education Car Park and St. Joseph's College. These survey points monitored the movement of the structures in response to tunnelling induced settlements.

The deep and shallow settlement gauges were installed along the Edmonton SLRT alignment during June and July of 2003 by Mobile Augers. The settlement gauges were installed in boreholes drilled using hollow or solid stem augurs. The shafts for the instruments were drilled such that the base of the shaft was located some 1.5m above the tunnel crown in the case of deep settlement points, or 1.5m from the surface for shallow settlement points. Once this base had been reached the settlement gauges were assembled and secured in place.

The settlement points were constructed from sections of threaded steel piping and end sections which possessed a steel spike. In the case of deep settlement points, the end section would be lowered partway into the shaft, and a section of threaded steel piping would then be screwed onto the end of the base section. This process would be repeated with new pipe sections added to the instrument, until the instrument had been lengthened sufficiently to reach the base of the shaft. The settlement gauge would then be vibrated from the surface with a jackhammer to drive the spike on the base end into the ground. A volume of grout was then poured into the shaft to bury the base of the shaft, and secure the base of the settlement gauge, under several feet of grout.

The repeatability of the settlement data obtained from the instrumentation during construction of the SLRT varied. The shallow settlement points and building instrumentation produced consistent results over the course of construction with repeatability in the order of 0.1mm. The deep settlement points produced a similar level of repeatability as the shallow settlement points were in the order of 0.1mm. However, the reliability of the deep settlement values, as an indicator of surface settlements, was less than that achieved with the shallow settlement points. This lower reliability was a reflection of the purpose of the deep settlement points: to provide an early warning of face loss during tunnelling.

The majority of deep and shallow settlement gauges installed were located along the centrelines of the tunnels. However, to monitor the settlement trough associated with constructing the two tunnels, a series of four arrays were constructed perpendicular to the alignment. Array D was located in the approaches to the Education Car Park, Arrays C and B within the Education Car Park, and Array A located on the approaches to St. Joseph's College.

The settlement data obtained during construction are presented in a series of plots arranged by their location along the alignment. Each line present on the plot represents the settlement profile of an individual gauge and the Stn. of each gauge is noted in the legend. Two Figures were constructed for the instruments in each region. The first is based on the proximity of the instrument to the face of the TBM. The second is based on time dependent settlement over the time needed to construct the Southbound and Northbound Tunnels.

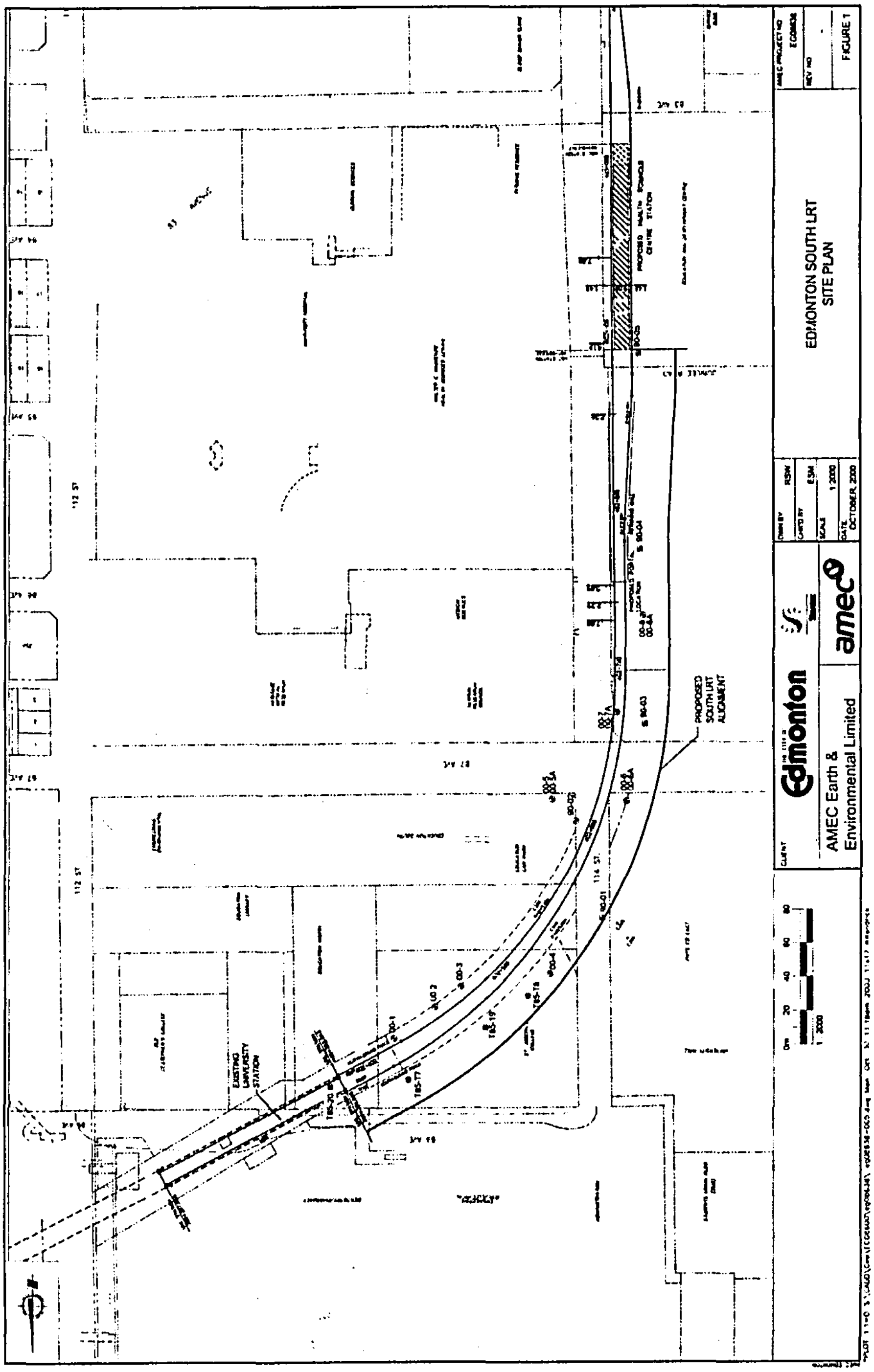


Figure 4-21 Edmonton SLRT Site Plan (AMEC)

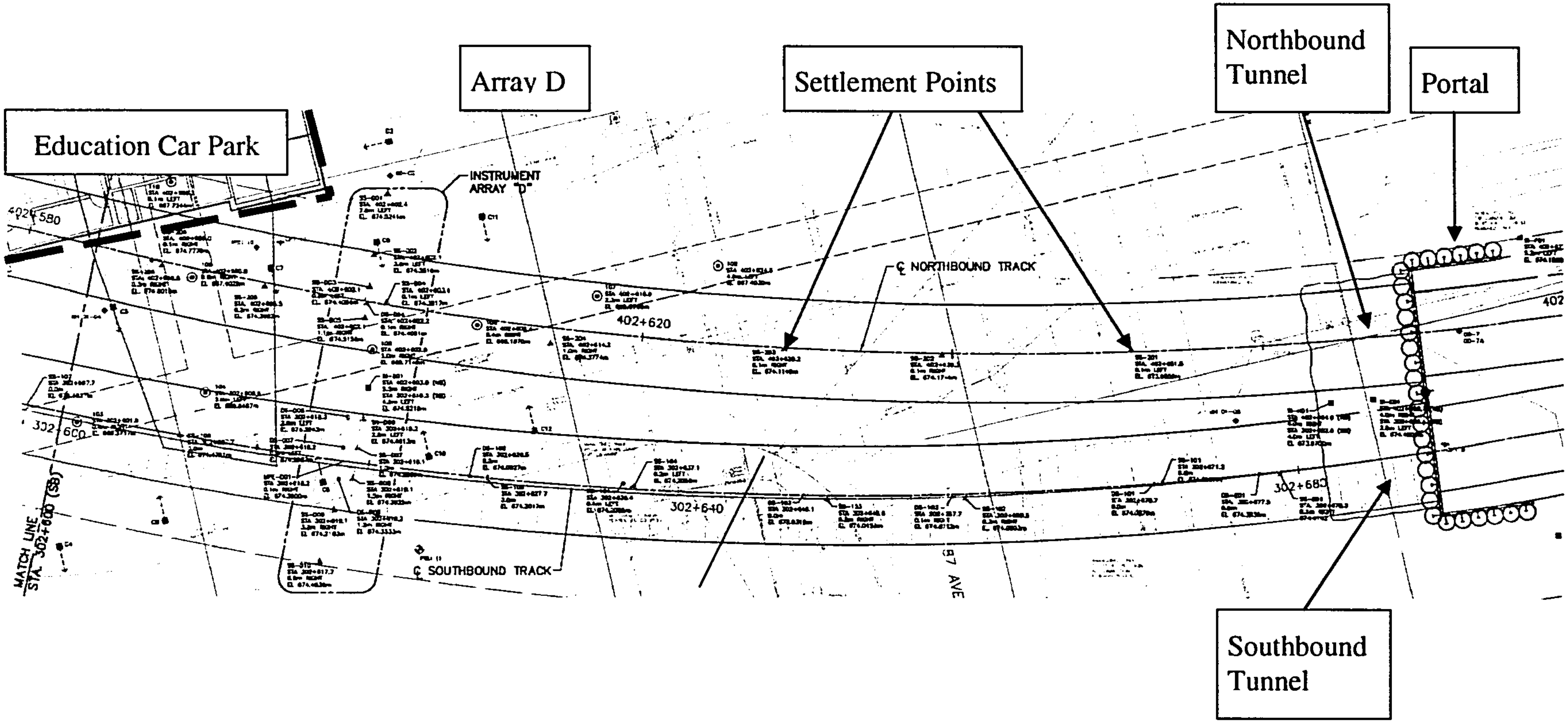


Figure 4-22 Instrumentation layout from portal to Education Car Park.

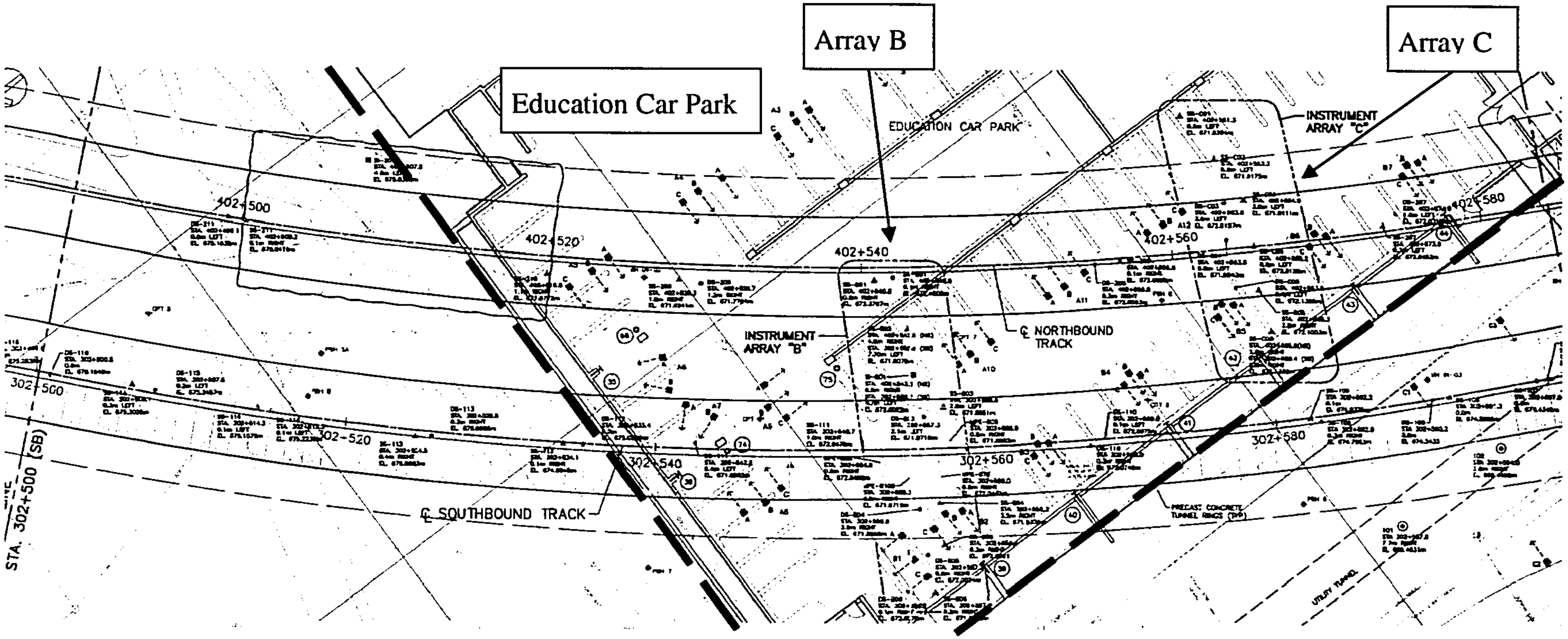


Figure 4-23 Instrumentation layout in Education Car Park.

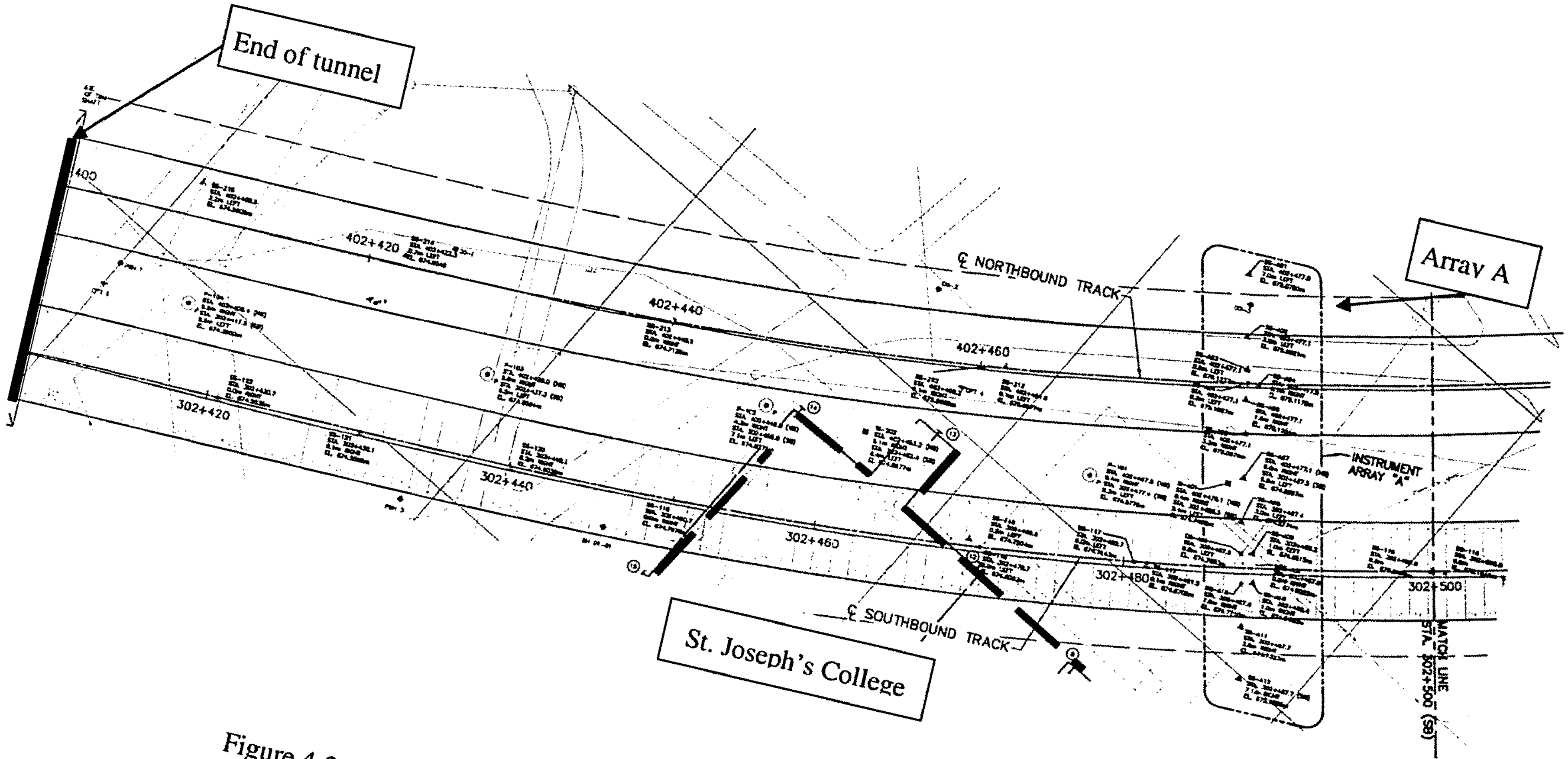


Figure 4-24 Instrumentation layout from St. Joseph's College to breakthrough.

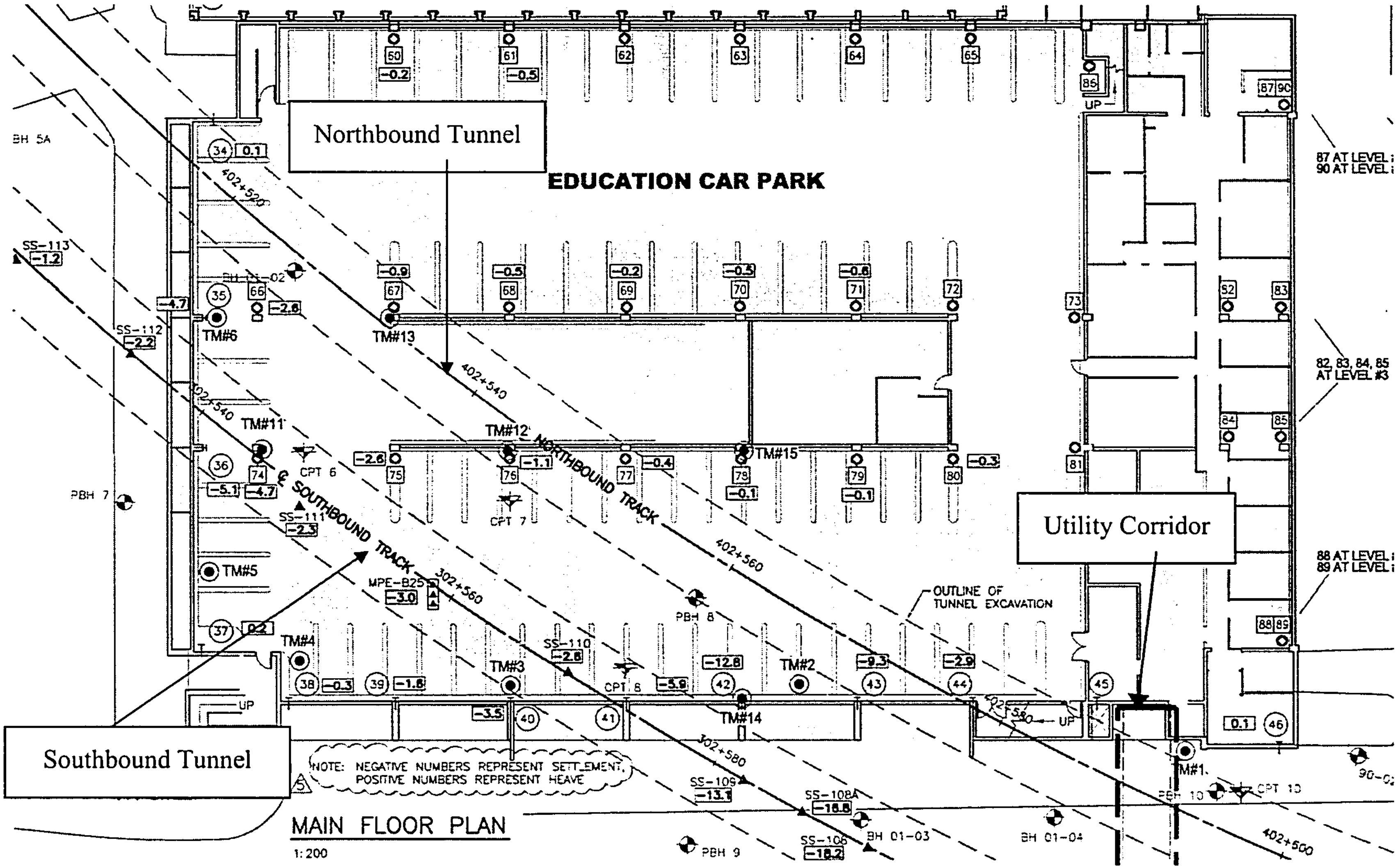


Figure 4-25 Layout of Education Car Park Survey Points

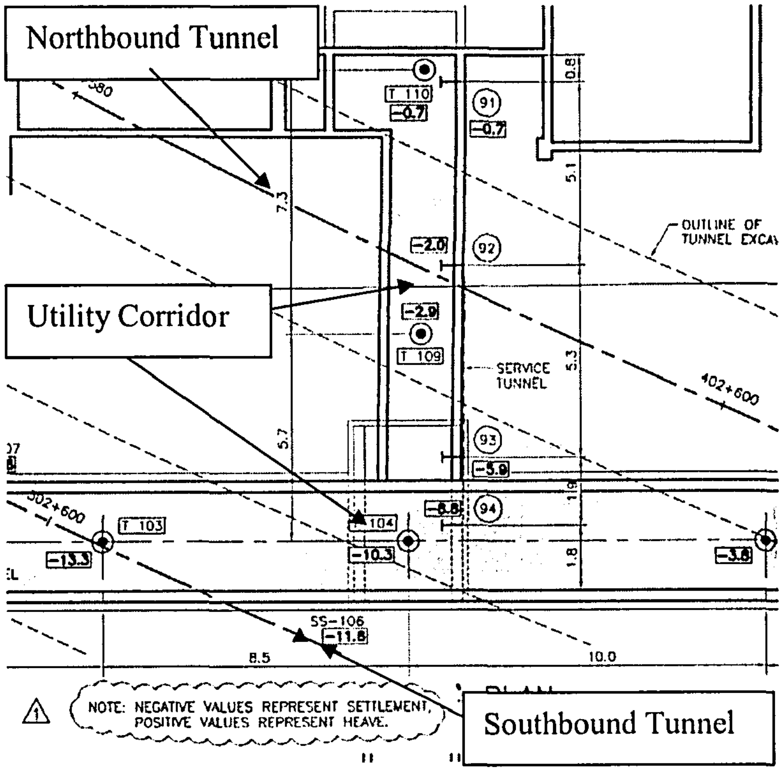


Figure 4-26 Layout of utility corridor and the location of survey points. Located just prior to passing under the Education Car Park.

4.3.1 Observed Southbound Settlements

Figure 4-27 (p 111) shows the observed shallow settlements in the region from the portal to the Education Car Park for the construction of the Southbound Tunnel. Figure G-1 (p 209) shows the deep settlements for this same region. The observed shallow settlements in this region were between -11mm and -18.6mm. Observed deep settlements in this region were between -15mm and -25.9mm.

In viewing Figure 4-27, it is useful to consider Figure 2-4 (p 13), which represents a generic EPB-TBM settlement curve. Figures 4-27 and G-1 show little settlement ahead of the face of the TBM, suggesting that excellent face control was being maintained. However, considerable settlement is observed over the length of the shield of the TBM, with additional settlement occurring at the transition from the tail shield to the segmental lining. This settlement pattern suggests that the grouting methodology employed was not effective in eliminating surface settlements. This settlement can be observed in profile in Figure 4-41 (p 132) which shows the settlements observed at Array D. Like Figures 4-27 and H-1 (p 220), the majority of the settlement observed at Array D (-25.3mm) occurs along the tail shield of the TBM at the transition from tail shield to segmental lining.

Figure 4-28 (p 112) shows the observed shallow settlement response during tunnel construction under the Education Car Park. This figure shows dramatically reduced settlements compared to the region prior to reaching the Education Car Park. The maximum observed shallow settlement in this region was -4.4mm. This response is also repeated in Figure G-2 (p 210) with the maximum observed deep settlement being -6.6mm. A lateral view of this settlement can be observed in Array C (Figure 4-43, p 134) and Array B (Figure 4-45, p 136). Array C shows a maximum shallow settlement of – 4.7mm. It should be noted that this array is centered over the Northbound Tunnel and the settlement point being reported is 5m off the centerline of the Southbound Tunnel. Array B, which is centered over the Southbound Tunnel, reported a maximum shallow settlement of -1.7mm.

Figure 4-29 (p 113) shows the observed shallow settlements in the region between the Education Car Park and St. Joseph's College. The observed settlement in this region did not exceed -5mm. One gauge (S114) did show -6.9mm of settlement; however this settlement had been observed prior to the arrival of the TBM and as such may be the result of a misreading of initial elevation or movement after placement. The observed deep settlements (Figure G-3, p 211) in this region were less than -4mm with two exceptions. Instrument DS116 reported a settlement of -7.4mm, and instrument DS115 a heave of +10mm. In both cases it is likely that the instruments in question were founded closer than the 1.5m from the crown specified. Array A (Figure 4-47, p 138) located prior to St. Joseph's College, reported a maximum shallow settlement of -4.3mm.

No deep settlement points or arrays were located in the region from St. Joseph's College to final breakthrough. The maximum observed shallow settlement in this region was

-6.7mm (Figure 4-30, p 114). However, the instrument reporting this settlement and the instrument immediately adjacent (-6.6mm), were the final two instruments along the alignment. At this stage of construction no structures were being undermined and tunnelling took place within the competent Bedrock and Glacial Till. Face pressures and injected grout volume were also reduced in the final 30 meters of construction. The response to this can be observed in the slightly larger settlements despite the competent ground. All other shallow settlement gauges reported settlement of less than -4.0mm.

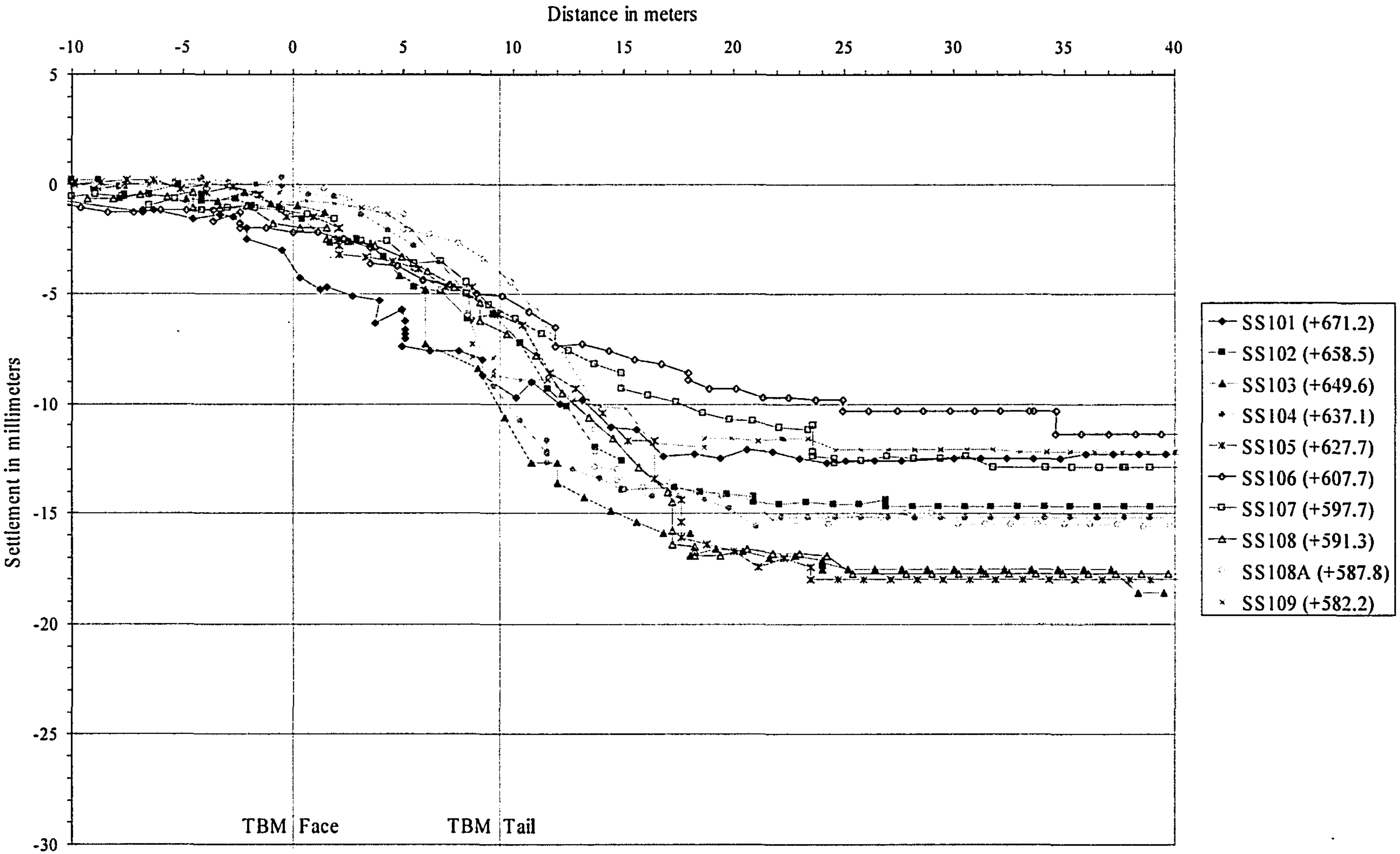


Figure 4-27 Observed shallow settlements during construction of Southbound Tunnel with proximity to TBM face, from Portal to Education Car Park.

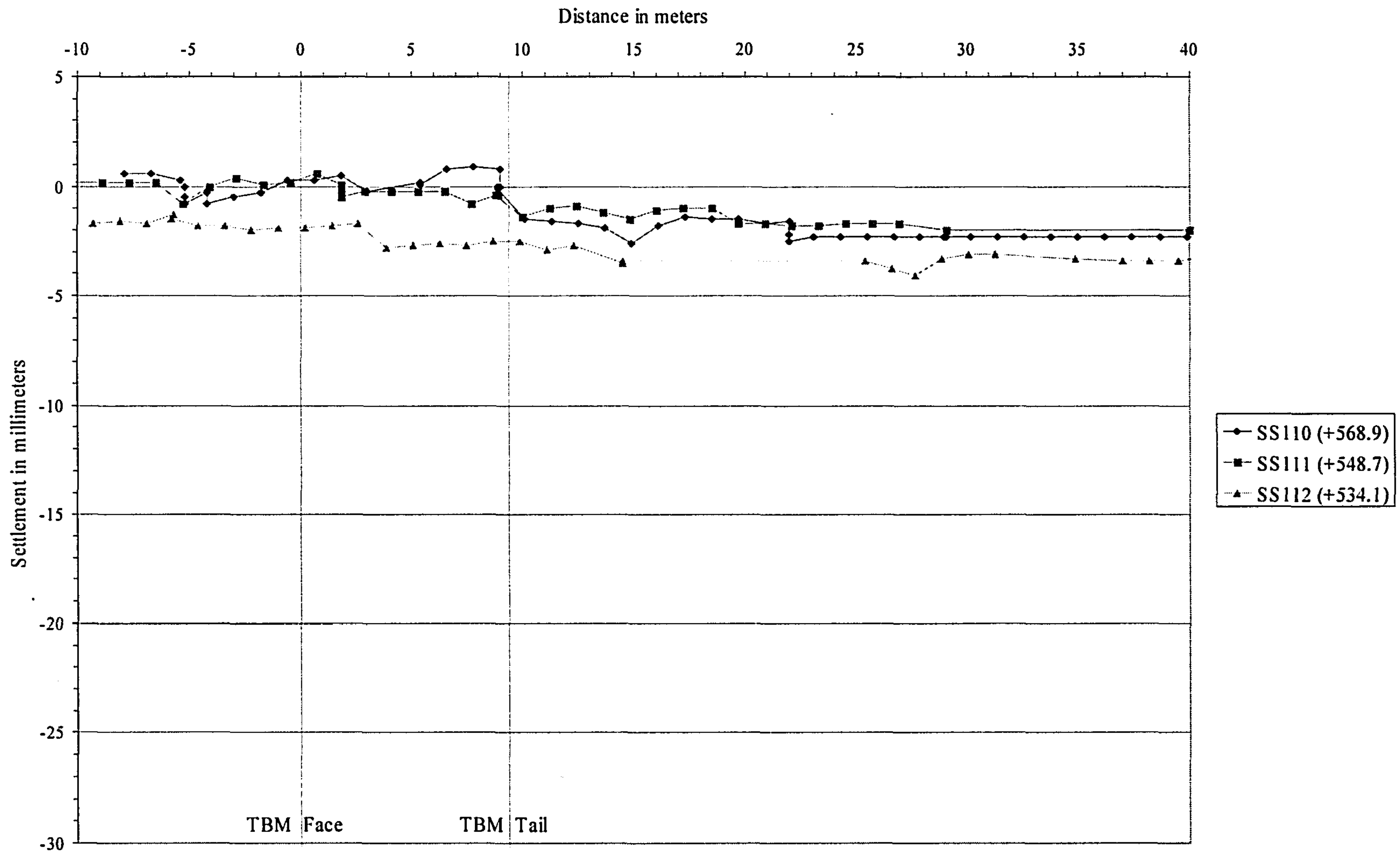


Figure 4-28 Observed shallow settlements during construction of Southbound Tunnel with proximity to TBM face, Education Car Park.

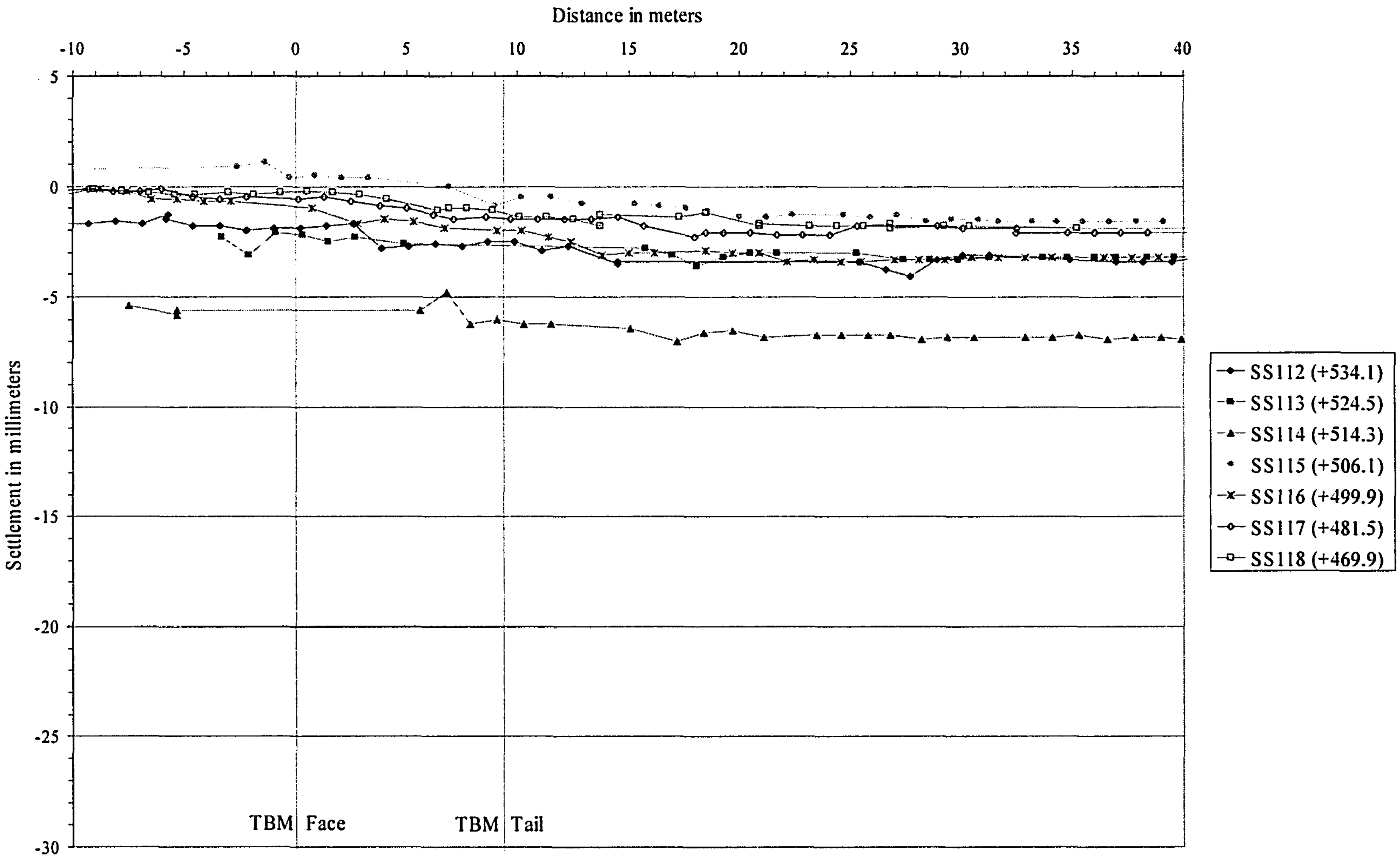


Figure 4-29 Observed shallow settlements during construction of Southbound Tunnel with proximity to TBM face, Education Car Park to St. Joseph's College.

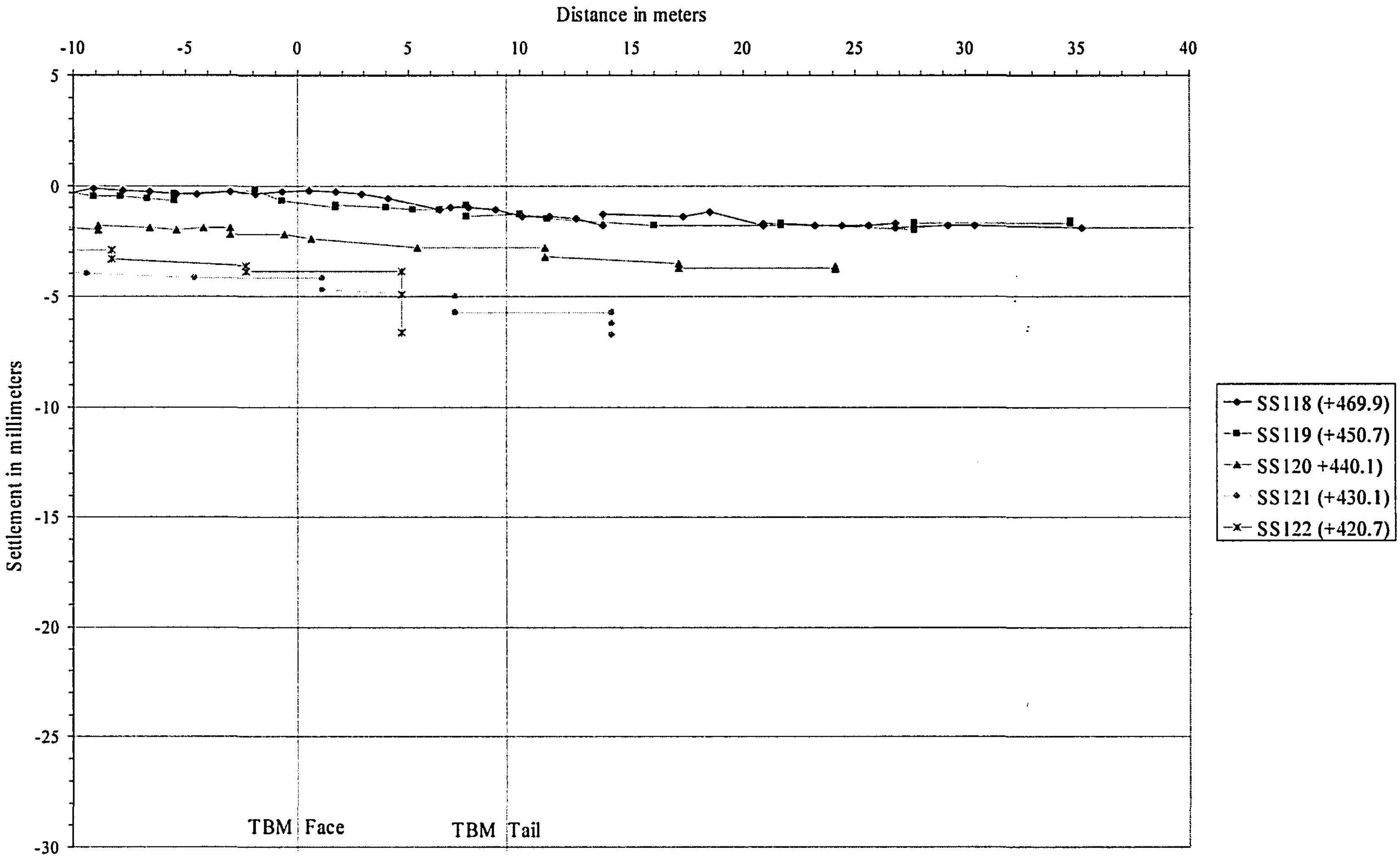


Figure 4-30 Observed shallow settlements during construction of Southbound Tunnel with proximity to TBM face, St. Joseph's College to breakthrough.

4.3.2 Observed Northbound Settlements

Figure 4-31 (p 117) presents the observed shallow settlements from the start of construction at the portal to the Education Car Park. The observed settlements in the three initial instruments (SS201, 202, 203) were between -19.8 and -21.6mm. The next three instruments (SS 204, 205, 206) reported settlements between -8.8 and -11.7mm. Figure G-4 (p 212) which presents the deep settlement data in this region, recorded settlements of -4.8 and -6.7mm with one instrument showing an ultimate heave of +3.7mm. The development of the settlement trough at Array D, with the construction of the Northbound Tunnel can be seen in Figure 4-42 (p 133). The maximum shallow settlement at a point located over the crown of the Northbound Tunnel increased from -2.6mm to -12.9mm with the construction of the Northbound Tunnel. The maximum shallow settlement over the centerline of the Southbound Tunnel increased from -25.3mm to -36.1mm with the construction of the Northbound Tunnel.

Figure 4-32 (p 118) presents the observed shallow settlements during construction beneath the Education Car Park. The maximum observed shallow settlement was -7.7mm with all other settlements being less than -4.3mm. Two arrays were present in this region, Array C (Figure 4-44, p 135) and Array B (Figure 4-46, p 137). Array C recorded a maximum shallow settlement of -4.7mm, increasing the maximum settlement of the array to -8.3mm with the construction of the Northbound Tunnel. Array B recorded a maximum shallow settlement of -2.8mm, increasing the maximum settlement of the array to -4.5mm with the construction of the Northbound Tunnel.

In the case of the deep settlement gauges (Figure G-5, p 213) only three were installed from the Education Car Park to the end of construction. Instrument DS212 was founded too close to the crown of the tunnel and recorded excessive settlement; DS211 recorded a heave of +4.8mm, and DS210 reported a settlement of -6.9mm.

Figure 4-33 (p 119) presents the observed shallow settlements from Education Car Park to the final breakthrough. The maximum observed settlement was at instrument SS212 and was -15.7mm. This settlement is also reflected in the corresponding deep instrument DS212 which reported a maximum settlement -239mm. It was in this region that the TBM encountered a pocket of intra-till sands and lost the soil face, leading to large ground settlement. If the two instruments closest to the end of construction are also neglected, due to the relaxed grouting procedures towards the end of construction, then the maximum observed shallow settlement in this region was -5.4mm. The observed shallow settlement at Array A (Figure 4-48, p 139) above the Northbound Tunnel increased from -2.3mm to -8.1mm after the construction of the Northbound Tunnel. The observed shallow settlement above the Southbound Tunnel increased from -4.3mm to -6.9mm after the construction of the Northbound Tunnel.

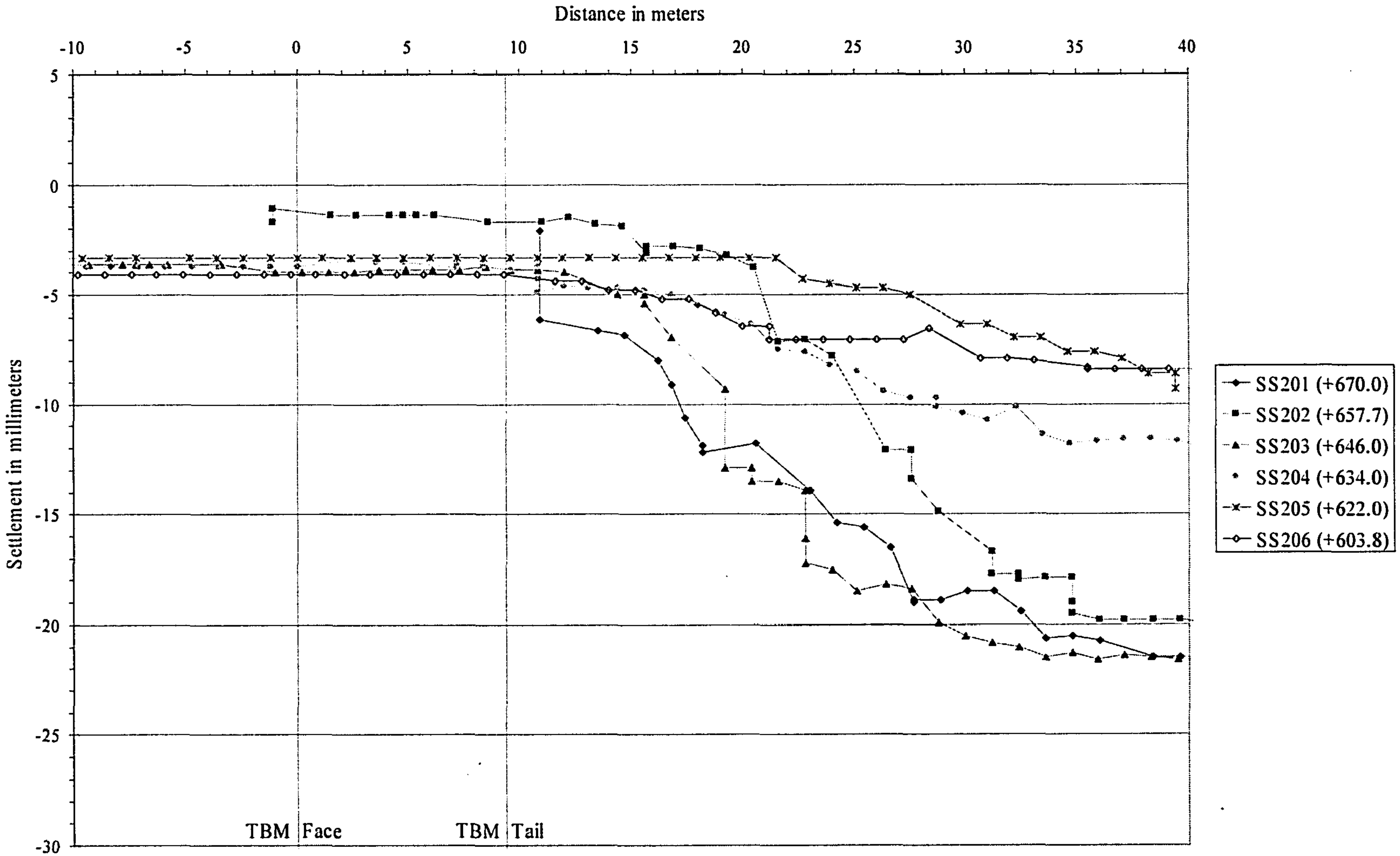


Figure 4-31 Observed shallow settlements during construction of Northbound Tunnel with proximity to TBM face, from Portal to Education Car Park.

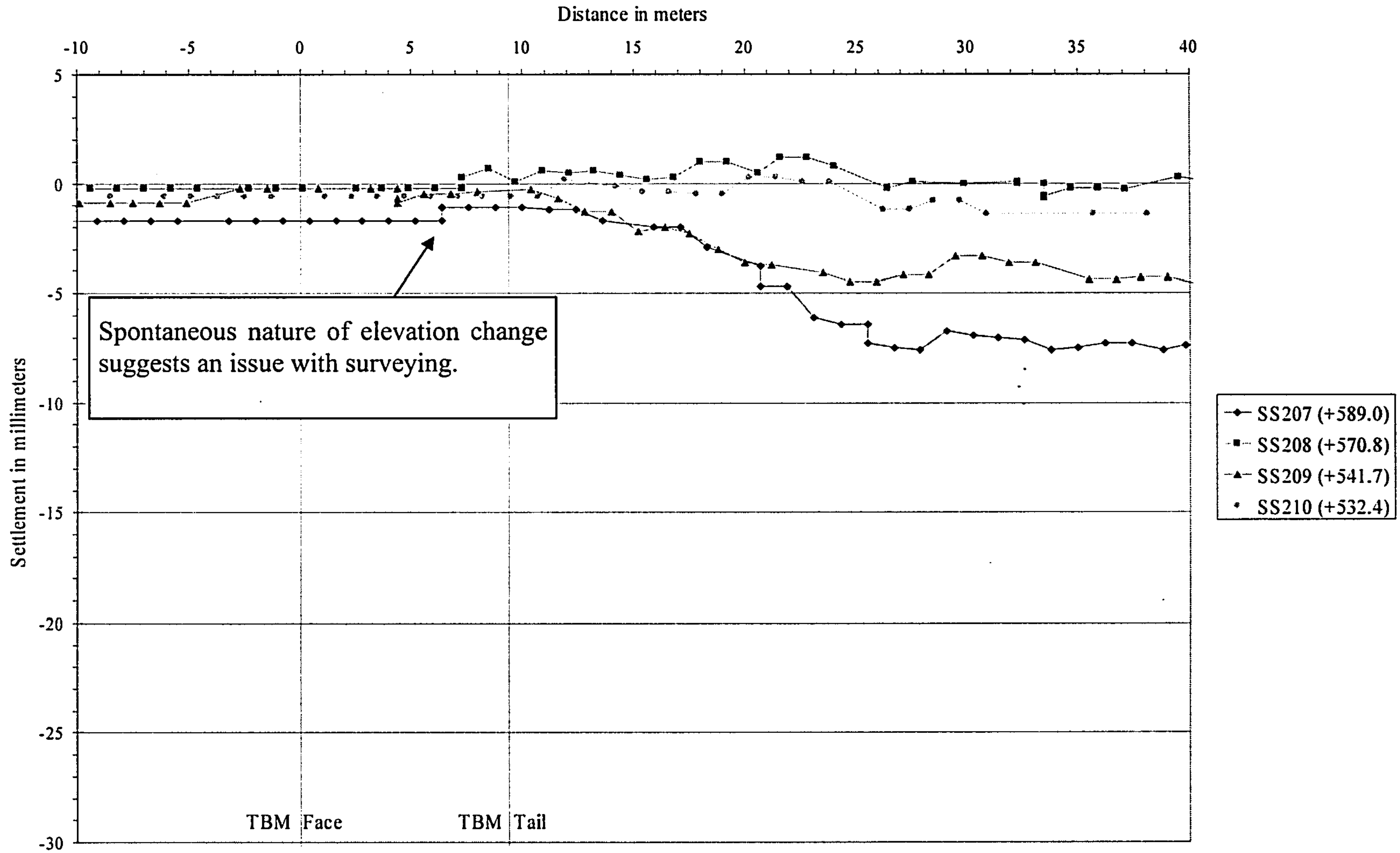


Figure 4-32 Observed shallow settlements during construction of Nouthbound Tunnel with proximity to TBM face, Education Car Park.

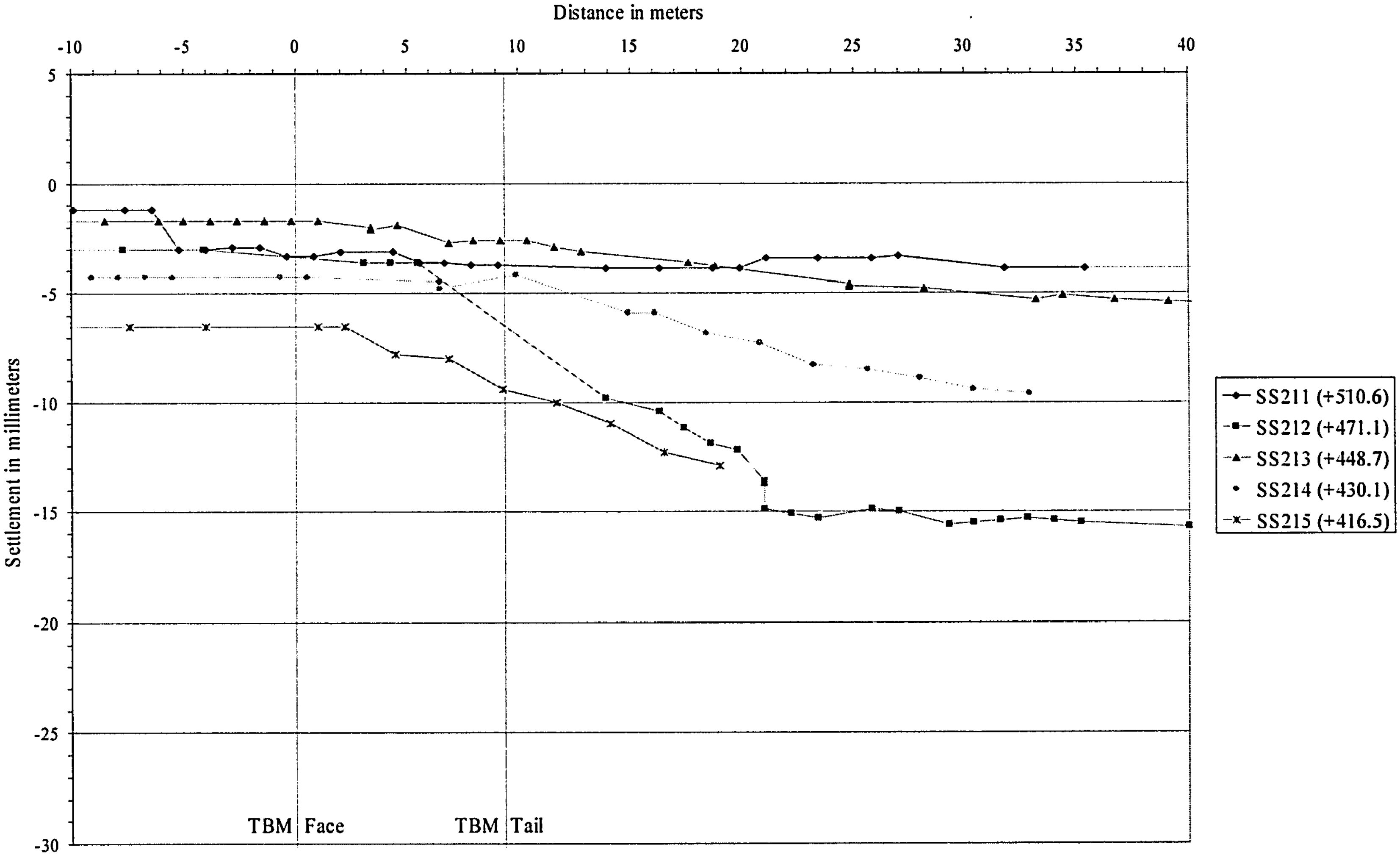


Figure 4-33 Observed shallow settlements during construction of Northbound Tunnel with proximity to TBM face, from Education Car Park to breakthrough.

4.3.3 Time Dependent Settlement – Southbound

The figures presented in this section detail the settlements observed at each shallow settlement gauge overlying the Southbound Tunnel with respect to time. The timeline for each Figure is from the start of construction of the Southbound Tunnel through to the completion of the Northbound Tunnel. The Figures for the deep settlement instruments can be found in 'Appendix G – Deep Proximity and Time Settlements' (p 208) and are referred to as Figure G-#.

Figure 4-34 (p 122) details the observed shallow settlements in the region from the Portal to the Education Car Park. Figure G-6 (p 214) presents the observed deep settlements for the same region. The shallow settlements induced by the construction of the Northbound Tunnel increased the total settlements to between -16mm and -26.4mm, the maximum occurring at instrument SS105.

Figure 4-35 (p 123) presents the shallow settlements observed in the region beneath the Education Car Park. Figure G-7 (p 215) presents the information for the deep settlement gauges in this region. The shallow settlements induced by the construction of the Northbound Tunnel increased the total shallow settlements to between -2.9 mm and -3.6mm.

Figure 4-36 (p 124) details the observed shallow settlements in the region from the Education Car Park to St. Joseph's College. Figure G-8 (p 216) presents the deep settlement information for the same region. Construction of the Northbound Tunnel increased the total shallow settlements observed in this region to between -1.7mm and -9.1mm. Instrument S115 reported the greatest settlement of -9.1mm with all other instruments being less than -5.5mm.

S122 reported a settlement in excess of -12mm. However, S122 was located within several meters of the removal shaft. Reduced face pressures and injected grout volumes towards the end of construction are the most likely explanation for the large settlement observed at S122.

Figure 4-37 (p 125) presents the observed shallow settlements in the region from St. Joseph's College to the end of construction at the removal shaft. Figure G-8 (p 216) presents the information for the deep settlement points. The construction of the Northbound Tunnel increased the observed shallow settlements in the region to between -4mm and -13.1mm. The instrument reporting the largest settlement, S122, was the instrument located adjacent to the removal shaft.

After the completion of the Southbound Tunnel the TBM was broken down and rebuilt in position for the Northbound Tunnel. During this ten week period, surveys of the instrumentation continued. No settlements were observed in any of the instrumentation during this period. This response is characteristic of the fact that the Outwash Sands and Silts and Glacial Till are materials that are not prone to long term settlements.

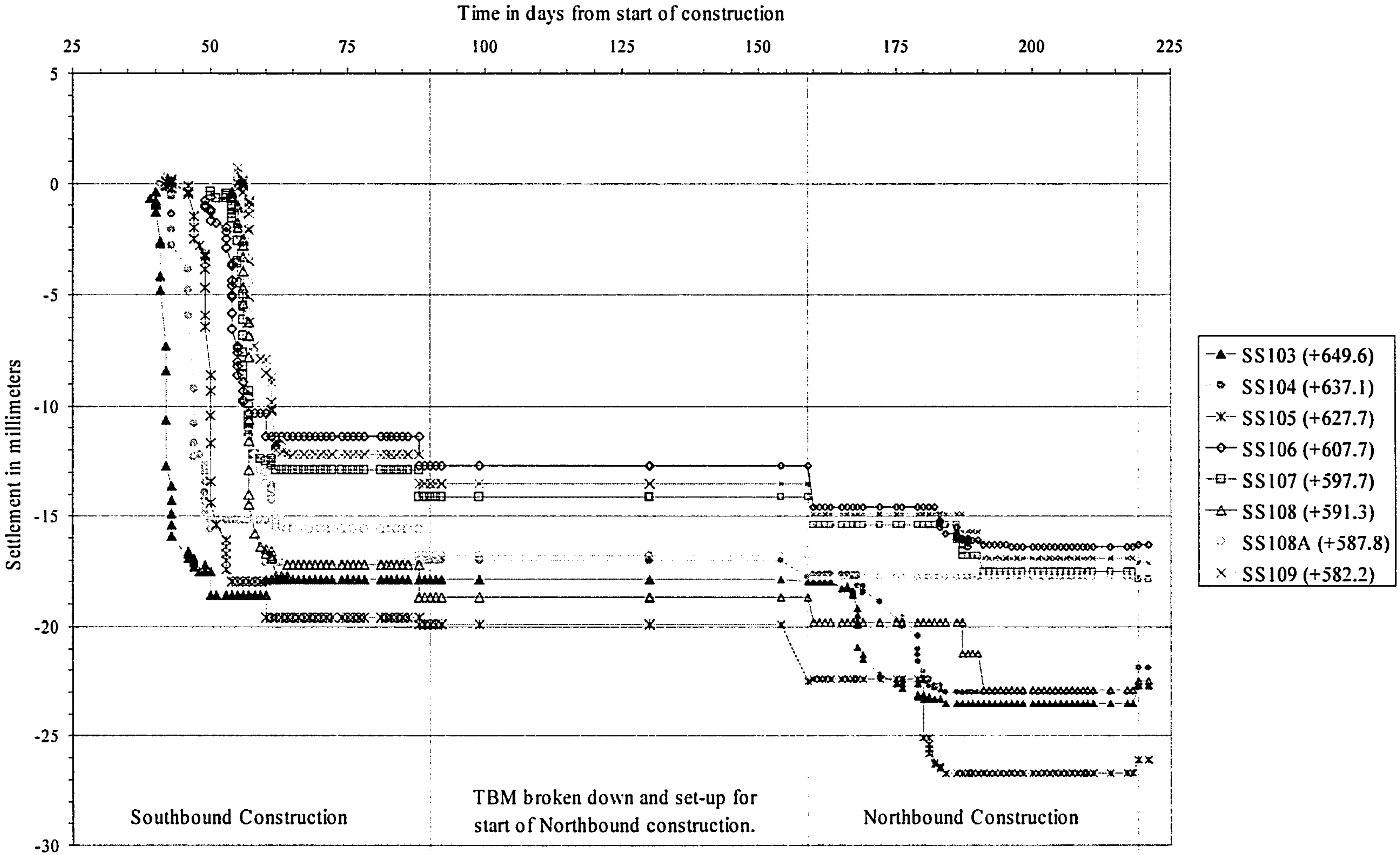


Figure 4-34 Observed shallow settlements in Southbound alignment instruments over the course of construction with respect to time, from Portal to Education Car Park.

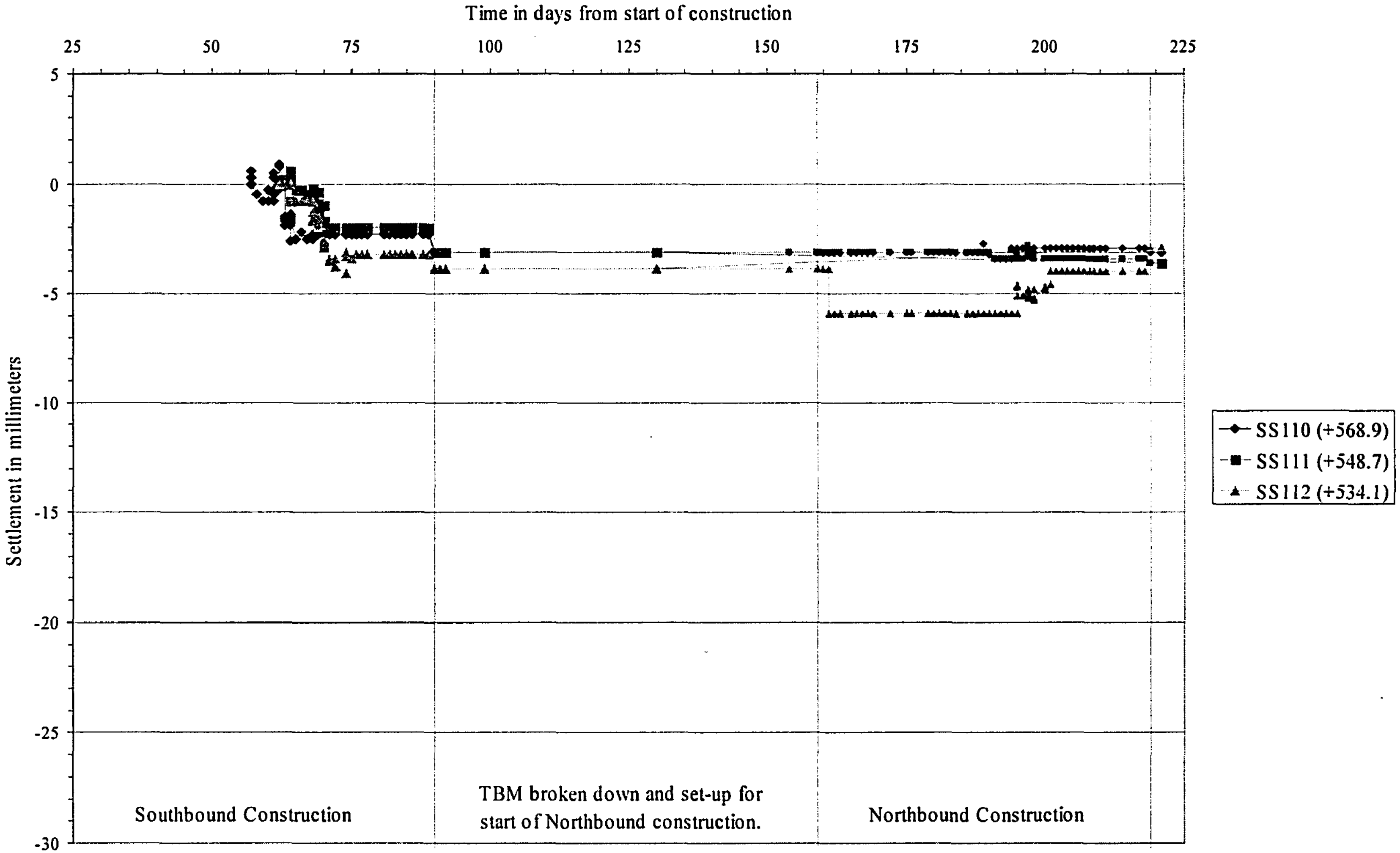


Figure 4-35 Observed shallow settlements in Southbound alignment instruments over the course of construction with respect to time, Education Car Park.

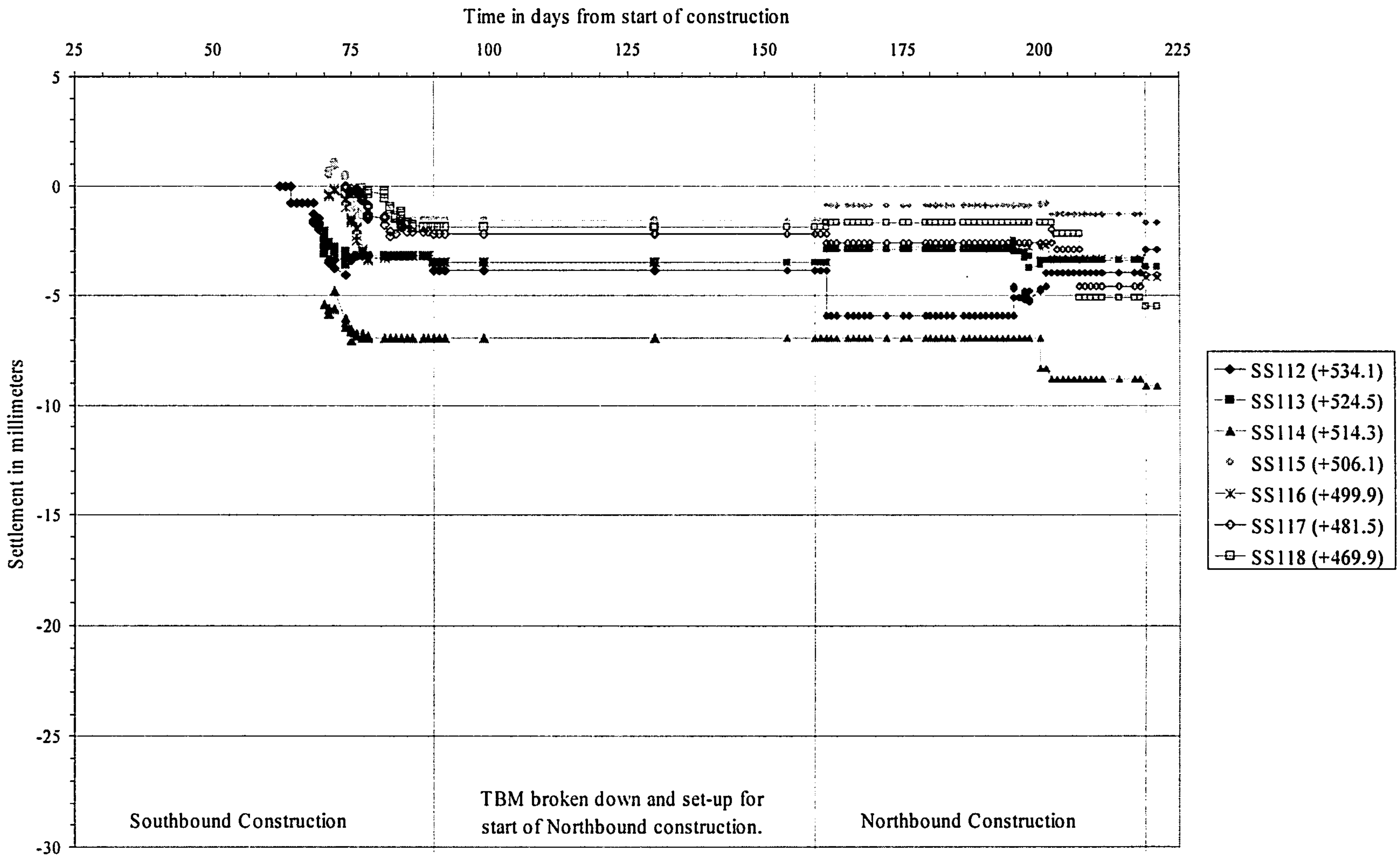


Figure 4-36 Observed shallow settlements in Southbound alignment instruments over the course of construction with respect to time, from Education Car Park to St. Joseph's College.

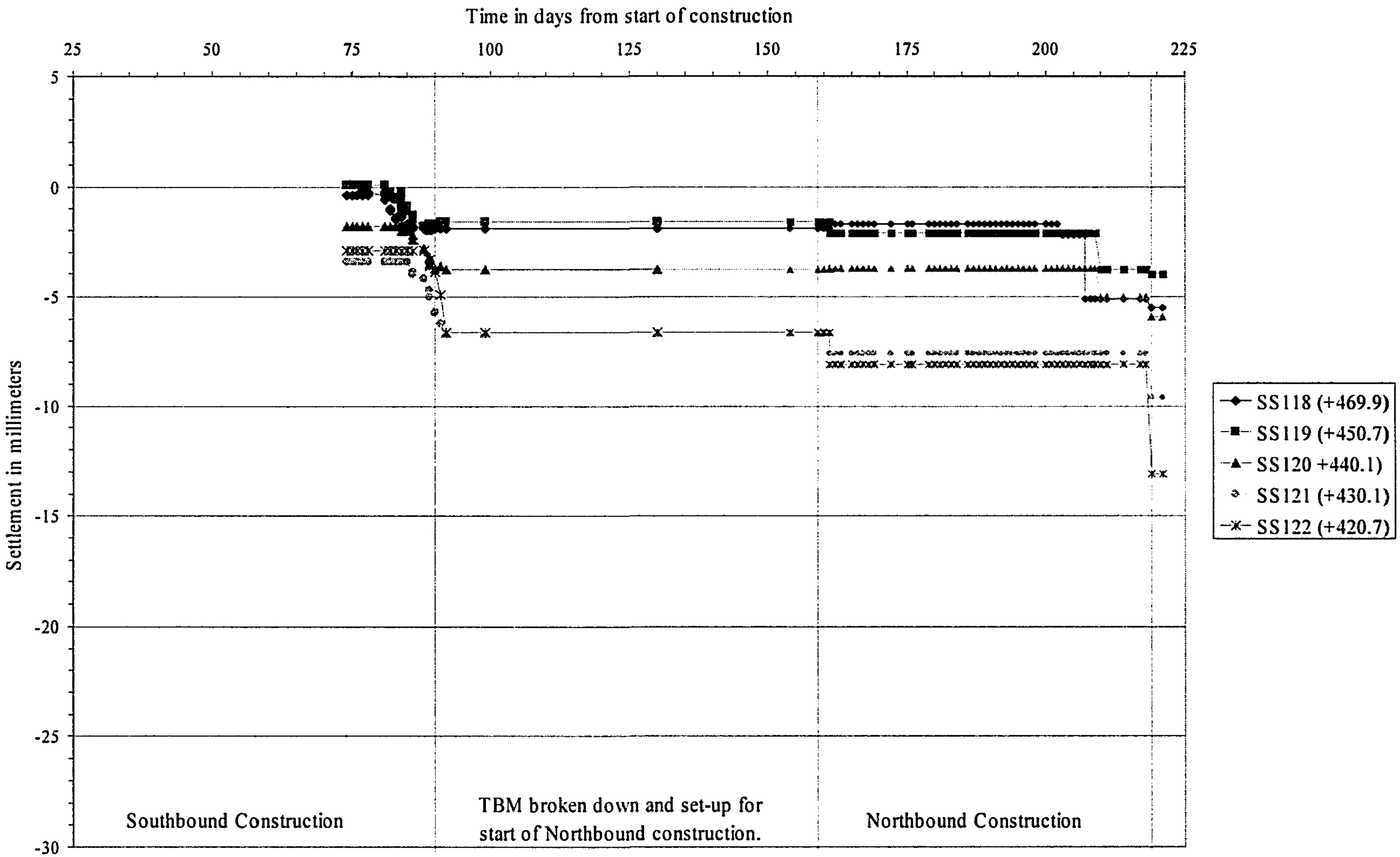


Figure 4-37 Observed shallow settlements in Southbound alignment instruments over the course of construction with respect to time, from St. Joseph's College to breakthrough.

4.3.4 Time Dependent Settlement - Northbound

The figures presented in this section detail the settlements observed at each shallow settlement gauge overlying the Northbound Tunnel with respect to time. The timeline for each Figure is from the start of construction of the Southbound Tunnel through to the completion of the Northbound Tunnel. The Figures for the deep settlement instruments can be found in 'Appendix G – Deep Proximity and Time Settlements' (p 208) and are referred to as Figure G-#.

Figure 4-38 (p 128) and Figure G-9 (p 217) detail readings from the region from the Portal to Education Car Park and present the shallow and deep settlements respectively. The observed settlements induced by the construction of the Southbound Tunnel did not exceed -5mm. No readings were taken using the deep instrumentation in this region during Southbound construction.

Figure 4-39 (p 129) details the observed shallow settlements in the region beneath the Education Car Park. Induced settlement from the construction of the Southbound Tunnel reached a maximum of -1.7mm at SS207. The remaining shallow instruments reported settlements of less than -0.7mm.

Figure G-10 (p 218) details the observed deep settlements in the region from the Education Car Park through breakthrough. Settlements induced by construction of the Southbound reached a maximum of -0.7mm at instrument DS209.

Figure 4-40 (p 130) presented the observed shallow settlements in the region from the Education Car Park through final breakthrough. The largest observed settlement was -1.4mm at SS215, the instrument nearest to the removal shaft.

The settlements observed during the construction of the Northbound Tunnel with respect to time, showed no observable long-term settlements. The observed settlements during the construction of the Northbound Tunnel were a function of the proximity to the Tunnel

Boring Machine with little additional settlement occurring after the passage of the TBM. The material characteristics of the Outwash Sands and Silts and the heavily over-consolidated Glacial Till, also make it highly unlikely that the settlements reported at the end of construction will increase with the passage of time.

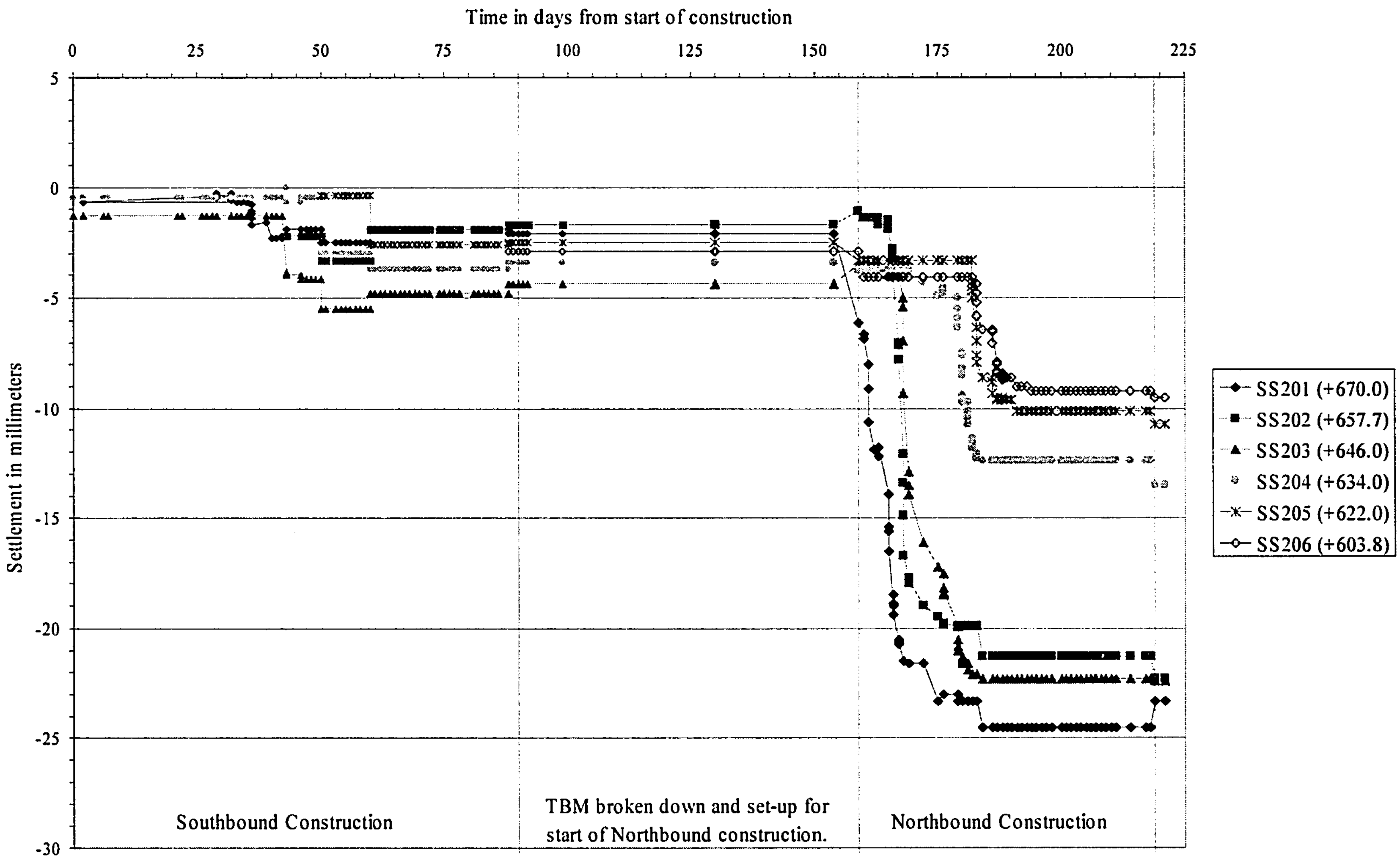


Figure 4-38 Observed shallow settlements in Northbound alignment instruments over the course of construction with respect to time, from Portal to the Education Car Park.

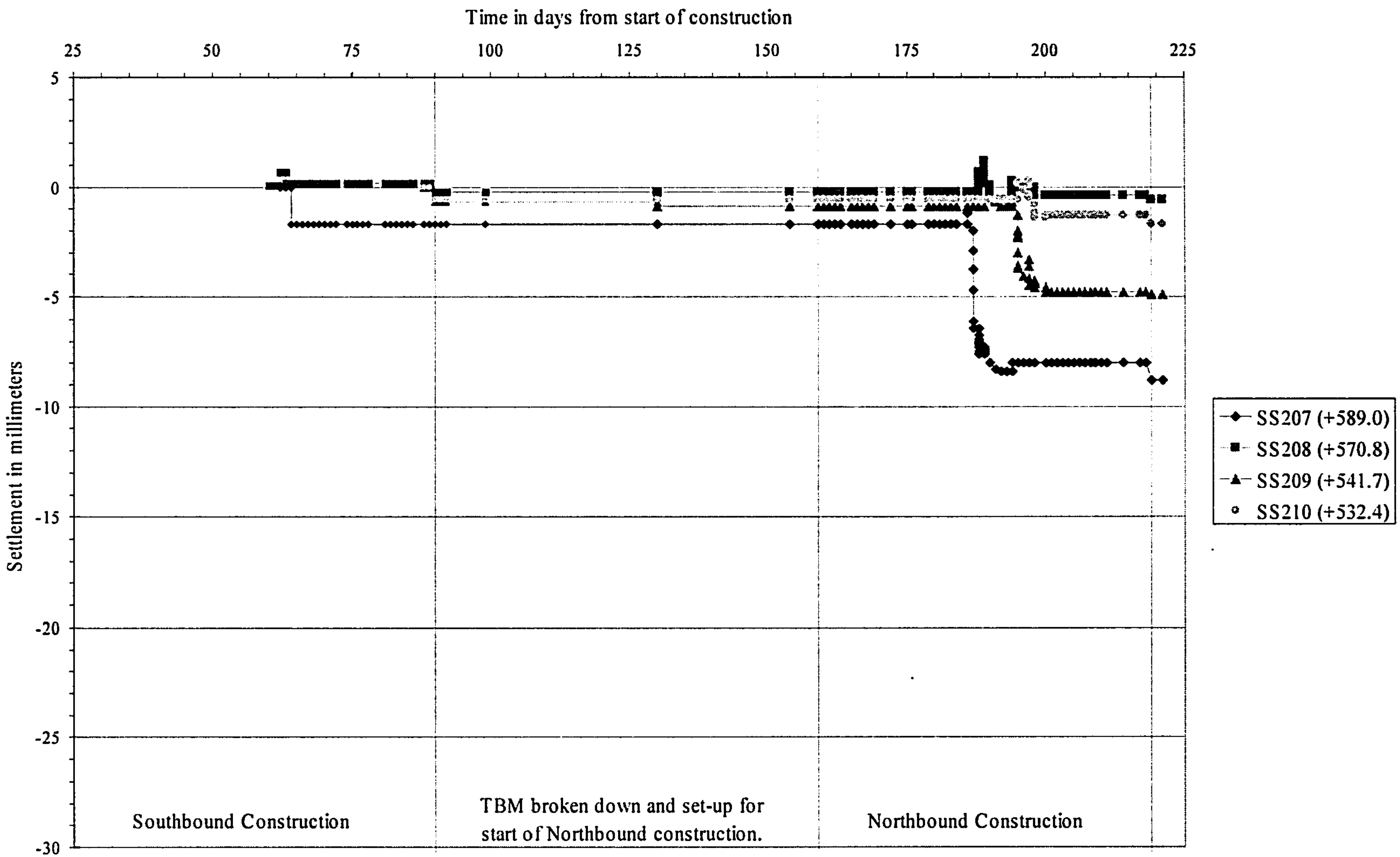


Figure 4-39 Observed shallow settlements in Northbound alignment instruments over the course of construction with respect to time, Education Car Park.

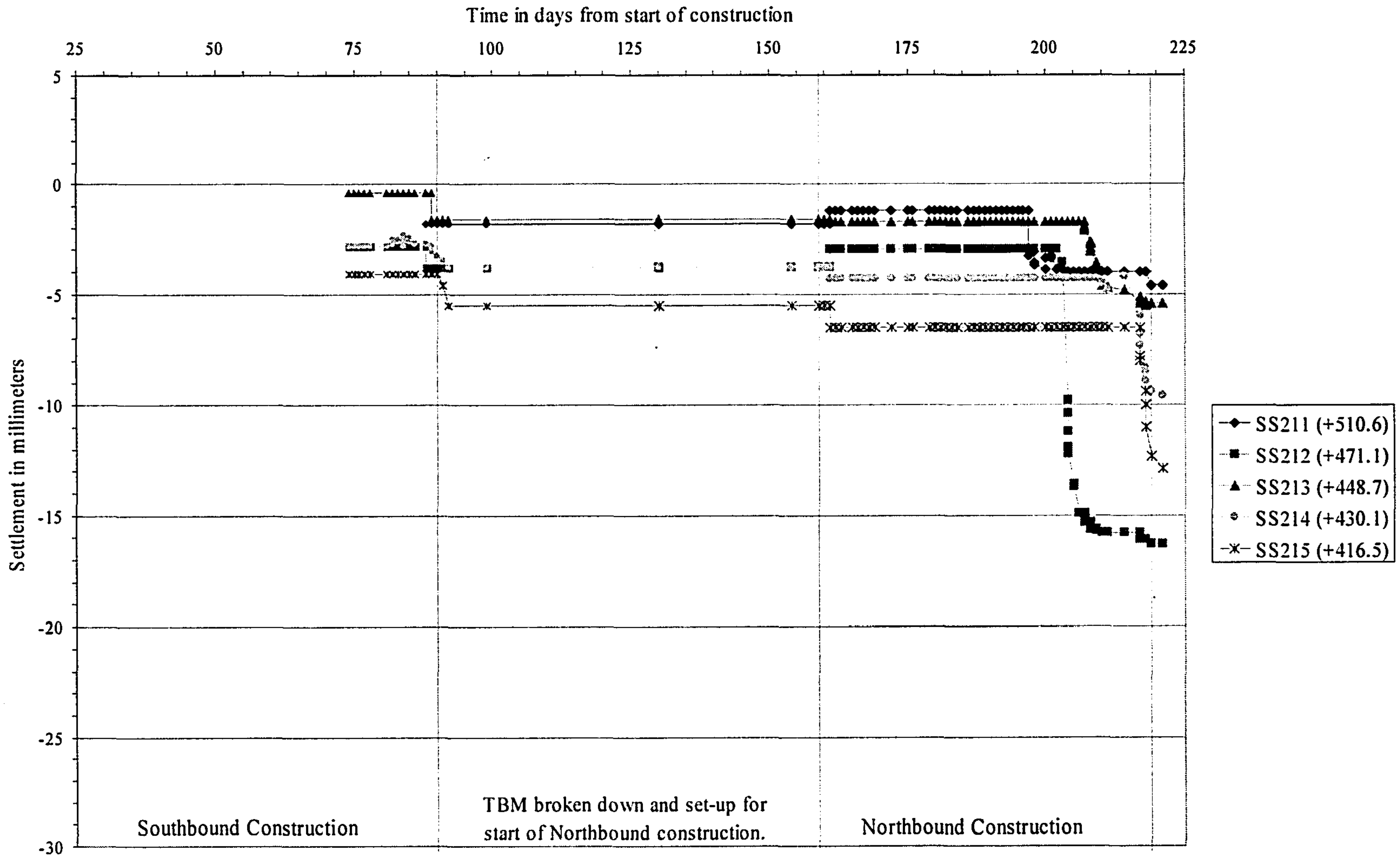


Figure 4-40 Observed shallow settlements in Northbound alignment instruments over the course of construction with respect to time, from the Education Car Park to removal shaft.

4.3.5 Settlement Troughs

The Figures in this Section present the settlement data for the arrays located along the SLRT alignment. Discussion relating to these Figures, specific to Southbound and Northbound construction, can be found in Section 4.3.1, "Observed Southbound Settlements", and Section 4.3.2, "Observed Northbound Settlements."

The arrays show the settlement perpendicular to the tunnel alignment. The elevation of the settlement gauges are reported over an interval of 50m. From when the TBM is 10m from reaching the gauge (-10m), to when the TBM has passed 40m beyond the settlement gauge (40m).

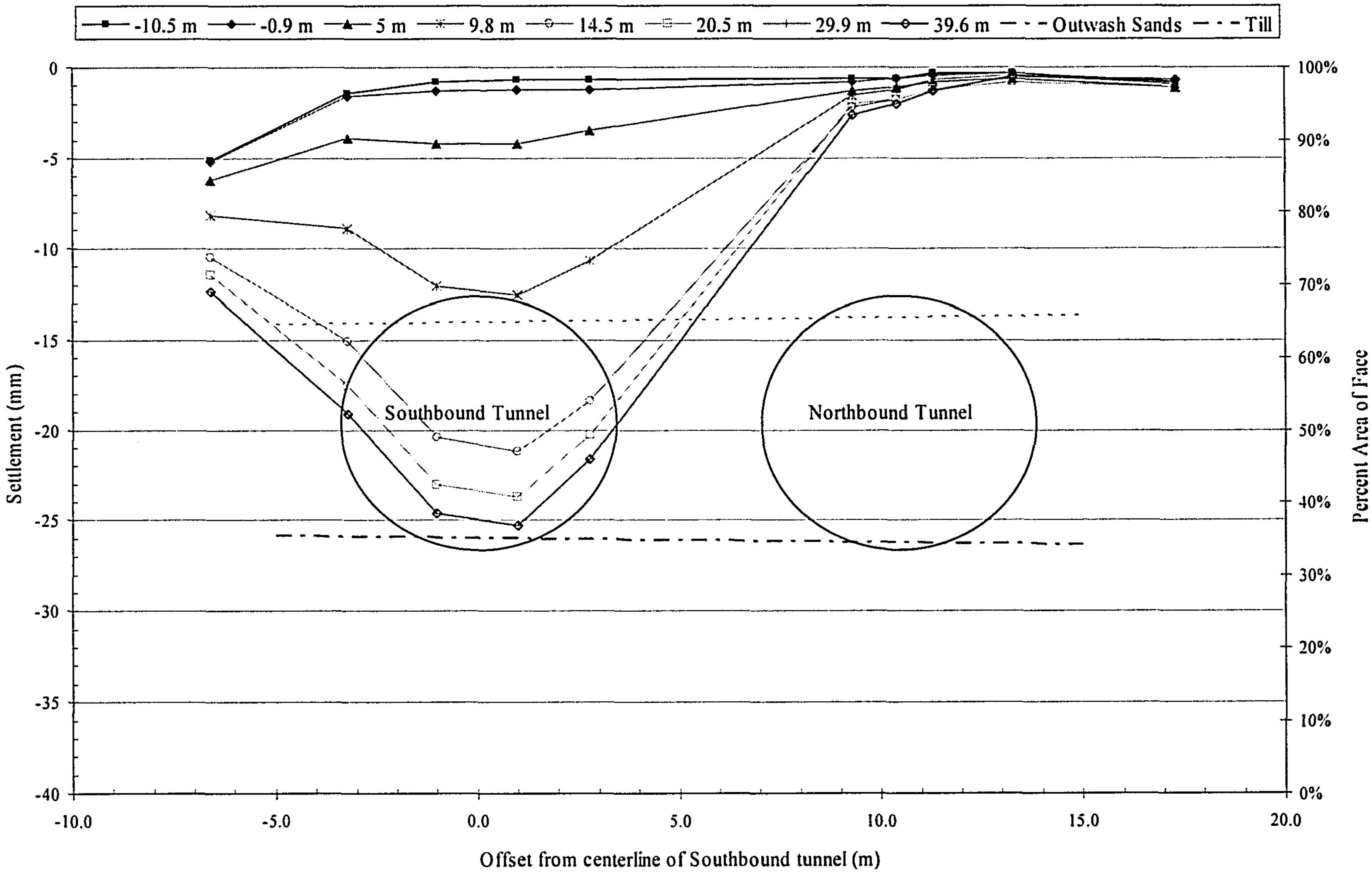


Figure 4-41 Observed shallow settlement at Array D during construction of Southbound Tunnel.

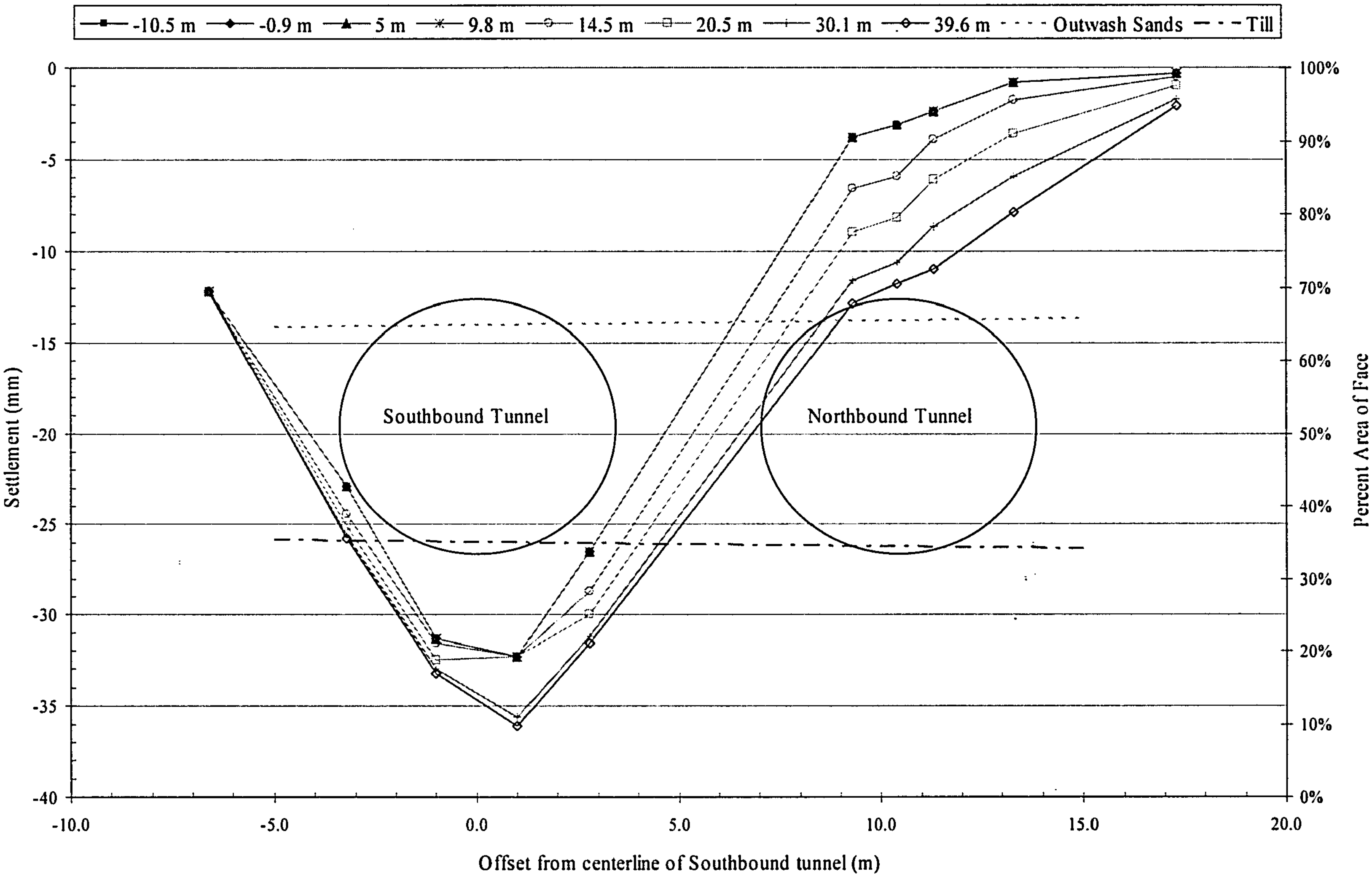


Figure 4-42 Observed shallow settlement at Array D during construction of Northbound Tunnel.

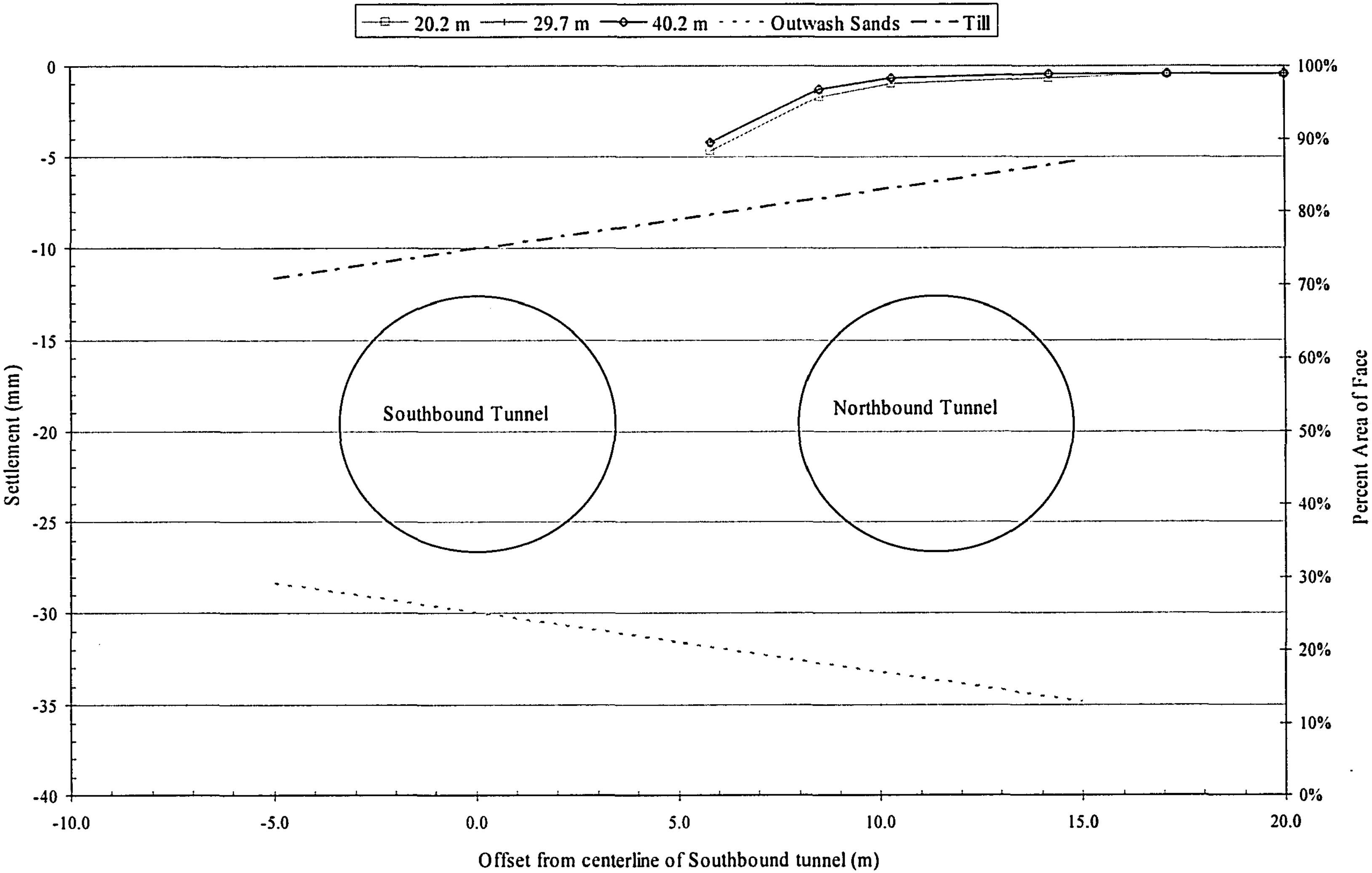


Figure 4-43 Observed shallow settlement at Array C during construction of Southbound Tunnel.

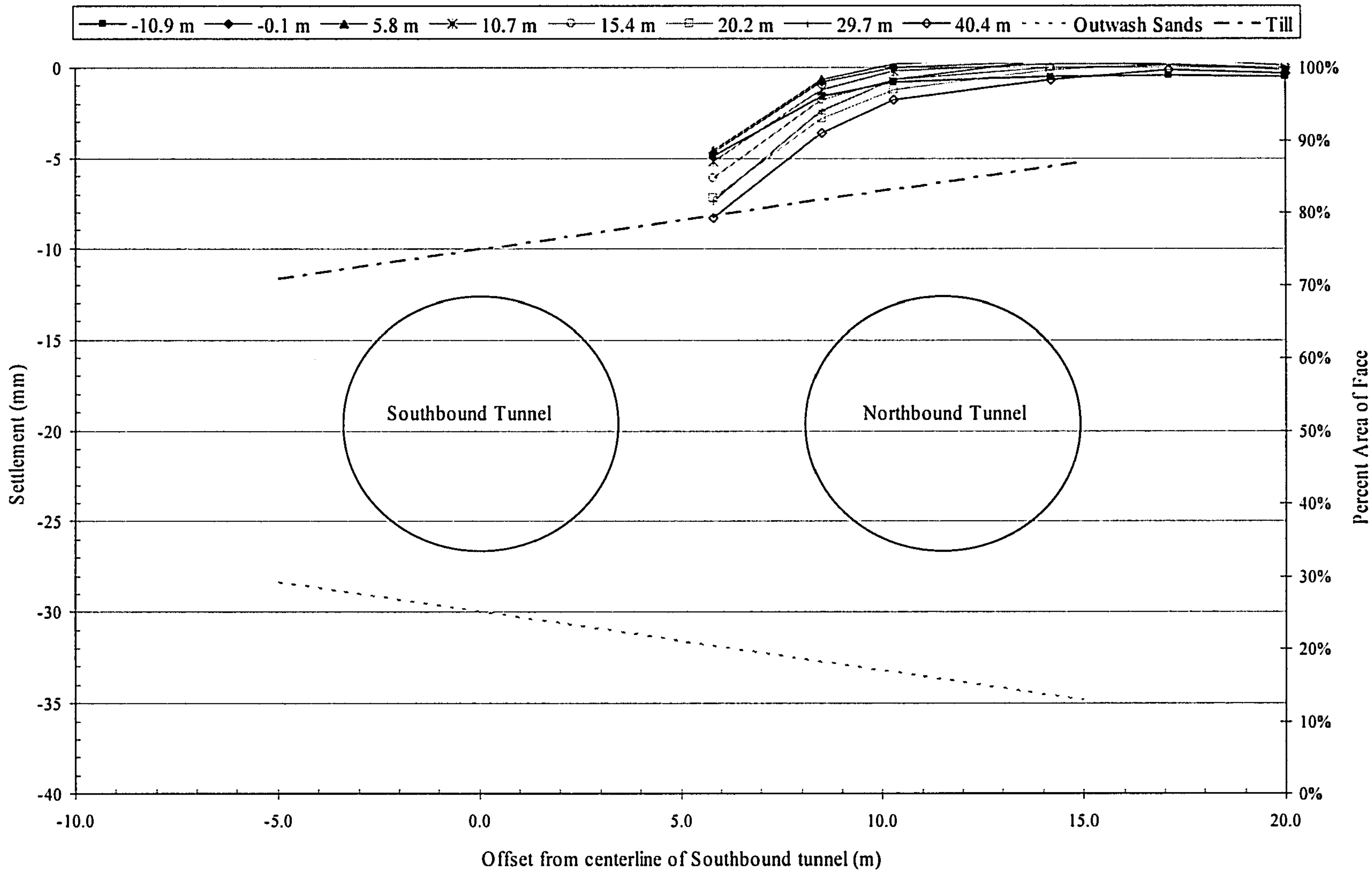


Figure 4-44 Observed shallow settlement at Array C during construction of Northbound Tunnel.

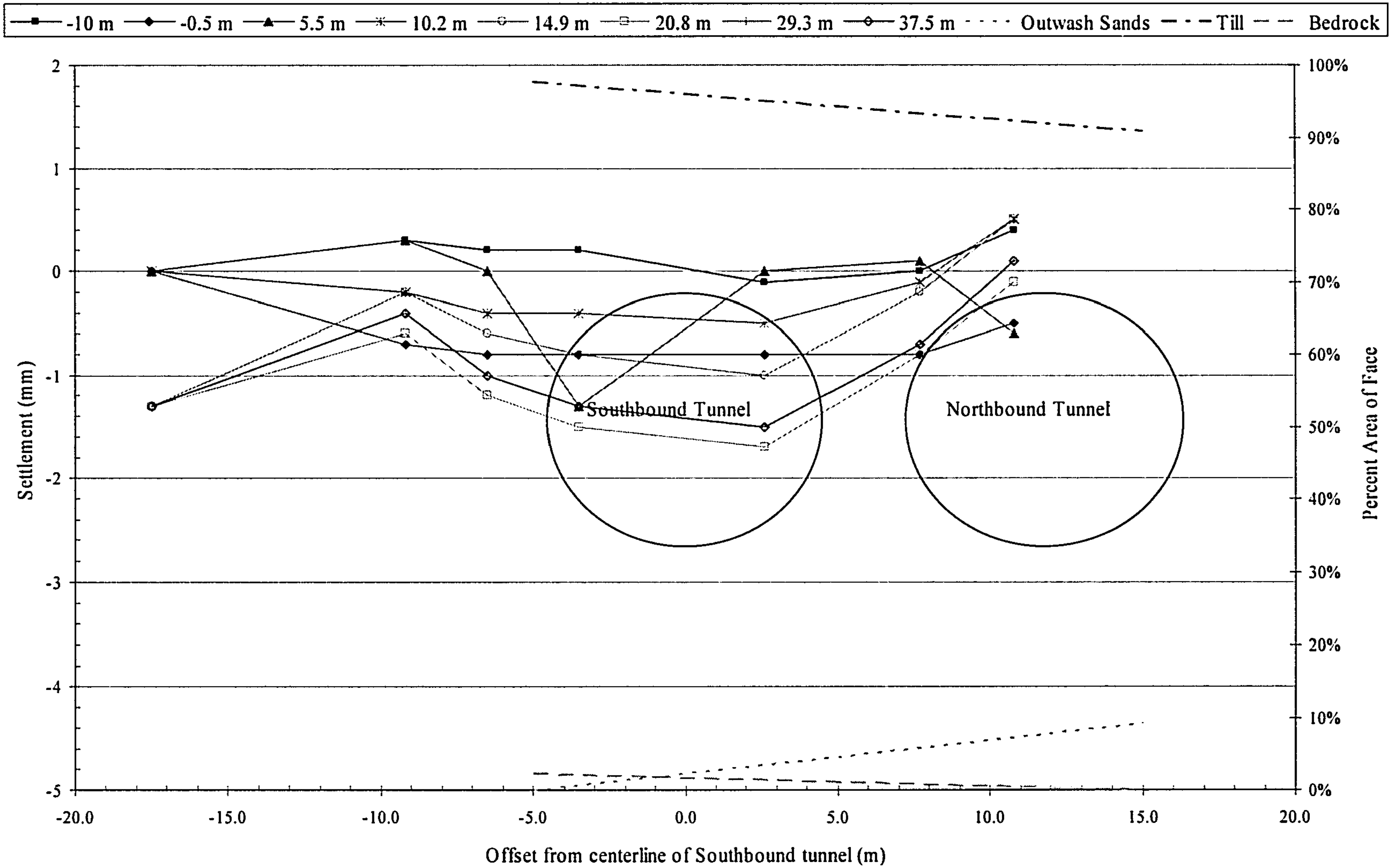


Figure 4-45 Observed shallow settlement at Array B during construction of Southbound Tunnel.

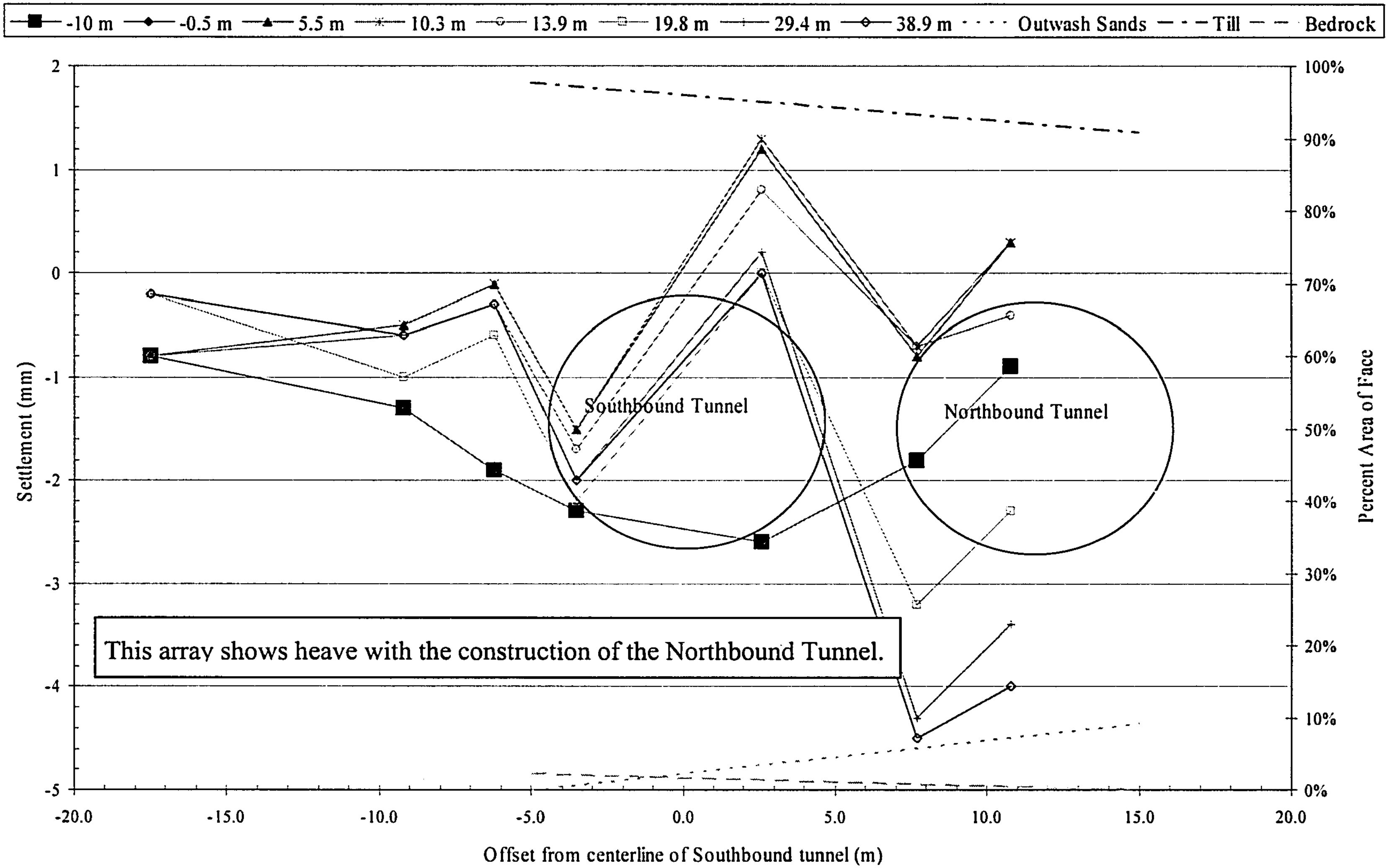


Figure 4-46 Observed shallow settlement at Array B during construction of Northbound Tunnel.

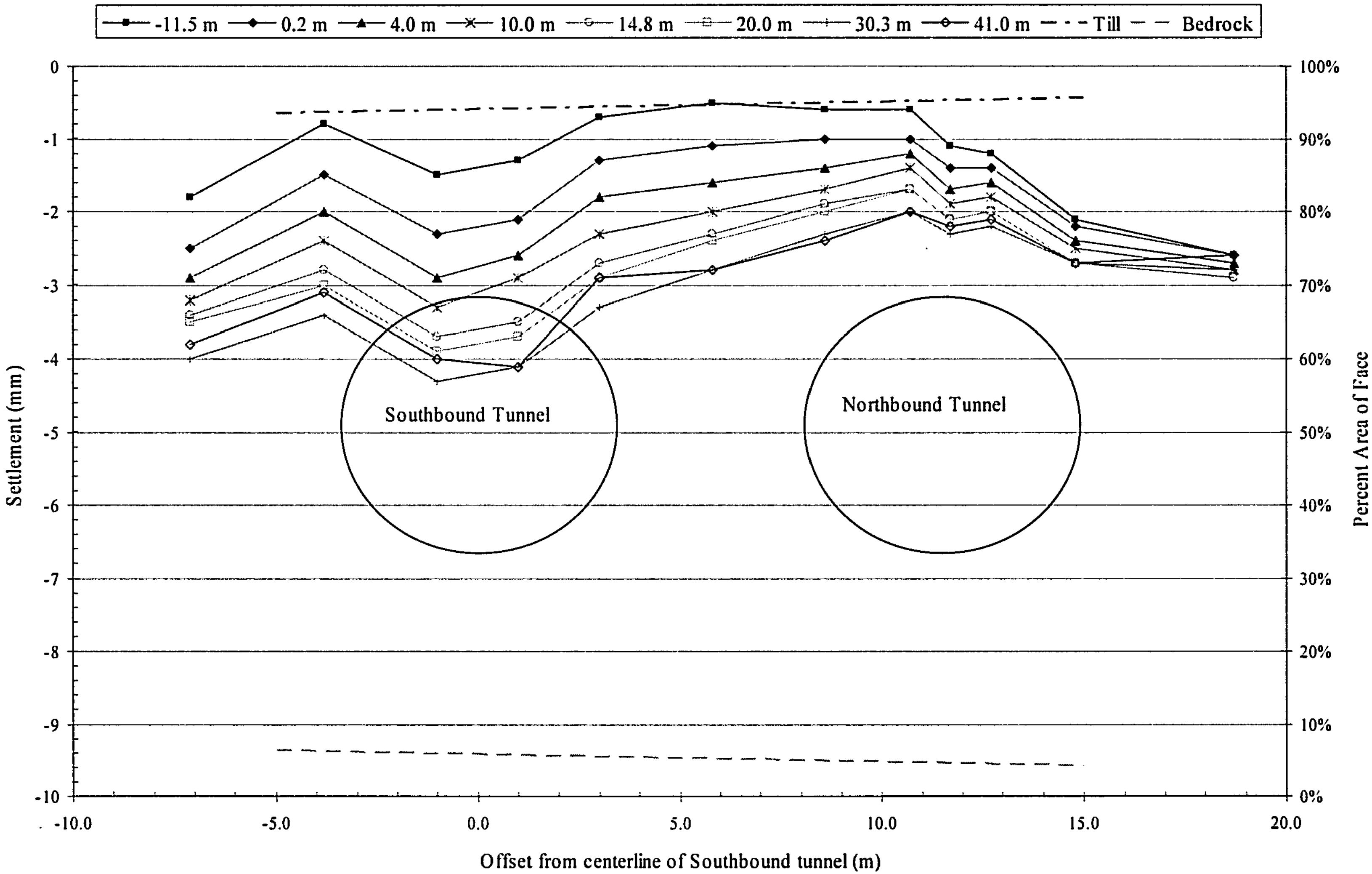


Figure 4-47 Observed shallow settlement at Array A during construction of Southbound Tunnel.

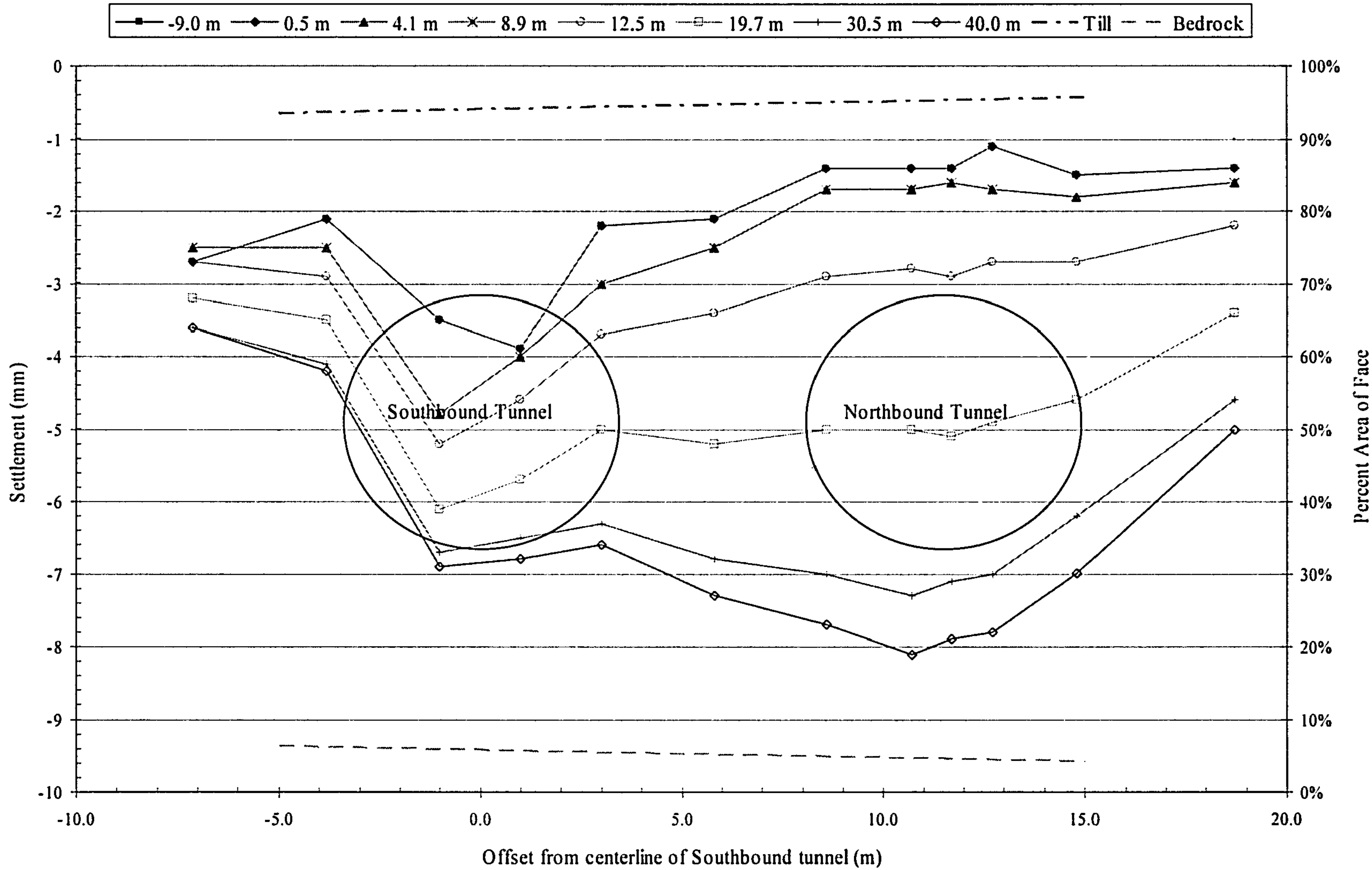


Figure 4-48 Observed shallow settlement at Array A during construction of Northbound Tunnel.

4.3.6 Settlement of Education Car Park, Footings and Utility Corridor.

The various structures encountered along the tunnel alignment were also monitored for settlement. The individual footings of the Education Car Park were monitored using traditional levelling techniques. This monitoring resulted in elevation profiles for the West and North Walls of the building as well as for the interior columns. The Utility Corridor was monitored by taking level shots along the floor of the structure. This monitoring provided a comparison between the ground movements and the structural movements.

A selection of array settlement profiles for the structures that were undermined by the construction of the Northbound and Southbound Tunnels can be found in 'Appendix I – Building Array Settlement' (p 228). This Appendix contains a series of five Figures detailing the observed settlements in three regions: the West and North Walls of the Education Car Park, and the Utility Corridor.

During the construction of the Edmonton SLRT, the first structure to suffer impact from the construction of the Southbound Tunnel was the Utility Corridor. During the construction of the Southbound Tunnel the maximum settlement observed on the Utility Corridor was -7.6mm at building monument 94, which can be located in Figure 4-26 (p 108). During construction of the Northbound Tunnel, the maximum settlement increased to -13.3mm at building monument 94.

The next structure to be passed under during construction was the Education Car Park. The two most vulnerable parts of the structure to the construction were the West and North walls. Settlements at the West Wall reached a maximum of -12mm during the construction of the Southbound Tunnel at building monument 42. Along the North Wall, the maximum observed settlement was -4.7mm at building monument 36. After the construction of the Northbound Tunnel, the maximum observed settlement on the West Wall increased to -19.4mm at building monument 42, and on the North Wall to -6.2mm at building monument 35.

4.4 Grout set time

It was shown in Figure 2-4 (p 13) that control of settlements beyond the TBM shield is related to the grouting methodology. Assuming that the void created in transitioning to the segmental lining is completely filled with grout: the amount of settlement must be related to the viscosity and set time of the grout. Using a grout of low viscosity and fast set time it is likely that ground settlements would be minimised. However, using a grout possessing a high viscosity and long set time, it is likely that settlements would be at a maximum. Because the grout viscosity and set-time are functions of mix design, the characteristics of the grout mix can significantly influence grout settlements. In this section the grout mix and its characteristics are reviewed.

During the construction of the Edmonton SLRT extension three mix designs were used (Table 4-1). Mix design '1' was the initial grout mixture proposed. Mix design '2', a revision of '1', was used for the initial construction of the Southbound Tunnel. Mix design '3' was used for grout injection after the adoption of a revised grouting procedure (Appendix F, p 202) prior to passing under the Education Car Park during the construction of the Southbound Tunnel.

Table 4-1 Grout designs used.

Mix Design	1	2	3	
Cement	193	225	225	Kg
Flyash	297			Kg
Sand	909			Kg
Fine Aggregate		1473	500	Kg
Extra Fine Aggregate			1500	Kg
Bentonite	24			Kg
Water	435	220	205	Kg
Water Reducing Agent	5.1	*	*	L
	* As required			

The grout used for injection was batched at a local concrete plant and driven to site; the time necessary for transportation was approximately 1 hour. After arriving on site, the grout was loaded via a drop pipe into the grout car which was part of the train. The material remained in the grout car, under agitation, until pumped out as needed.

A series of vane shear tests were conducted on collected grout samples to develop a strength profile with time. The test was conducted using grout mixture '3' as identified on Table 4-1 (p 141). Samples were collected on the morning of January 19, 2004, and tested. The sample collected was taken to a cold room, poured into multiple trays, and allowed to set. The temperature of the cold room was maintained near 7C to approximate the in-situ ground conditions. Figure 4-49 (p 143) shows the set up for experiment, and Figure 4-50 (p 144) a tray of grout.

The grout was muddy grey in colour and contained numerous large lumps of fines. The presence of these lumps showed the nature of the grout to segregate severely over short periods of time. During tunnelling operations before the grout was piped down to the grout car it would be passed through a screen to remove the lumps of fines. When the grout was left standing free water would collect on the surface indicating a significant bleed factor.

Figure 4-51 (p 144) illustrates the strength development of the grout over a period of approximately nine hours. At the end of approximately nine hours, the grout had developed mean shear strength of 13 KPa. Although at the time of the last strength test at approximately nine hours the grout could still easily be indented using one's finger. A later reading was attempted at 24 hours; however, the material had set sufficiently to refuse the vane. It should be mentioned, that the material at this time was still sufficiently weak to be crumbled using hand pressure.

Figure 4-49 Set-up for testing of grout.

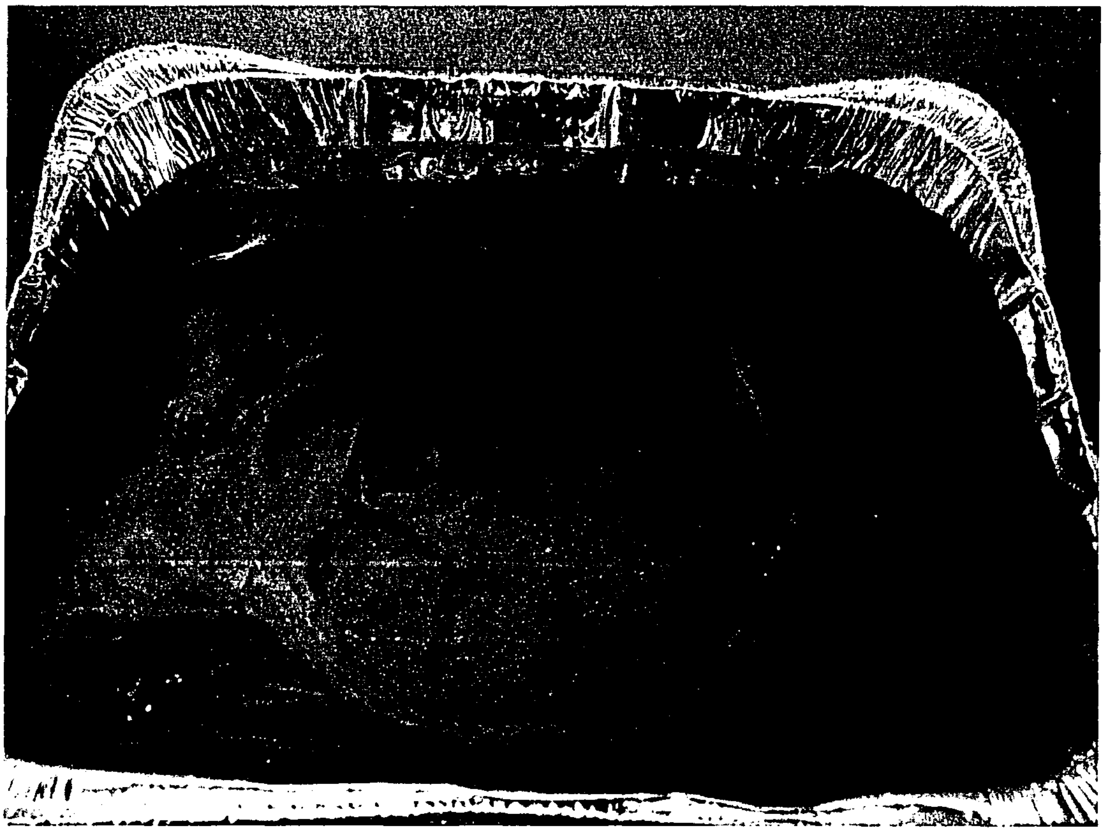


Figure 4-50 Grout 2 hours after being placed to set.

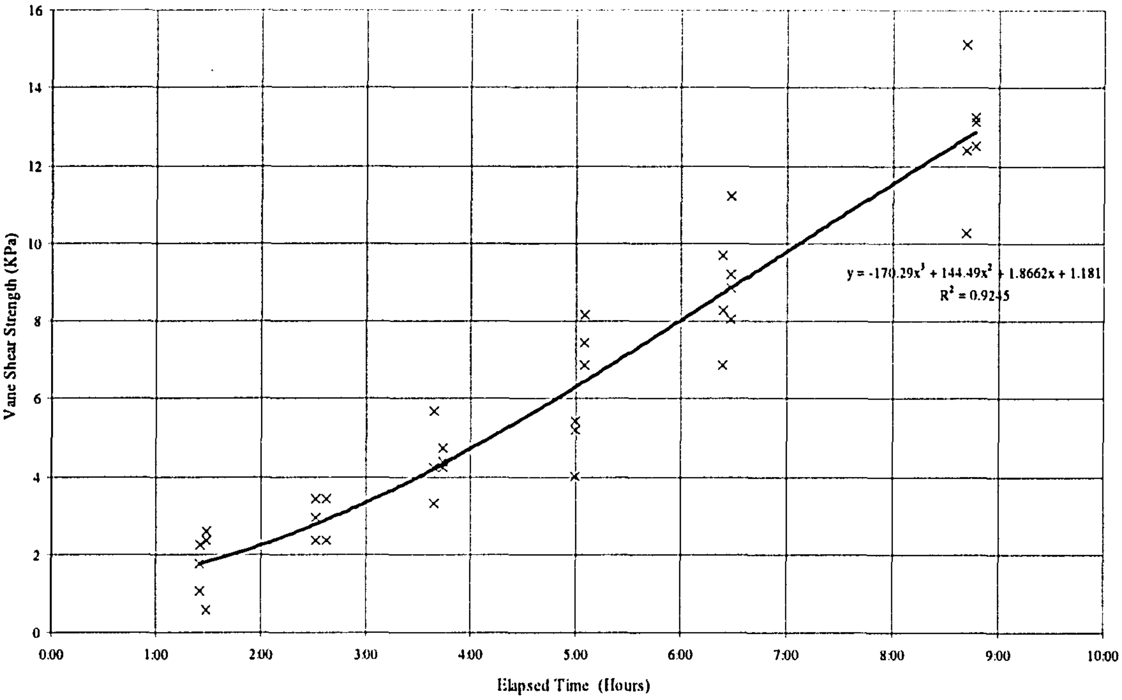


Figure 4-51 Strength development of grout sample.

5.0 Interpretation of TBM Performance and Ground Settlements

In this Section, the two aspects of this thesis will be considered. The first aspect of this review will be an assessment of the performance of the EPB-TBM used for the construction of the Edmonton SLRT. The second aspect will be an evaluation of the settlements observed during the construction and resulting from the performance of the TBM and the local geology.

5.1 TBM Measurements as a Measure of Performance

Discussion in this section will focus on an assessment of the performance of the TBM using measurements related to the operation of the machine.

5.1.1 Process Control of Soil Treatment

Conditioning of the soil in the excavation chamber was accomplished using two agents, foams and polymers. On the Edmonton SLRT project, EKOFOAM, a Morrison Mud product, was selected and used as the foaming agent. TK60 polymer, also manufactured by Morrison Mud, was selected as the polymer agent (Aecon/McNally, 2003).

Figure 5-1 (p 147) and 5-2 (p 148) detail the spoil moisture content in addition to the inferred in-situ moisture content along the Southbound and Northbound alignments respectively. Included in each Figure is the fraction of TBM spoil passing a #200 sieve.

Ideally, a good correlation should exist between the fraction passing a # 200 sieve and the moisture content of the spoil. This is because additional wetting is required when operating in finer grained materials in order to render them into a plastic state. In viewing Figure 5-1 (p 147) one apparent feature is a number of spikes in the graph of spoil moisture content, showing high moisture content at Stn. +625m, +495m, +460m, and +430m, and a reduced moisture content at Stn. +535m and +475m. Each of these occurrences represents a case where the spoil moisture content changed significantly from adjacent samples with varying degrees of change in geology. In the case of the positive moisture content spikes, values of 30% or greater were observed. In contrast, Figure 5-2 (p 148) lacks the number of abrupt spikes that can be observed in Figure 5-1, significant spikes occurring at Stn. +635m, and between Stn. +450m and +425m.

The variations in moisture content observed during the construction of the Southbound Tunnel can be attributed to the 'learning curve' associated with any project. In one instance, during construction of the Southbound Tunnel when encountering significant quantities of Bedrock for the first time, large quantities of polymer were injected as opposed to increasing the injected quantity of surfactant foam. The result of this dose of polymer was to reduce the mixture of bedrock, glacial till, and intra-till sands into a viscous, sticky mass as the polymer served as a binding agent between the fines present. This occurrence delayed production as excavation of the viscous material proved difficult. When construction of the Northbound Tunnel was started in January of 2004, the process 'hiccups' that occurred during the Southbound construction could have been, and were, largely avoided.

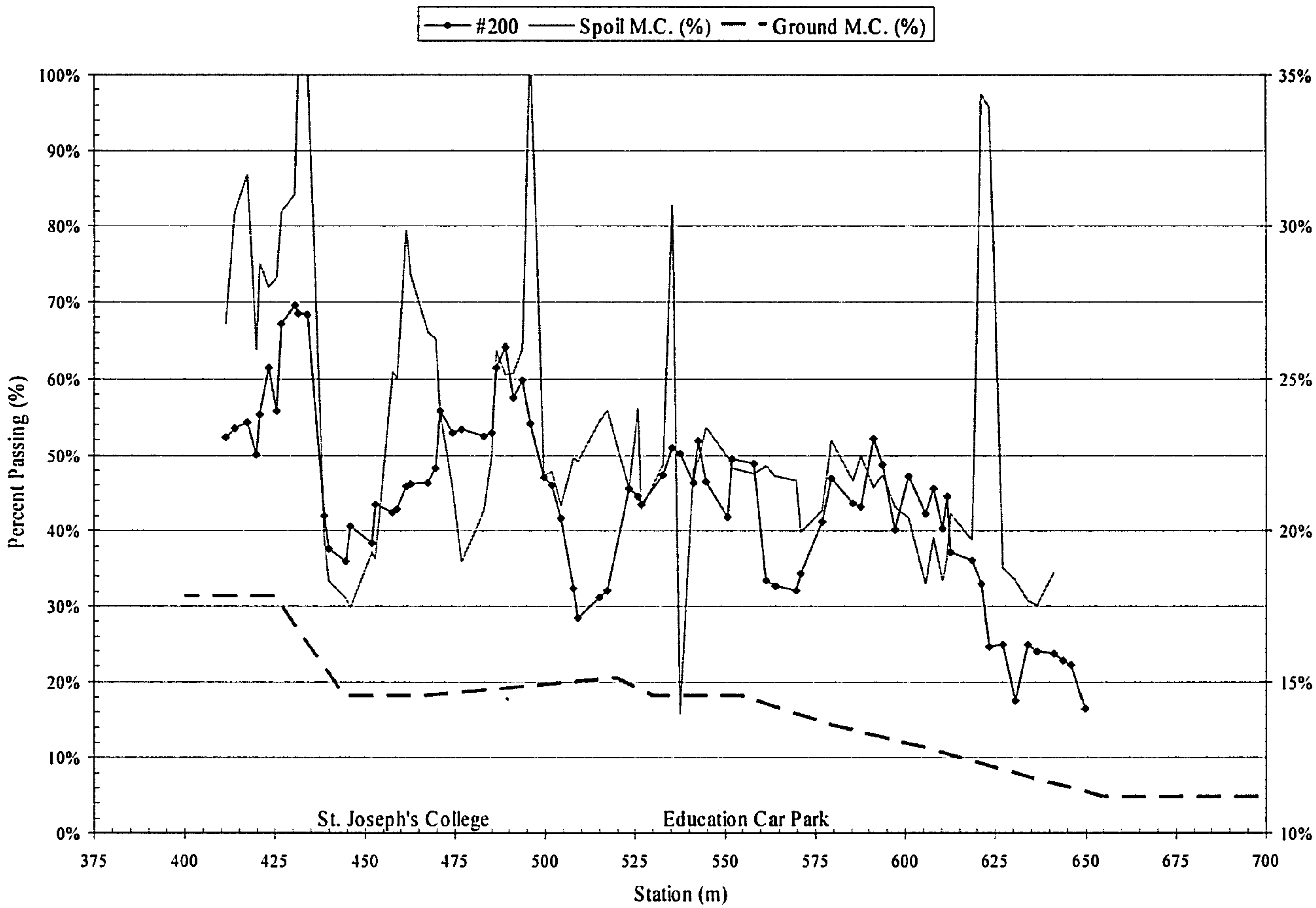


Figure 5-1 TBM spoil moisture content with inferred in-situ moisture content with the percentage of TBM spoil passing the #200 Sieve during Southbound construction.

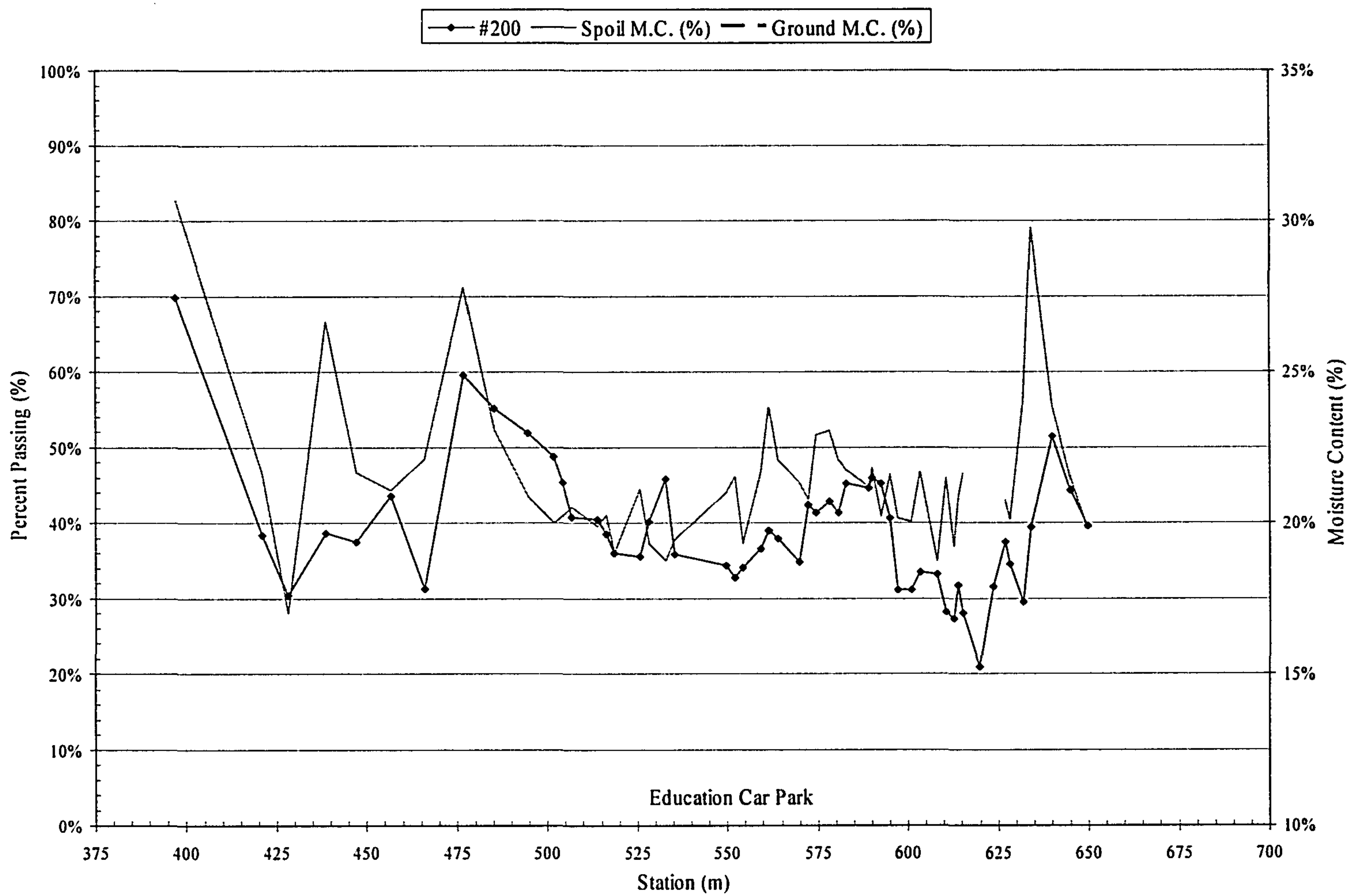


Figure 5-2 TBM spoil moisture content with inferred in-situ moisture content with the percentage of TBM spoil passing the #200 Sieve during Northbound construction.

5.1.2 TBM Advance Rates in Changing Ground Conditions

Figure 5-3 (p 154) shows the construction of the Southbound Tunnel in the format of rings constructed per day. Each ring, when installed, lengthened the tunnel by 1.2m and each work day was in the order of 10 to 12 hours. The Figure shows the progress of construction from ring 0 through to the end of construction. The initial period of construction, which involved the burying of the shield, from ring -8, was neglected, but is presented in Figure 5-5 (p 155) and 5-7 (p 157). One consideration that must be given in viewing this plot is the limitation imposed by the short tunnel length. Construction using a full TBM gantry and a full length train was not possible until the TBM had advanced itself sufficiently to permit the full length of either the train or gantry to be used.

Initial construction of the Southbound Tunnel progressed at a piecemeal rate from the initial launch until September 14, 2003. The limited construction rate at this time was largely a function of technical issues identified once operation of the TBM began, and gaining familiarity with the equipment. Construction after September 14, to the Education Car Park at Stn +475m was in the order of 4.5 rings per day of active operation. After reaching the Education Car Park production reached an average of roughly 5.5 rings per day for the rest of the advance. Although between October 15, and 18, 2003, an average of near 7 rings per day was maintained.

Figure 5-4 (p 154) shows the rings built per day during construction of the Northbound Tunnel from the start of construction through final breakthrough. As with the Southbound Tunnel an initial period of construction took place between January 7 and 24, 2004, where on average only 3 rings were constructed per day. From January 24, 2004, until passing under the Education Car Park, the progress of the TBM was in the order of 4.5 rings per day. Shortly after reaching the Education Car Park at Stn. +590m construction of the tunnel reached an average of near 8 rings per day. This rate of construction was maintained until near the end of the construction of the Northbound tunnel.

Figures 5-3 and 5-4 (p 154), present the construction of the tunnel as a day by day progression. However, it is also possible to view the construction of the tunnel as ring by ring progression in terms of the time needed to excavate each ring of advance.

Figure 5-5 (p 155) details the time needed to excavate each ring of advance during the construction of the Southbound Tunnel, contrasting it with the fraction of the TBM spoil passing the #60 and #200 sieve. Figure 5-7 (p 157) presents the excavation time per ring of advance, but contrasts it with the inferred stratigraphy for the Southbound alignment. A trend line with a moving average of four was used to present the average mine time in both Figures. The initial period of construction, when the TBM was operating with a significantly shortened gantry and train the advance times observed were all in excess of one hour. Between Stn. +640m and Stn. +620m, the average mine time decreased from an average of near one hour to approximately 18 minutes. This region, between Stn. +640m and +620m, corresponded to the first significant contact with Glacial Till, which can be observed in the fraction of the spoil passing the #200 sieve. Between Ring 52, Stn. +620m, and Ring 152, Stn. +500m, the excavation face became predominantly Glacial Till with the average mine time increasing from 18 to 24 minutes. Between Stn +500m and Stn. +450m, Ring 195, the average mine time increased from 24 to 42 minutes. After this stage, the average mine time spikes between Stn. +435m and Stn. +415m. These spikes marked the initial contact with significant quantities of Bedrock. Mining times in this region were in the order of one hour at the highest concentration of Bedrock. After Stn. +415m, the average mine time decreased to approximately 32 minutes. This mine time was maintained to the end of construction. It should be noted that the mining times stated are ideal times, a point which will be discussed in detail after the presentation of Figure 5-6 (p 156).

Figure 5-6 (p 156) details the mining time per ring of advance for the construction of the Northbound Tunnel, contrasting it with the fraction of the TBM spoil passing the #60 and #200 sieve. Figure 5-8 (p 157) presents the excavation time per ring of advance, but contrasts it with the inferred stratigraphy for the Northbound alignment. A trend line with a moving average of four was used to present the average mine time in both Figures.

Mine times during the initial phases of construction prior to Stn. +620m, Ring 30, were greater than 48 minutes. Between Stn. +620m and +600m, Ring 46, the average mine time decreased from 48 to 18 minutes. Like the Southbound Tunnel, this region corresponded to the first significant contact with Glacial Till. Mining times in the order of 18 minutes were maintained from this point to Stn. +455m, Ring 169. After Stn. +455m to Stn. +425m, Ring 194, the average mine time increased to 24 minutes in a region of intra-till sands. The spike visible in this region at Stn. +435m was the result of technical difficulties that stemmed from the welds on the tail shield breaking after the tail shield was grouted in place. After Stn. +425m contact with significant quantities of Bedrock increased the average mine time from 24 minutes to 48 minutes.

In the discussion of Figures 5-5 (p 155) and 5-6 (p 156), no attention was given to the occurrence of spikes in average mining time during the construction of the South and Northbound Tunnels. Figures 5-9 and 5-10 (p 158) contain elements of the previous series of Figures, showing the average mining times per ring of advance, and the measured spoil moisture content for the Southbound and Northbound Tunnels respectively.

During the construction of the Southbound Tunnel, mine time spikes were observed at a number of points, Stn. +630m, +610, +560m, +540m, +515m, +490m, and +425m. Of all the observed spikes in mine times, only at Stn. +495m, +460m, and +425m do they bear any relationship to the soil conditioning in terms of moisture content. Only in two of the three cases does the spike in mining time correspond to a significant change in geology: the interception of significant quantities of Bedrock at Stn. +495m and +425m. In the case of Stn. +460, an increase in mining time occurred in a region of extensive intra-till sands where production should have improved.

Of the eight major process upsets identified during Southbound construction a correlation to changing geologic (Figure 5-5, p 155) or moisture content (Figure 5-9, p 159) conditions can be observed at Stn. +630m, +610m, +560m, +540m, +515m, +490m, +460m, and +425m. Only at Stn. +495m, +460m, and +425m did the increases in mining time correspond to significant increases in the spoil moisture content. In addition, two of those three regions identified, corresponded to regions of significant change in geology: contact with significant quantities of Bedrock at Stn. +495m and +425m. Only at Stn. +460m did an increase in mining time correspond to an upset in the soil conditioning in ground conditions that were becoming more favourable (region of intra-till sands). Of the remaining increases in mining time, moisture content spikes can be seen as a factor in only two of them: Stn. +630m and +540m.

In the cases of Stn. +610m, +560m, and +515m, geologic change drove the increase in mining times. Soil conditioning in the excavation chamber was even and was not upset, and no mechanical breakdowns with the Tunnel Boring Machine were observed. As a result the increases in mining time in the region of Stn. +610m, +560m, and +515m can reasonably be attributed to the change in geology. Stn. +610m correspond to roughly 50% of the TBM face being Glacial Till, Stn.+560m and +515m regions where the TBM face transitioned to 100% Glacial Till.

Figure 5-10 (p 158) presents the average mine time for the Northbound Tunnel with TBM spoil moisture contents. There are only two significant increases in moisture content and mining time during the construction. At Stn. +475m, increases were observed in both mining time and spoil moisture content. However, this region, Stn. +475m, corresponds to a significant change in geology. Only at Stn. +440m is there an occurrence of a spoil moisture content spike accompanied with an increase in mining time with no significant geologic change.

In this section, the daily productivity of construction and the per ring excavation times have been examined. In viewing the daily productivity and mining times recorded during Southbound construction, a number of key points can be gleaned. The first is the need to gain direct experience with the local geology. The mining time spikes observed during the construction of the Southbound Tunnel were in large part a function of geology. Process upset involving moisture content played a significant factor in only three of the eight spikes. During the construction of the Northbound Tunnel, the problems that were encountered during Southbound construction were avoided. However, in looking closely at the mining times obtained during the construction of both tunnels, process upsets aside, the mining times are remarkably similar. Establishing an expected mining time within the Outwash Sands is made difficult because the TBM was operating with a limited gantry and train. From limited Northbound construction, 30 minutes per ring is a tentative estimate for operation within the Outwash Sands and Silts. When operating within mixed face Outwash Sands and Silts and Glacial Till, mine times of between 18 and 24 minutes per ring were obtained. Construction within the Glacial Tills in both tunnels was reliably between 15 and 20 minutes per ring. Construction within the mixed conditions of Glacial Till, intra-till sands, and Bedrock was between 30 and 50 minutes per ring.

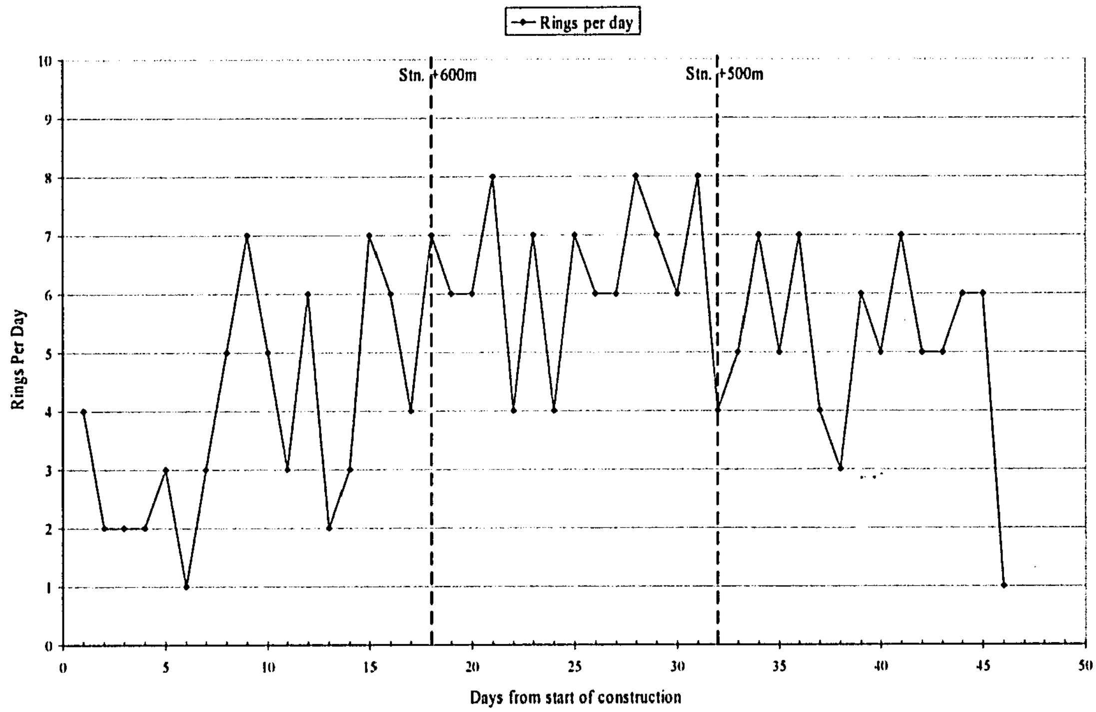


Figure 5-3 Rings constructed per day during construction of the Southbound Tunnel.

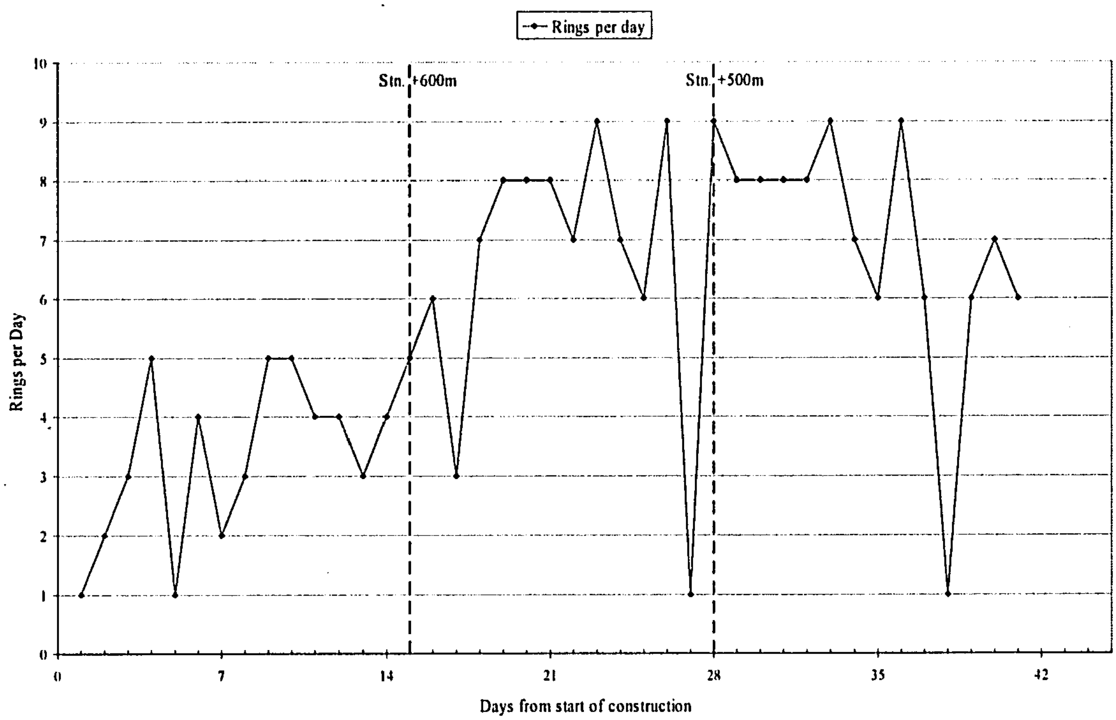


Figure 5-4 Rings constructed per day during construction of the Northbound Tunnel.

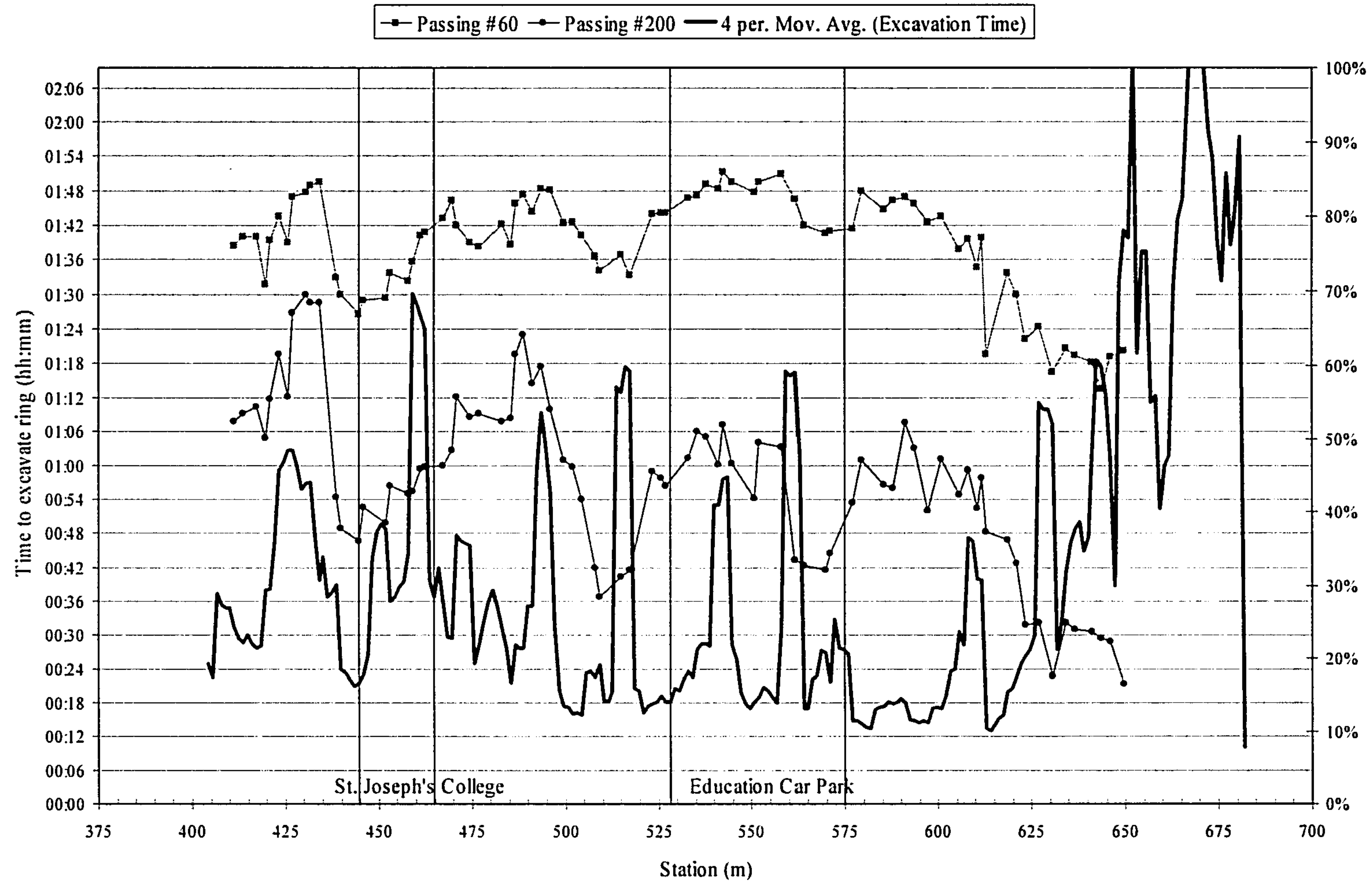


Figure 5-5 Excavation time per ring for construction of Southbound Tunnel with the fraction of soil passing the #60 and #200 sieve.

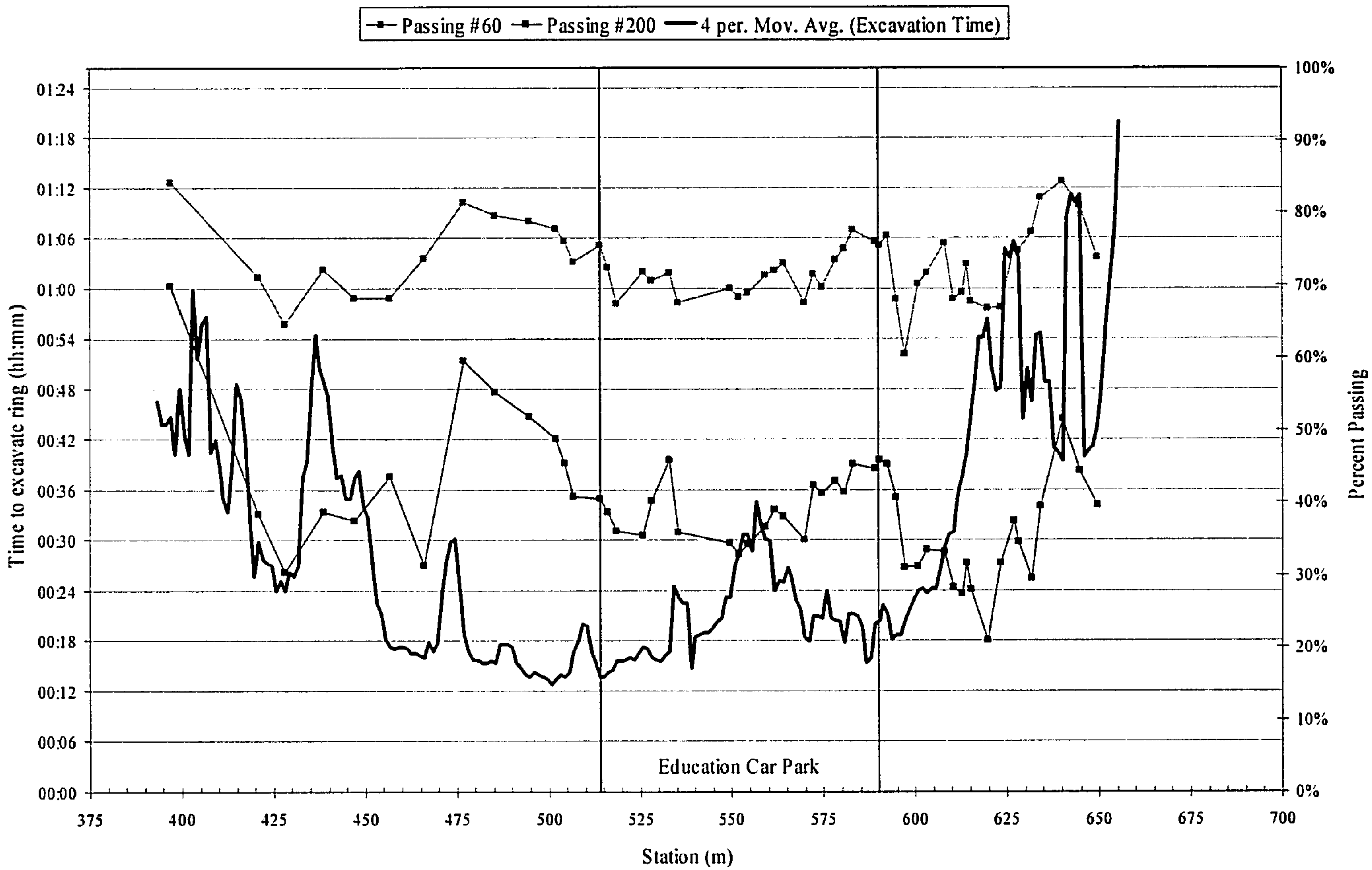


Figure 5-6 Excavation time per ring for construction of Northbound Tunnel with the fraction of TBM spoil passing the #60 and #200 sieve.

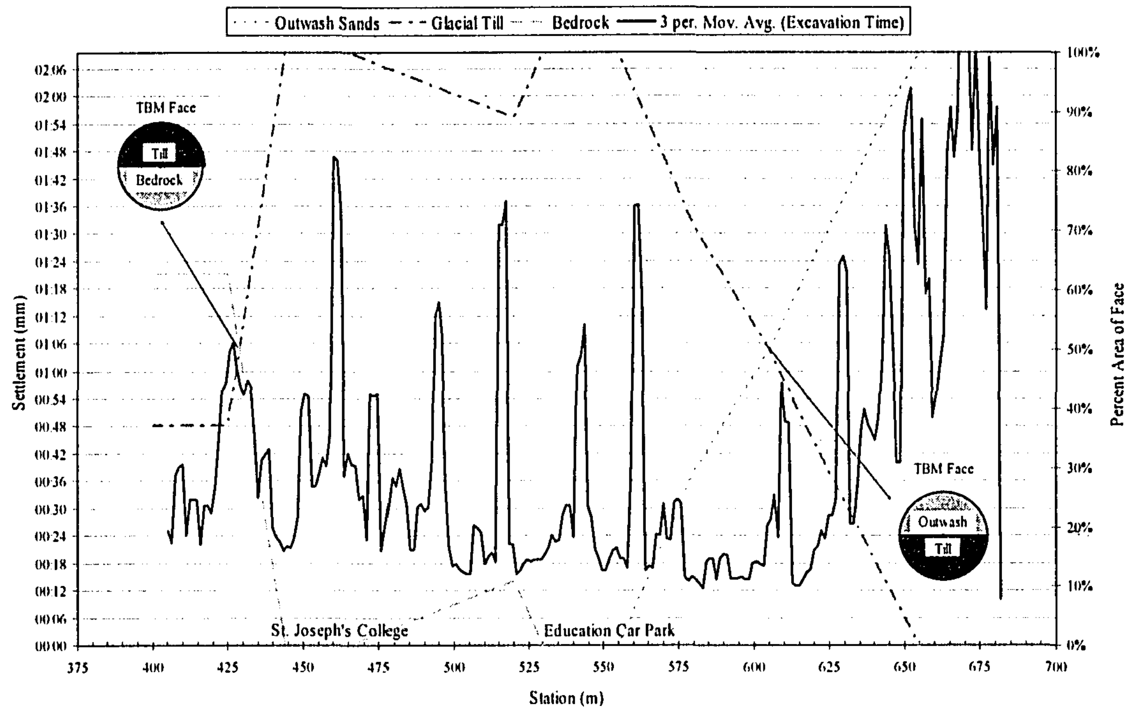


Figure 5-7 Excavation time per ring for construction of Southbound Tunnel with inferred

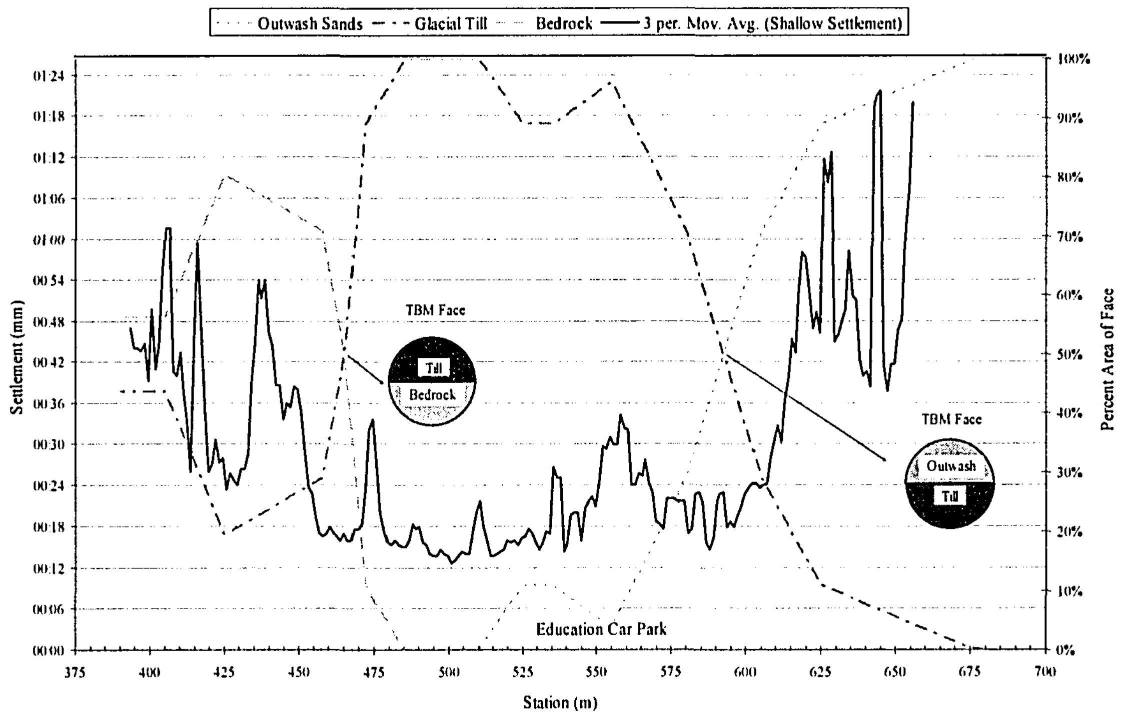


Figure 5-8 Excavation time per ring for construction of Northbound Tunnel with inferred geology.

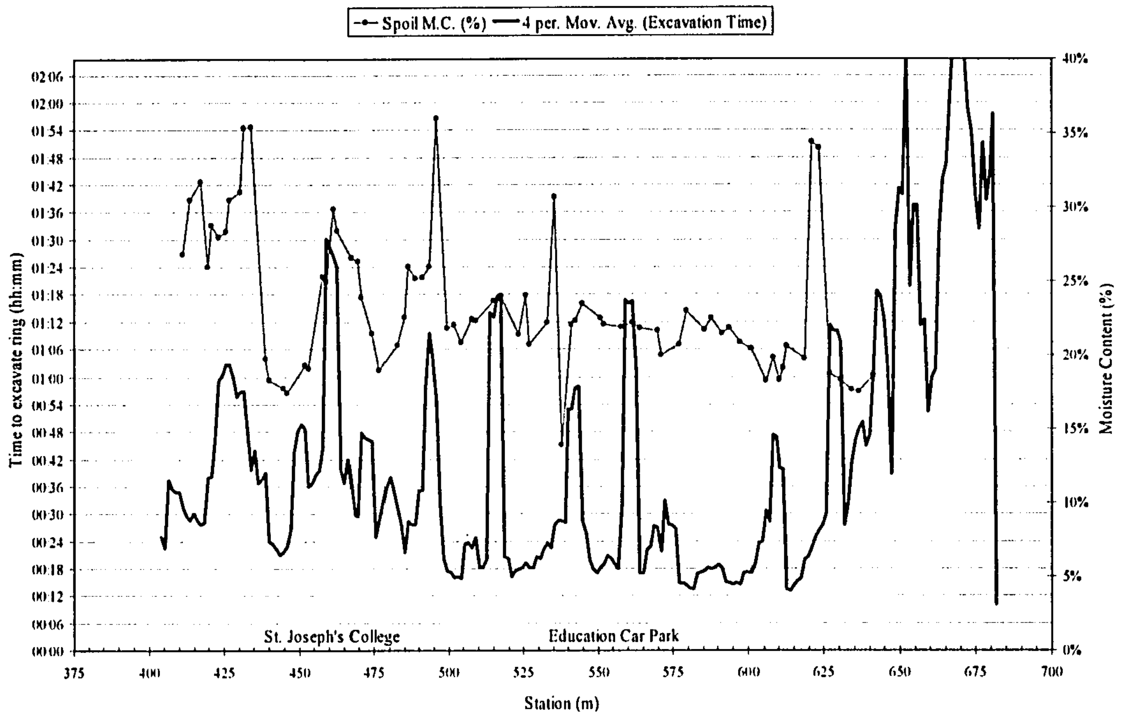


Figure 5-9 Correlation between process upsets as shown by TBM spoil moisture content and average mine times during Southbound construction.

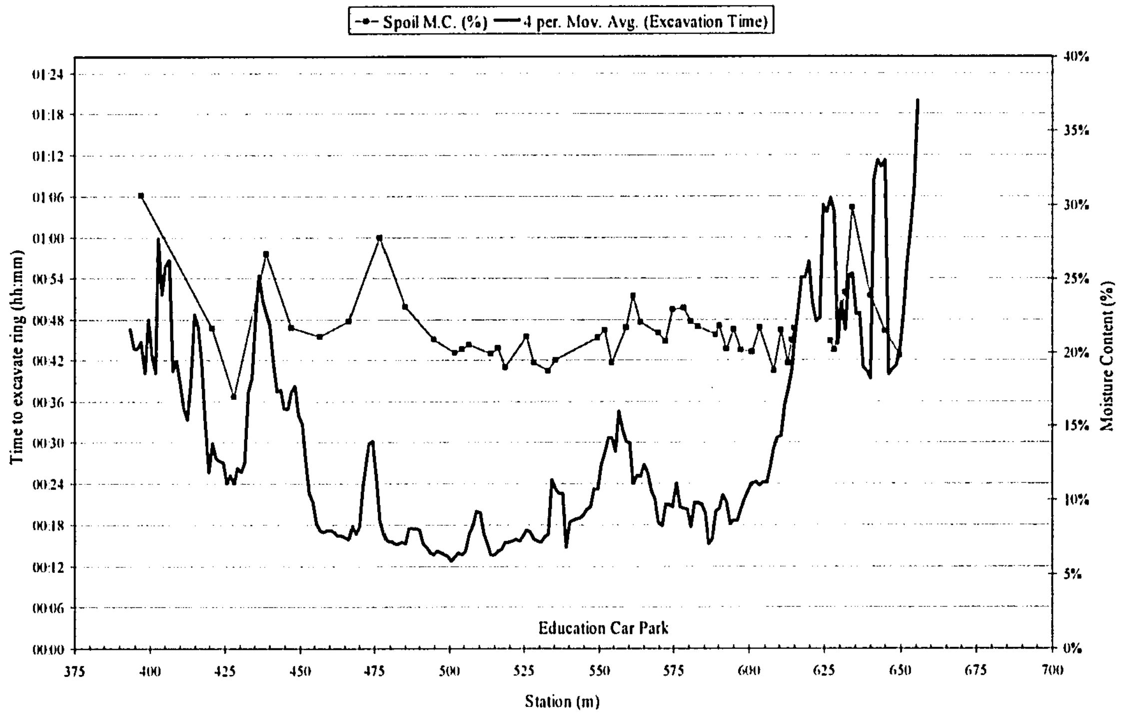


Figure 5-10 Correlation between process upsets as shown by TBM spoil moisture content and average mine times during Northbound construction.

5.1.3 Operational Torque of TBM in Changing Ground Conditions

Figure 5-12 (p 160) presents the mean operational torque recorded during the construction of the Southbound Tunnel, in addition to the fraction of TBM spoil passing a #200 sieve. The mean operational torque was determined by identifying all of the torque values recorded while the cutting head was in motion. A correlation was then sought between the fines fraction of the TBM spoil, and the mean operational torque (Figure 5.11, p 154). The best fit interpolation made use of a linear interpolation and obtained an R^2 value of 0.31.

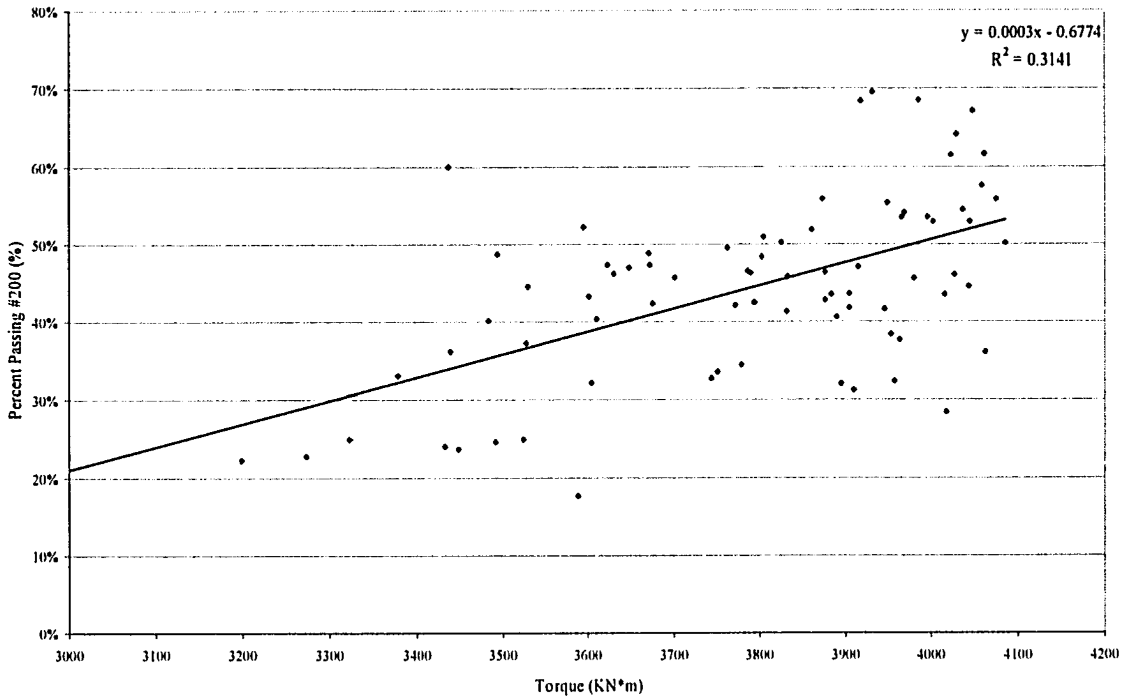


Figure 5-11 Correlation between percent passing #200 and mean operational torque for the construction of the Southbound Tunnel.

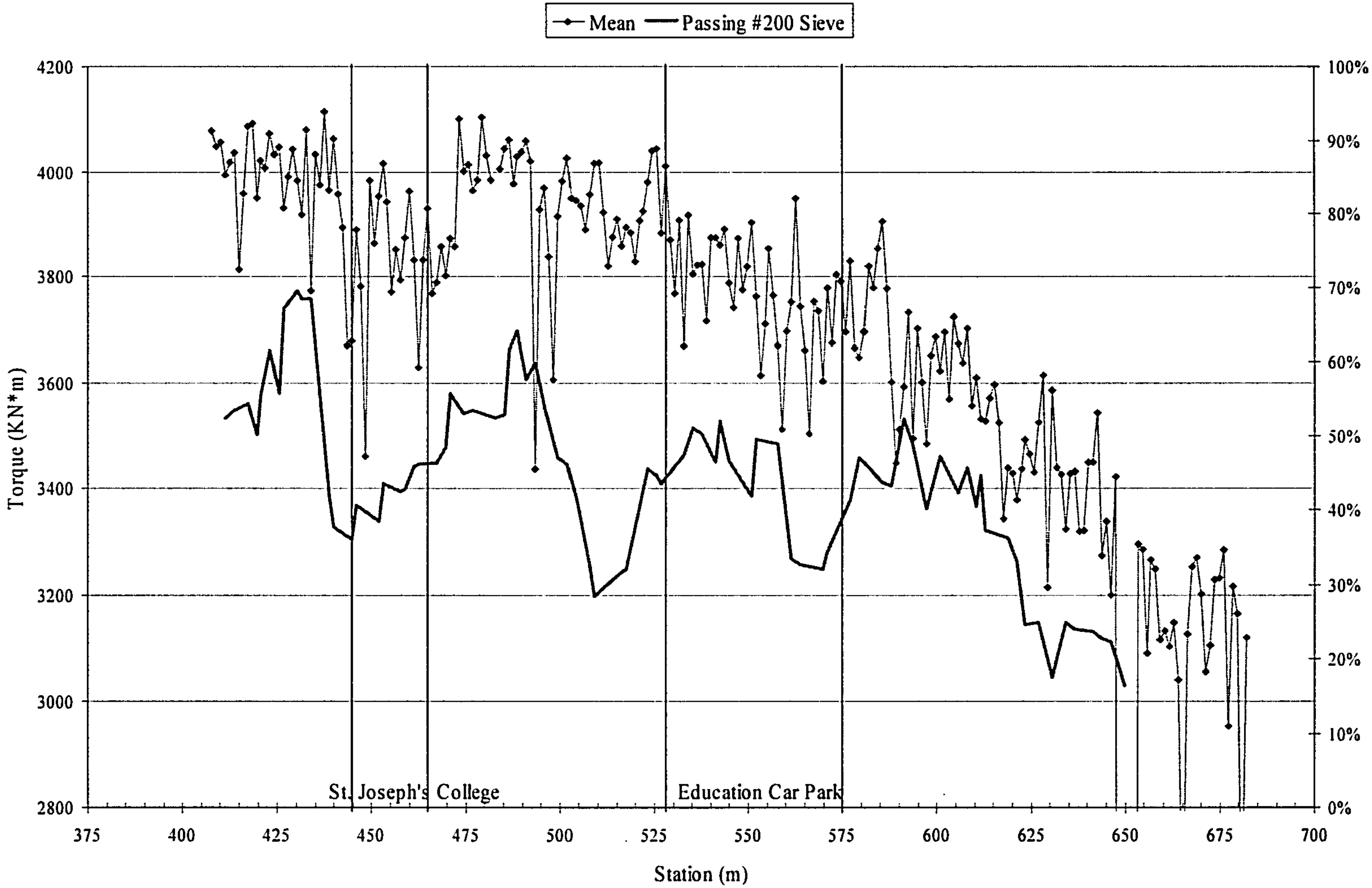


Figure 5-12 Mean operational torque during construction of the Southbound Tunnel.

5.2 Settlement Measurements as a Measure of Performance

In this Section, the settlements observed during the construction of the Southbound and Northbound Tunnel will be evaluated with respect to the performance of the TBM and the local geology.

5.2.1 Face Control and Subsequent Ground Loss as a Source of Settlement.

The development of EPB-TBM technology was driven by concerns over face control. As settlements ahead of the face of the TBM are a function of over-mining during advance, the best measure of the effectiveness of face control is the observed settlements. The effectiveness of the face control during the construction of the Southbound and Northbound Tunnels can most clearly be seen in Figure 4-28 (p 112) and Figure 4.31 (p116). These Figures detail the shallow settlements in the region from the portal to the Education Car Park. It is in this region that the Outwash Sands and Silts occupy the largest proportion of the face area and posed the greatest risk for construction induced settlements due to over-mining.

During Southbound construction, the largest observed settlement occurring prior to the TBM face was -3mm at instrument SS101, the first settlement gauge. All other instruments in this region reported less than -2mm of settlement, prior to the TBM face passing beneath. In comparison, the total settlements were in the order of -10 to -18mm. During Northbound construction, no observable settlement was recorded prior to the TBM passing beneath the settlement gauges. The lack of settlement prior to the passage of the face of the TBM indicates that face control achieved using an EPB-TBM was excellent with little to no over-mining during advance.

5.2.2 Grout Injection as a Mitigating Factor to Settlement

Figure 5-13 (p 164) presents the injected grout volume for each ring of advance during Southbound construction, and contrasts it with the observed settlements. Figure 5-14 (p 165) relates the injected grout pressure for each ring of advance to the observed settlements. During the initial advance, prior to the Education Car Park, injected grout volumes were on the order of 3.3 m^3 for each ring of advance. The theoretical volume of grout needed to compensate for the over-cut was 3.2m^3. As the Southbound Tunnel neared the Education Car Park, the concern over the magnitude of the settlements observed and their effect on the Education Car Park increased. This concern led to the amendment of the grouting procedures and the implementation of a new grout design.

The initial grouting procedures called for the required volume of grout to be injected be at least 3.2 m^3, to compensate for the over-cut with the grout injection pressure at the point of entry to be 206 KPA. This specification was revised to call for the manual adjustment of grout injection rate to correspond with the advance rate. In addition, the maximum allowable pressure at the point of grout injection was increased to 412 KPA. The effect of these revisions was to increase drastically the volume of grout injected per ring of advance from ~3.25 m^3 to ~4.25 m^3. When construction of the Northbound Tunnel began in January of 2004 the grout injection volumes were held at an average of ~4.25 m^3 per ring of advance for much of construction.

When construction of the Northbound Tunnel began in January of 2004, the impact of the grouting procedures on settlements became apparent. Figures 5-15 and 5-16 (p 165) present the injected grout volume and pressures per ring of advance on the Northbound Tunnel, in addition to the observed shallow settlements. The settlements induced by the construction of the Southbound Tunnel have been subtracted from Figures 5-15 and 5-16.

During construction of the Southbound Tunnel, settlements in the region prior to the Education Car Park were between -12mm and -20mm, with some -2mm attributable to face loss. The result is that between -10mm and -18mm of settlement was directly attributable to the transition between the tail shield and the segmental lining. During Northbound construction, the ultimate settlements in this region were between -25mm and -27mm at the first three settlement gauges, and between -10mm and -14mm for the next three settlement gauges. If the settlement induced by the construction of the

Southbound Tunnel is considered, the settlement induced by the construction of the Northbound Tunnel is between -6mm and -10mm. In making the previous statement the initial three instruments have been removed from this finding as grouting had not yet begun for the initial two, and only just had begun when it passed beneath the third.

Whereas Southbound construction produced between -10mm and -18mm of settlement at the end of the tail shield, Northbound construction reduced this settlement to between -6mm and -10mm. An improvement occurred in observed settlements in the order of 40%, when tunnelling within the Outwash Sands and Silts with some Glacial Till content. This improvement can be attributed to the improved grouting procedures implemented during the construction of the Southbound Tunnel.

In the case of construction after passing under the Education Car Park the settlements observed at the tail shield transition were comparable between the two tunnels: in the order of -5mm or less, and between -6mm and -10mm for the final two instruments prior to breakthrough. The large settlement observed at Stn. +460m during Northbound construction was the result of the loss of the excavation face at that location when a large, isolated, pocket of water charged, intra-till sands was encountered.

The construction of the Northbound and Southbound Tunnels produced comparable orders of settlement when using the revised grouting standard. However, in the region prior to reaching the Education Car Park, improvements in grouting procedure between the Southbound and Northbound Tunnels resulted in a reduction in observed settlements in the order of 40%.

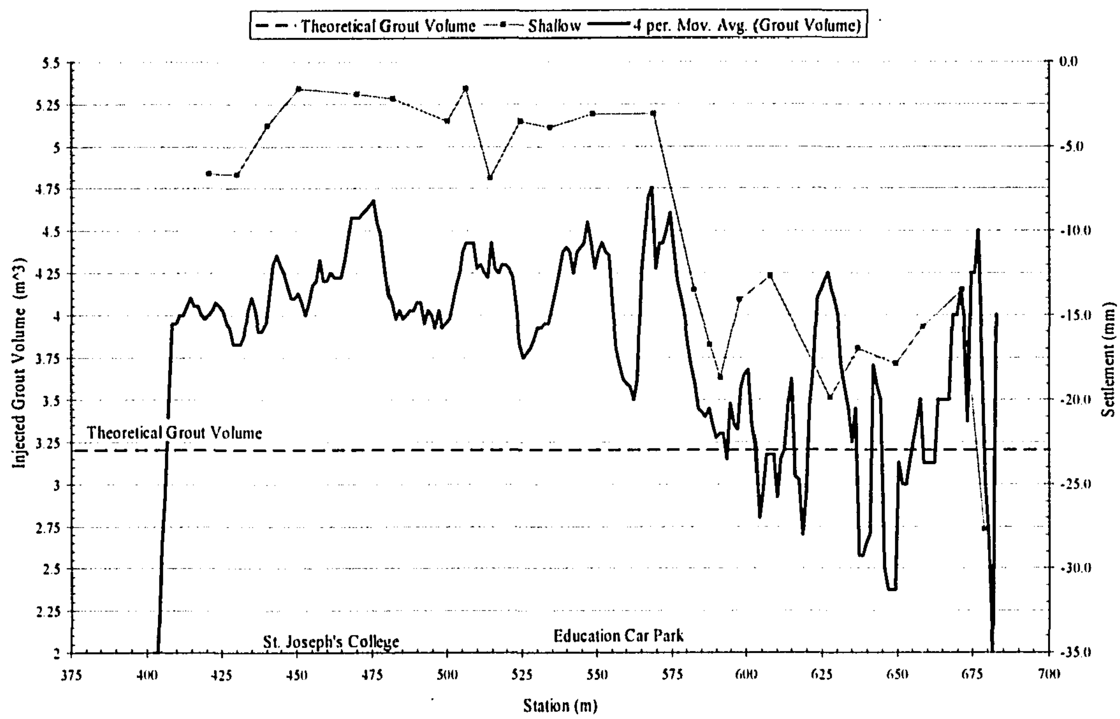


Figure 5-13 Injected grout volume per ring of advance during Southbound construction contrasted with observed shallow settlements.

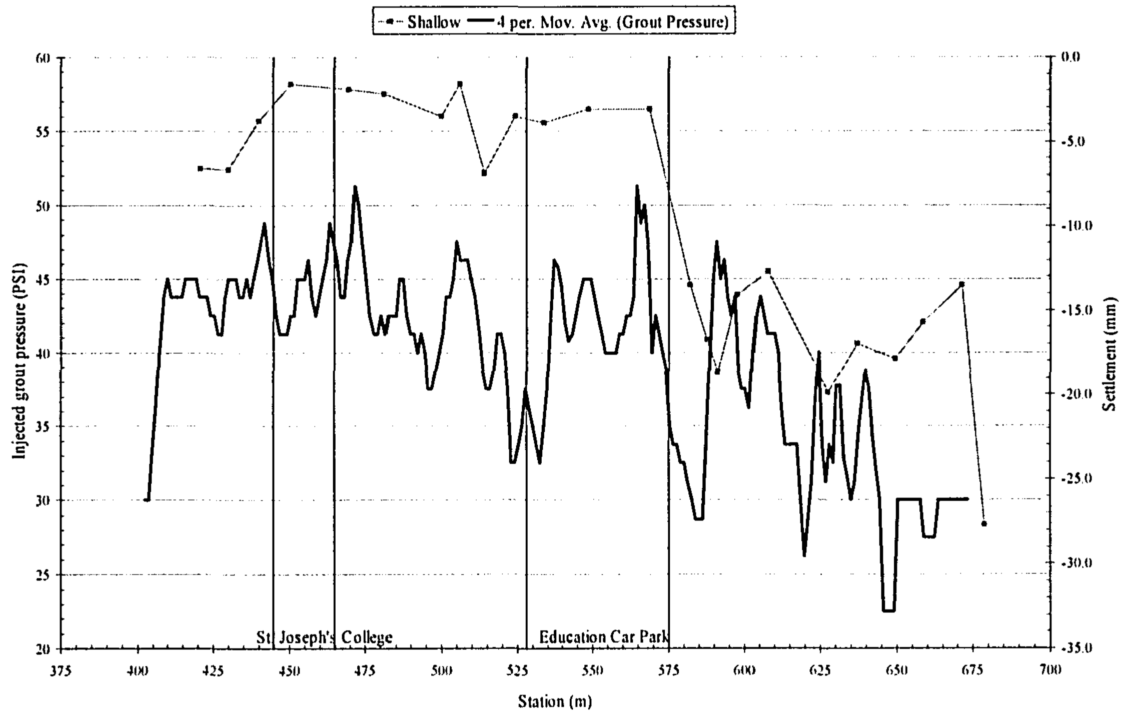


Figure 5-14 Injected grout pressure per ring of advance during Southbound construction contrasted with observed shallow settlements.

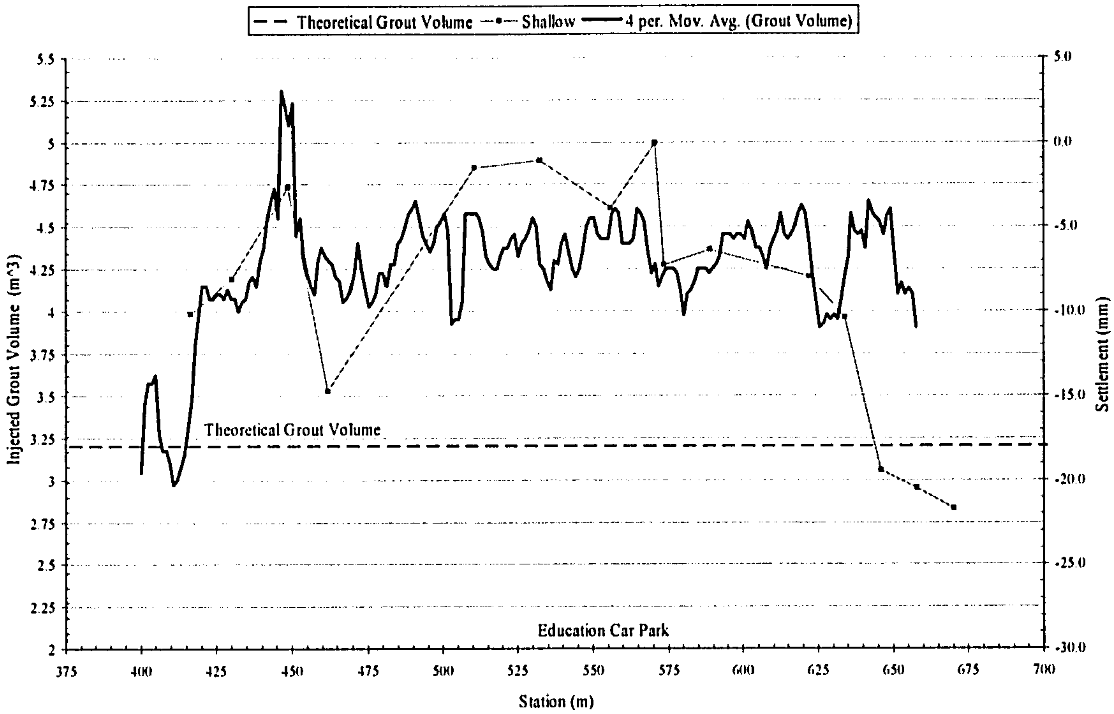


Figure 5-15 Injected grout volume per ring of advance during Northbound construction contrasted with observed shallow settlements.

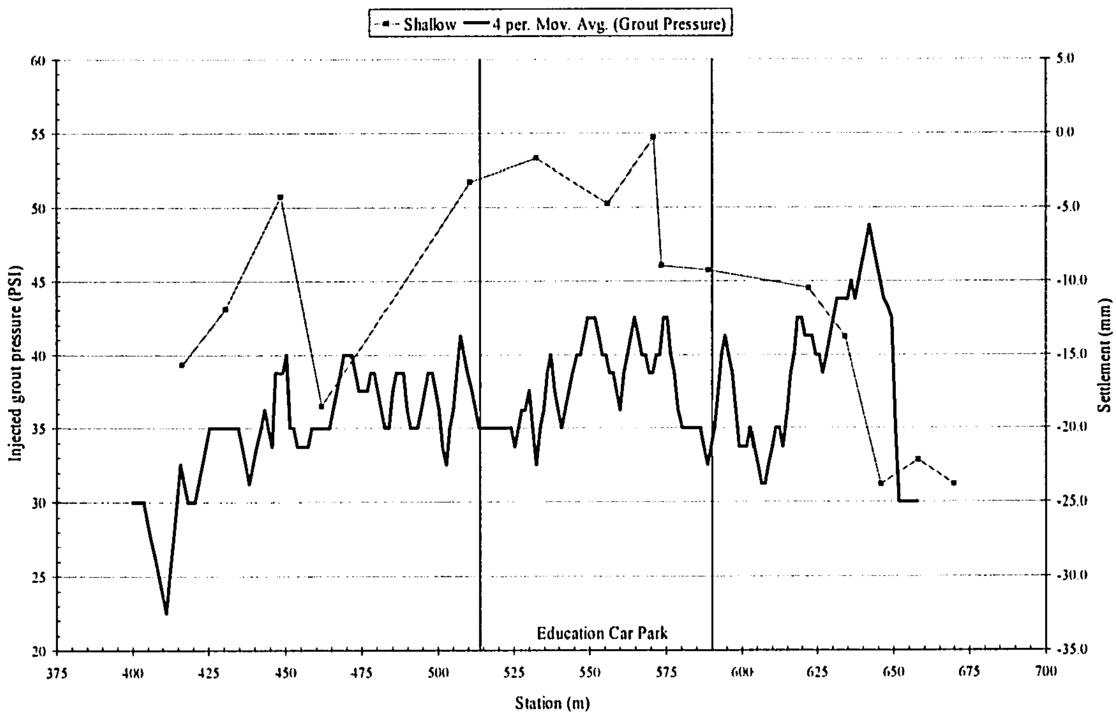


Figure 5-16 Injected grout pressure per ring of advance during Northbound construction contrasted with observed shallow settlements.

5.2.3 Geology as a Factor in Settlement

Initial construction of the Edmonton SLRT began in the Outwash Sands and Silts and over the course of the alignment transition, moved to competent Glacial Till and Bedrock.

In Figure 5-17 (p 168) the observed shallow settlements for the Southbound alignment can be observed, and are contrasted with inferred stratigraphy along the alignment. Figure 5-18 (p 168) shows the same settlement information, but contrasts it with the fraction of TBM spoil passing a #200 sieve. Construction after Stn. +575m reflects operations within competent ground with settlements of less than -5mm. Stn. +575m also marks the implementation of the revised grouting procedures.

However, in the initial region from the portal to the Education Car Park, the impact of geology can be observed. Observed shallow settlements in the initial six settlement gauges were in the order of -13mm to -20mm, with the fines content of the same region in the order of 25% or less. However, after Stn. +625m, increasing quantities of silt and traces of glacial till increased the observed fines content to some 40% by Stn. +615m. This region also shows a reduction in the observed settlement by some +7mm compared to that of the instruments located prior to Stn. +625m. Over the course of the subsequent two settlement points, the settlements reached a value of roughly -5mm which was then maintained for the duration of construction.

The response observed during construction of the Northbound Tunnel can be seen in Figure 5-19 and 5-20 (p 169) which present the same information as the Southbound plots. Settlement in the initial three gauges, are in the order of -20mm, and dramatically improve after Stn. +645m despite a general coarsening of the TBM spoil. Also, a fining trend after Stn. +600m produced little in the way of observable change in the settlement. However, in both these cases, the area of the face occupied by competent Glacial Till increased.

In considering the settlement during Southbound construction, the improvement in settlement can easily be seen to reflect changes in geology in the vicinity of Stn. +625m. However, the gains realised through improvements in geology, with little influence from grouting procedure, are in the order of +7mm. The dramatic gains shown at Stn. +575m have little to do with improvements in geology, but are owed to the improvements in grouting procedure, as discussed in the Section 5.2.2.

Initial contact with the Glacial Till was anticipated to be in the region of Stn. +650m during the construction of the Northbound Tunnel. While the grain size of the spoil reflects a coarsening trend in this region, this trend could be attributed to intra-till sands or a reduction of silt in the Outwash Sands and Silt. Between Stn. +650m and +600m, the observed settlements decrease from in the order of -20mm to -10mm. However, of the initial three settlement instruments, the first two were located in regions with no grouting. Consequently, the +10mm improvement in settlements between the third and fourth shallow settlement gauges is likely a function of grout injection and geology.

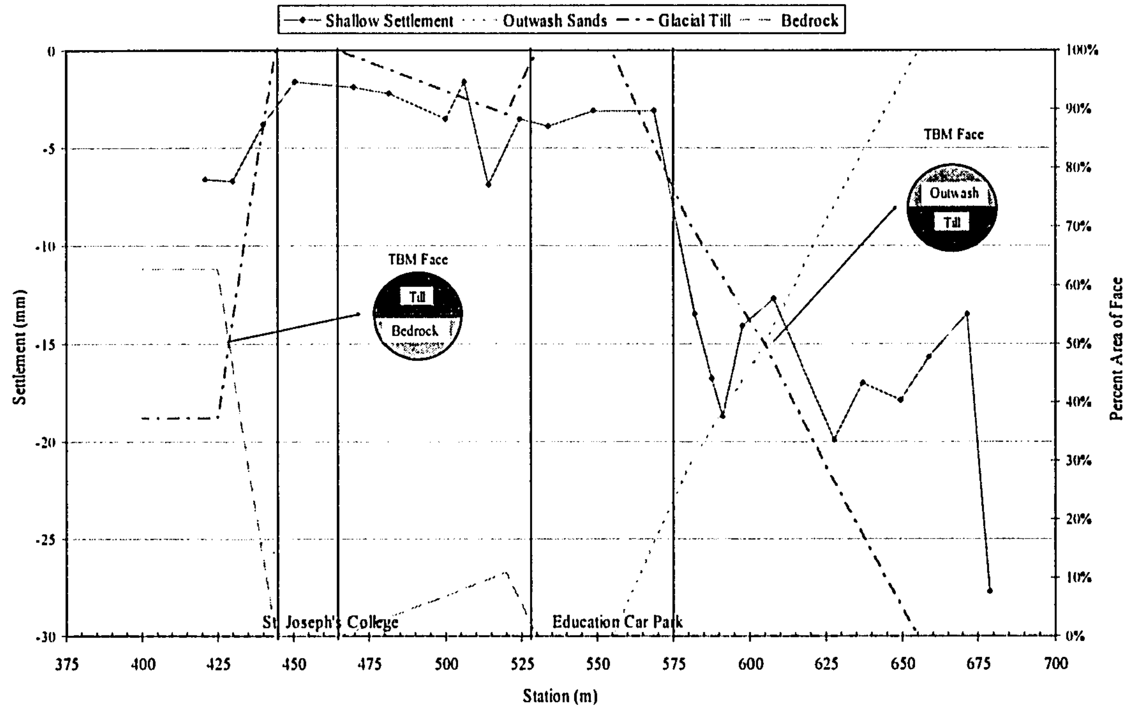


Figure 5-17 Observed shallow settlements during Southbound construction contrasted with inferred stratigraphy.

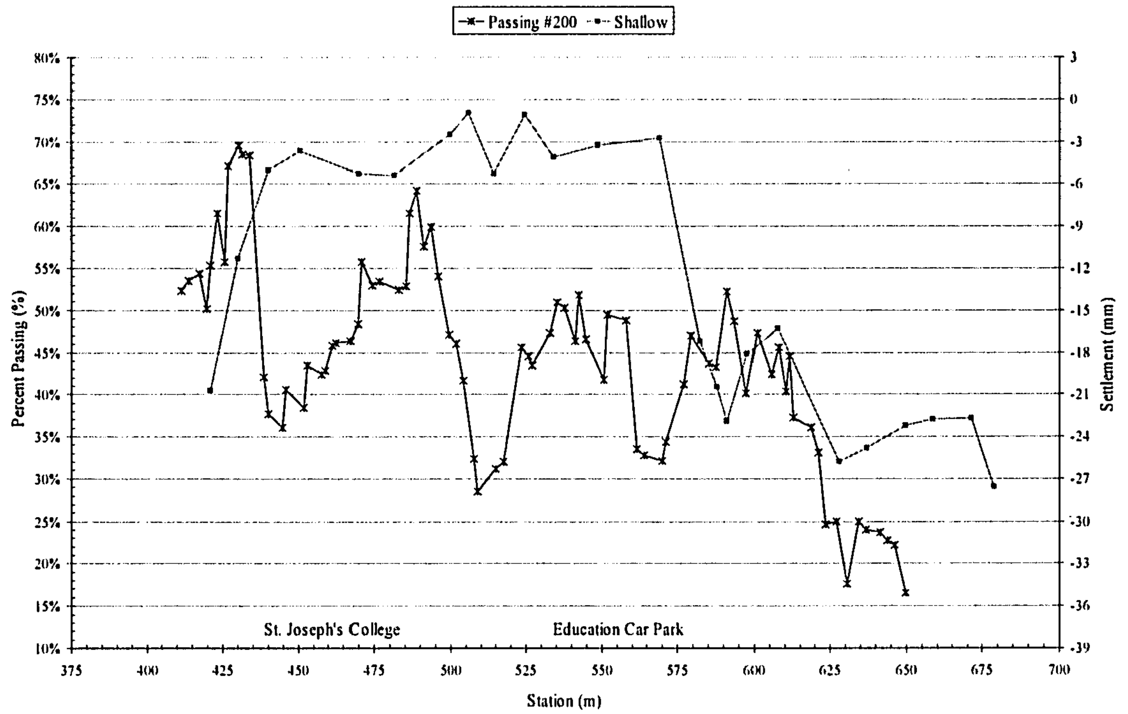


Figure 5-18 Observed shallow settlements during Southbound construction contrasted with grain size distribution of TBM spoil.

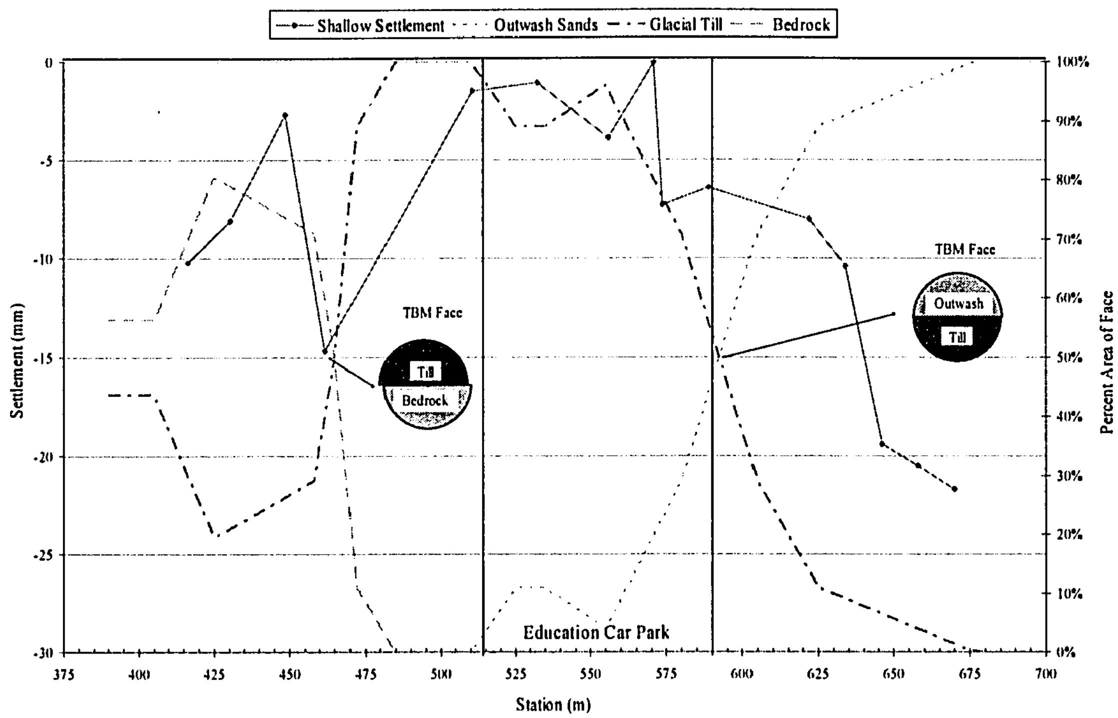


Figure 5-19 Observed shallow settlements during Northbound construction contrasted with the inferred stratigraphy.

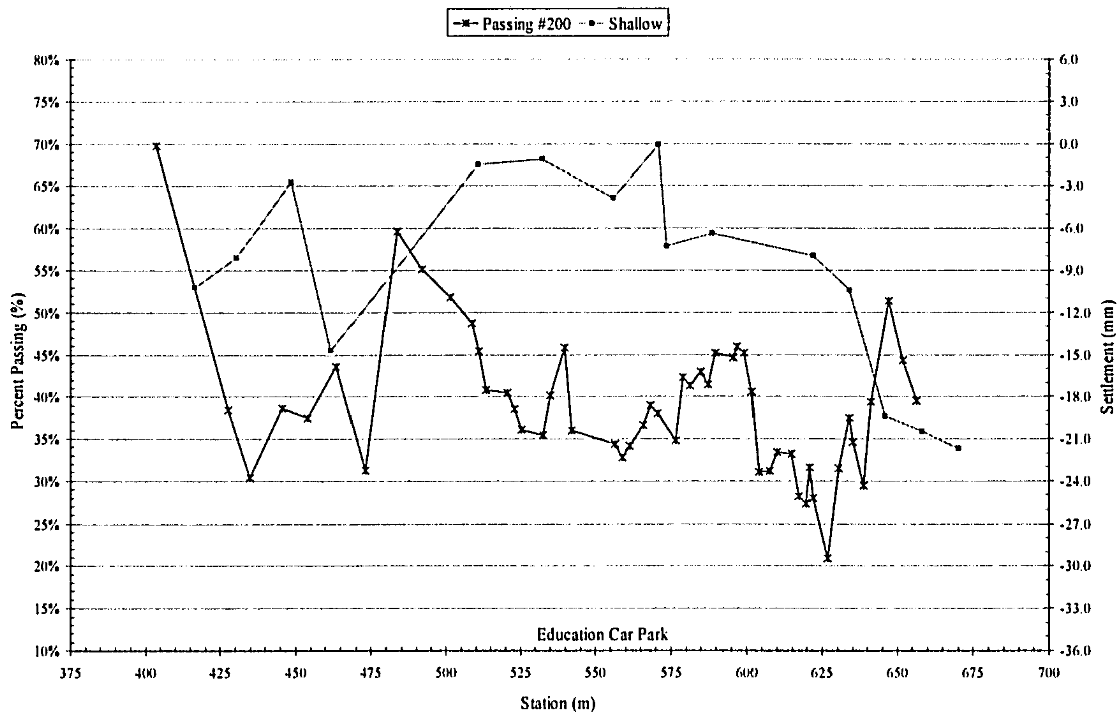


Figure 5-20 Observed shallow settlements during Northbound construction contrasted with grain size distribution of TBM spoil.

6.0 Conclusions

6.1 TBM Performance

6.1.1 Soil Conditioning

During the construction of the Southbound Tunnel, soil conditioning of the material excavated resulted in observed moisture contents in the order of 8% higher than the inferred in-situ moisture content. Construction after Stn. +430m, with tunnelling involving large quantities of Bedrock, the level of observed wetting was in the order of 15%. Construction of the Northbound Tunnel resulted in a similar level of wetting, with the increase in moisture content, in the order of 8% until Stn. +550m. Corresponding to the region where tunnelling full face within the Glacial Till was anticipated, the level of wetting was in the order of 4% for the remainder of construction, save for regions with large quantities of Bedrock. In the regions of high Bedrock content, between Stn. +500 and +475m and in the region of Stn. +400m, the observed increase in wetting was on the order of 7%.

6.1.2 Mining Times

The Southbound Tunnel was 300.4m in length, and completed in 46 days of active mining for an average daily productivity of 6.5 rings per day. The Northbound Tunnel was 271.4m in length, and completed in 41 days of active mining for an average of 6.6 rings per day.

Initial construction of the Southbound and Northbound Tunnels was hindered by the size of the construction site. Mining times in excess of one hour per ring (1.2m of advance) on the Southbound Tunnel, and in excess of 50 minutes on the Northbound Tunnel were common. However, once the TBM had advanced sufficiently, it was possible to use a full-sized gantry and muck train. This decreased the time needed to excavate one ring of advance. Table 6-1 (p 171) shows the typical mining times for the remainder of construction.

Table 6-1 Typical mining times for construction of Edmonton SLRT

Zone	Mining time per ring of advance
Outwash Sands and Silts	approximately 30 minutes
Mixed OSS and Till	18 to 24 minutes
Glacial Till	15 to 20 minutes
Bedrock	30 to 50 minutes

6.2 Control of Settlements

The selection of an Equalised Pressure Balance Tunnel Boring Machine for the construction of the Edmonton SLRT extension can be considered a success. The selection of EPB technology was based on the perceived need to control the settlements along the tunnel alignment. The initial construction of the Southbound Tunnel, from the Portal to Education Car Park, indicated that despite the selection of an EPB-TBM settlement along the alignment would be significant. However, these settlements were not due to over mining, which were very well controlled: but due to the differential between the tail shield and segmental lining diameter. Once the grouting procedure was revised to more effectively mitigate this source of settlement the overall settlement caused by construction was well within the specified boundaries.

EPB technology controlled the settlements it was designed to control resulting in little to no settlement due to over mining. While it may have been possible to build the tunnel using a traditional open faced TBM the likelihood of significant damage being inflicted on the Utility Corridor or Education during construction imposed a significant hazard cost which was not tolerable.

6.2.1 Face Control

Construction of the Edmonton SLRT extension using an Equalised Pressure Balance Tunnel Boring Machine has shown that settlements caused by over-mining are not a significant cause for concern. The level of face control maintained during the construction of the Southbound and Northbound tunnels was excellent. A level of control reflected in the small observed settlements ahead of the TBM face.

Shallow settlement instruments reported settlement ahead of the TBM face in only one region during construction: from the Portal to Education Car Park during Southbound construction. This settlement was in the order of -2mm. No settlement ahead of the TBM face was observed in the same region during Northbound construction with the shallow settlement instruments. Deep settlement instruments reported a maximum settlement of -3.2mm at DS101; all other instruments reported less than -1.5mm for the rest of construction (Southbound and Northbound).

6.2.2 Settlements at Tail Shield-Segmental Lining Transition

Construction of the Edmonton SLRT has shown that the tail shield-segmental lining must be considered as a potential source of significant settlement. In considering the potential risk two things must be considered. The first is the difference between the diameter of the tail shield and the diameter of the segmental lining, the greater the difference the greater the potential settlement. The second point to consider is the grout injection procedure used. Construction of the Edmonton SLRT extension has indicated that grouting procedures should not be based directly on theoretical void volume. Instead the injected grout volume should be in the order of 30% greater than the theoretical void created. Table 6-2 (p 173) presents the observed settlements at the tail shield-segmental lining transition during construction.

Table 6-2 Settlements at the tail shield-segmental lining transition

	From Portal to Education Car Park	Remainder of Construction
Southbound Tunnel	-10 to -18mm	< -5mm
Northbound Tunnel	-6 to -10mm	< -5mm

This improvement in observed settlement between the two tunnels was attributed to the improvements in grouting procedure implemented during the construction of the Southbound Tunnel prior to passing under the Education Car Park. Initial grout injection volumes were in the order of $3.25m^3$: only slightly larger than the theoretical void ($3.2m^3$ per ring of advance) created by the difference between the diameter of the tail shield and the segmental lining. Under the revised grouting procedure, grout injection volumes increased to ~$4.25m^3$ to ensure adequate filling of the void. These changes to the grouting procedure reduced the observed settlements approximately 40%.

6.2.3 Building Settlements

Three structures were directly undermined by the construction of the Edmonton SLRT extension. Two of the structures, the Utility Corridor and Education Car Park overlay a portion of the alignment that involved tunnelling within the Outwash Sands and Silts. St. Joseph's College was undermined while tunnelling occurred within the Glacial Till and Bedrock. The settlements observed did not exceed the criteria established in the construction documents to ensure no damage to the overlying structure. Table 6-3 (p 174) shows the maximum settlements acting on the structure at various stages in construction.

Table 6-3 Observed maximum building settlements during construction

		Southbound Construction	Northbound Construction
Utility Corridor			
	Point Over Southbound Tunnel	-13.3mm	-15.8mm
	Point Over Northbound Tunnel	-2.2mm	-9.1mm
Education Car Park			
	North Wall	-5.6mm	-5.9mm
	West Wall	-13.9mm	-20.5mm
St. Joseph's College		-2.1mm	-7.8mm

6.2.4 Influence of Geology

Construction of the Edmonton SLRT involved two very different zones of construction: the Outwash Sands and Silts which were viewed as major potential source of settlement, and the Glacial Till and Bedrock which were considered competent ground. A region of interest was in the transition from Outwash Sands and Silts to Glacial Till and Bedrock. During construction of the Southbound and Northbound Tunnels significant improvements in settlement began to be observed when Glacial Till occupied more than 50% of the TBM face area (transitioning from invert to crown). Table 6-4 (p 174) shows this improvement in settlement.

Table 6-4 Typical settlements in Outwash Sands and Silts and Glacial Till/Bedrock

	Southbound Tunnel	Northbound Tunnel
Outwash Sands and Silts	-18 to -24mm	-7 to -10mm
Glacial Till/Bedrock	~ -3mm	~ -3mm
Note: Southbound Tunnel settlements reflect original grouting procedure		

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Appendix A – Geologic History of the Edmonton Region

This Appendix contains a discussion of the geology of the Edmonton region.

Geological History of the Edmonton Region

The geology of the Edmonton region provides a window of some two billion years into the geologic processes that formed the Edmonton region. The modern City of Edmonton is located on a succession of depositional strata, which eventually finds its bottom on the buried margin of the Precambrian basement. The erosion of ancient mountain ranges, and millions of years as a shallow sea placed a mantle of some 2 to 3 km of erosional debris over modern Alberta. In recent geologic history, the characterisation of the Edmonton region is driven by the Wisconsin glaciation. Burying the region under more than a kilometer of ice, the advance and retreat of the Wisconsin glaciation dramatically reformed the region in less than 20,000 years.

Pre Glacial History of the Edmonton Region

The oldest geological stratum in the Edmonton Region is the Precambrian Basement that can be found at a depth of some 2 km, and is the buried margin of the Canadian Shield. The deposition of the basement took place between 3.5 and 2.5 billion years ago and was composed of sedimentary and volcanic rocks. Between 2.4 and 1.7 billion years ago, the deposits were metamorphosed after a collision of continental and oceanic plates drove the rocks down to a depth in the order of 10 to 20km. Under the great temperatures and pressures at this depth, the volcanic and sedimentary deposits metamorphosed into schists and gneisses. During the collision of the continental and oceanic plates, a series of mountain ranges formed over the Western Canadian Shield. After the relaxation of the crustal collisions that drove their formation around 1.7 billion years ago, river erosion worked to erode the Precambrian mountain range.

The erosion of the Precambrian mountain range led to a number of depositions that can be identified in Alberta. Deposits of large quantities of clastic debris were deposited by the rivers flowing west from the Precambrian mountains. The remains of this erosional event can be found in the Proterozoic Athabasca group which is made up of sandstones and conglomerates and is found only in the North-Eastern reaches of Alberta. It is likely that Proterozoic deposits covered much of the province, but were later eroded over much of Alberta. However in moving to the western reaches of Alberta, near Banff and Jasper, it is possible to find a Proterozoic stratum deposited under marine conditions between 1100 and 570 million years ago.

The Palaeozoic era which lasted between 570 and 225 million years ago, saw the continued erosion of the Precambrian mountain ranges with the erosional material carried into the Pacific Ocean. This period also witnessed a succession of flooding events which left much of Alberta in the form of a shallow sea. The first prominent flooding between 570 and 440 million years ago led to the formation of limestone and dolomite over a great period of time. However, most of the earlier Palaeozoic formations are restricted to the western reaches of Alberta in the Rocky Mountains. In the Edmonton region, and in much of the rest of the province, subsequent erosion eliminated the early Palaeozoic deposits.

A second period of flooding occurred between 380 and 360 million years ago and saw the deposition of further limestone and dolomite strata. In the Edmonton region it, is material from this period that overlies the Precambrian Basement. During the time between 360 and 245 million years ago, Alberta remained a shallow sea with brief periods of uplift. This final series of flooding covered the province with a thick mantle of marine deposits, which in the Edmonton region make up a significant portion of the 2km of sedimentary deposition. However, 245 million years ago, a major period of uplift saw much of Alberta become dry.

245 million years ago, much of Alberta became and remained dry, the ocean having been driven to the western portion of the province. During this time, large quantities of silts and clays were deposited into broad mud deltas along the coast. This gradual procession of the sea to the west 245 million years ago, continued with little interruption until 140 million years ago with the appearance of the volcanic Cordillera off the western coast of Alberta. As the volcanic chain of mountains grew, material was being deposited into Alberta from the eroding shield and the growing Cordillera. Eventually the sliver of

ocean that remained in Alberta became largely enclosed by the movement of the land, evolving into a narrow sea running from the southern US to the modern Arctic. The formation of the Cordillera also heralded the start of the second major formation of mountains in Alberta.

The formation of the volcanic Cordillera was tied to a growing collision of the Pacific plate with North America crustal plate. As this collision progressed, large sheets of older rock were thrust over younger rock and driven into the sedimentary formation of Alberta. With time, the collision saw the formation of the Rocky Mountains and foothills of western Alberta. With the rising of the mountains came a new bout of clastic erosion, in this case moving from the new mountains, east into Alberta. This period of erosion deposited large quantities of sands and gravels over Alberta. Remnants of this deposition, while not present in the Edmonton region, can be found in areas to the south and closer to the mountains such as the Edson and Red Deer regions.

Glacial History of the Edmonton Region

Since the formation of the Precambrian basement, much of the 'history' of Alberta had it located near the equator. Only after 300 million years ago did Alberta show a trend of progressively moving north. This movement over the course of 300 million years brought the Edmonton region near to its modern latitude of 53 degrees north.

Approximately two million years ago, a period of global cooling saw an increase in the size of the polar ice caps, they began to move their margins south over the northern areas of Europe, Russia, and N. America. At the same time, glaciers in mountainous regions of the world also began to advance down their valleys.

In the previous two million years, there have been over one dozen notable glacial advances into Canada. Between each of these major advances, as well as lesser advances, were short interglacial periods of relatively warm temperatures. Despite this repeated glaciation, it is thought that Alberta in large part escaped being covered in ice for much of this time. The most recent period of glaciation, the Wisconsin, began some 80,000 years ago and reached its peak some 23,000 years ago. At the height of the advance into Alberta, glaciers covered virtually the entire province. At that time, the Edmonton region was buried under some 1.5km of slowly moving ice.

As the glaciers advanced and moved over Alberta, they carried in with them vast quantities of mixed rock and spoil from the Rocky Mountains, Northern Alberta and places even farther removed. Their bases loaded with large boulders and debris, in addition to having the force of many hundreds of meters of ice above, glaciers scoured the land with immense force. Over most of Canada glaciation scoured the land to the bedrock carrying away surficial material, and sometimes rafts of bedrock, to be deposited elsewhere.

Another legacy of the period of glaciation is extensive deposits of glacial till over Western Canada. Glacial till is a heterogeneous mixture of fine and coarse sediments deposited with no apparent stratification along with the occasional boulder and sand lens. Tills are formed by several processes acting within the glacier. Till deposited under the ice sheet into lodgement areas are typically referred to as basal tills, or when accompanied with the melting of the base of the glacier, englacial tills. The final deposition process is that of ablative till, which is deposited by material spilling off the glacier. Also contained within many tills are numerous boulders, and water-charged sand deposits. In most regions with the older surficial materials stripped away by glacial action, the tills deposited by the Wisconsin glaciation were placed directly over bedrock or remaining pre-glacial deposits. In the Edmonton region, this movement of the glacier led to the deposition of thick till blankets over the bedrock and persisting preglacial sand and gravel deposits.

Post-Glacial History of the Edmonton Region

After the peak of the Wisconsin glaciation some 23,000 years ago, the slow retreat of the glaciers began. As the glaciers had advanced and retreated, blankets of till had been deposited over the local bedrock, and surviving preglacial deposits over much of Western Canada. The Edmonton region at this time was still buried under a considerable mass of ice, and it was not until roughly 12,000 years before the present time that the glacial ice front was situated over Edmonton.

As the Wisconsin ice sheet retreated to the north across western Canada, it acted as a broad dam against the movement of water. Much of western Canada is part of the Hudson's Bay basin and the region's large rivers flow to the east to meet with this body of water. Because of the presence of the ice sheet, the retreat across Western Canada was punctuated by a series of large glacial lakes that formed along the line of retreat. The largest of these, Glacial Lake Aggasiz, at its peak covered some 285,000 square kilometres and lasted as long as 4000 years, covering much of modern Manitoba. While Glacial Lake Edmonton covered a very modest area compare to Lake Aggasiz, and only endured for perhaps as long as 100 years, it played a significant role in the formation of the local topography. Figure A-1 provides an illustration of the extent of Glacial Lake Edmonton.

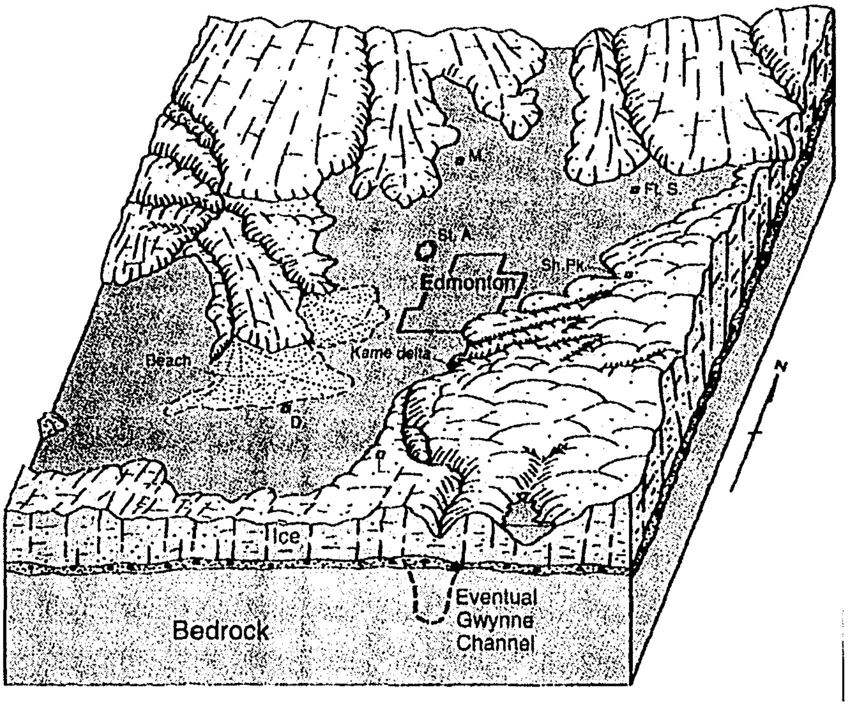

Figure A-1 Maximum extent of Glacial Lake Edmonton (Shaw, J., 1993)

As the front of the glacier melted and drainage from the ice sheet flowed into the Edmonton basin, large volumes of material were carried from the glacier. In areas close to outflows this movement led to the deposition of considerable volumes of sands and gravels. The most significant outflow in the Edmonton region is located some 25km west of Edmonton in the areas around Stony Plain. In addition to the outflow of sands and gravels, large quantities of fines were carried in the outflow and spread over and deposited under the forming Glacial Lake Edmonton. Consequently large areas of the Edmonton region are characterised by outwash sands and silt deposited in proximity to a glacial outflow, and occasionally form channels incised into the till layer. After the expansion of Glacial Lake Edmonton and the further recession of the ice front, depositions transitioned from outwash sands and silts to silts and clays. These silt-clay depositions are referred to as Lake Edmonton Clay and form a varved mantle overlying the outwash material and occasionally the till.

The failure of Glacial Lake Edmonton took place through, what is now, the Gwynne Outlet Channel. It was through this valley, some 1km wide and 50m deep much of Glacial Lake Edmonton drained roughly 12,000 years ago. From this breach, virtually all of Lake Edmonton drained to the south through the Battle River drainage system. After the failure of Glacial Lake Edmonton around 12,000 years ago, the ice front receded sufficiently to open a natural drainage course through the Edmonton region: the North Saskatchewan River. After the opening of the North Saskatchewan and its subsequent erosion and aggradation over the course of the next 12,000 years, the topography of the Edmonton reached its modern form.

Appendix B – Lake Edmonton Clay

This Appendix provides a series of Figures of charted logs from two boreholes and one CPT test in the region of the portal. These Figures are provided as an example of the soil classification and stratigraphic variation present. The logs were recorded by Golders.

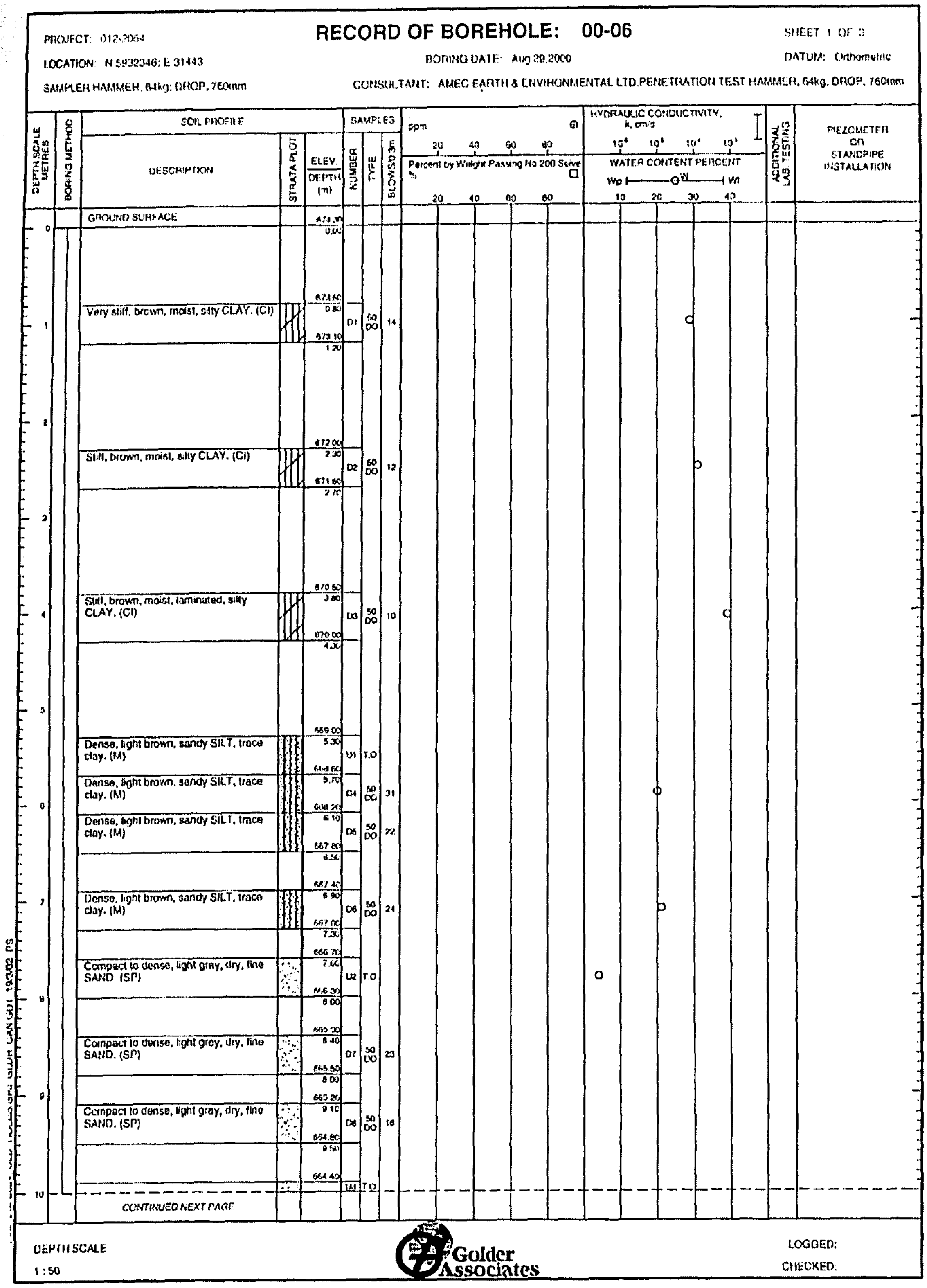

Figure B-1 A selection of the borehole log for Borehole 00-06 focusing on the Lake Edmonton Clay (AMEC)

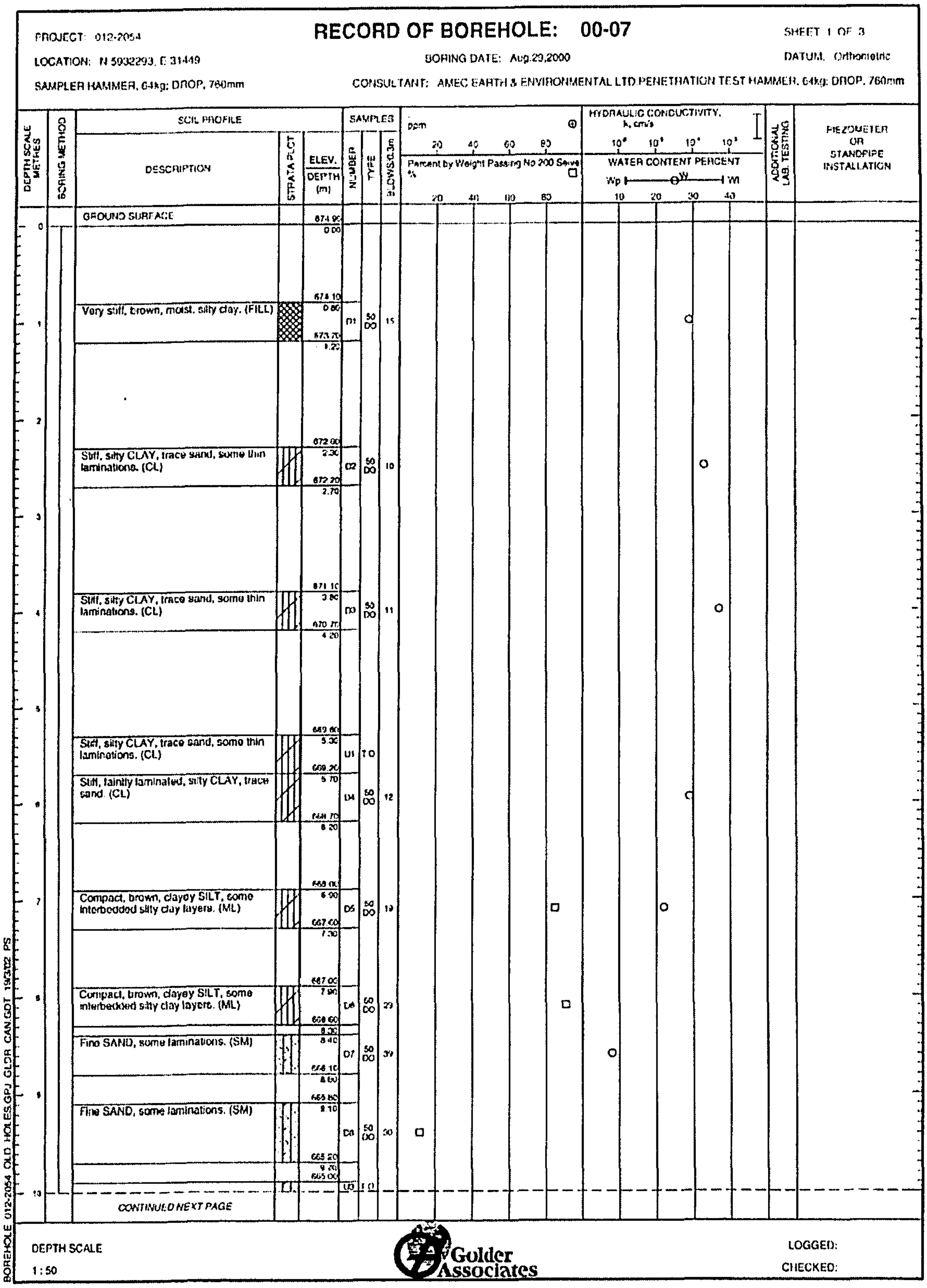

Figure B-2 A selection of the borehole log for Borehole 00-07 focusing on the Lake Edmonton Clay (AMEC).

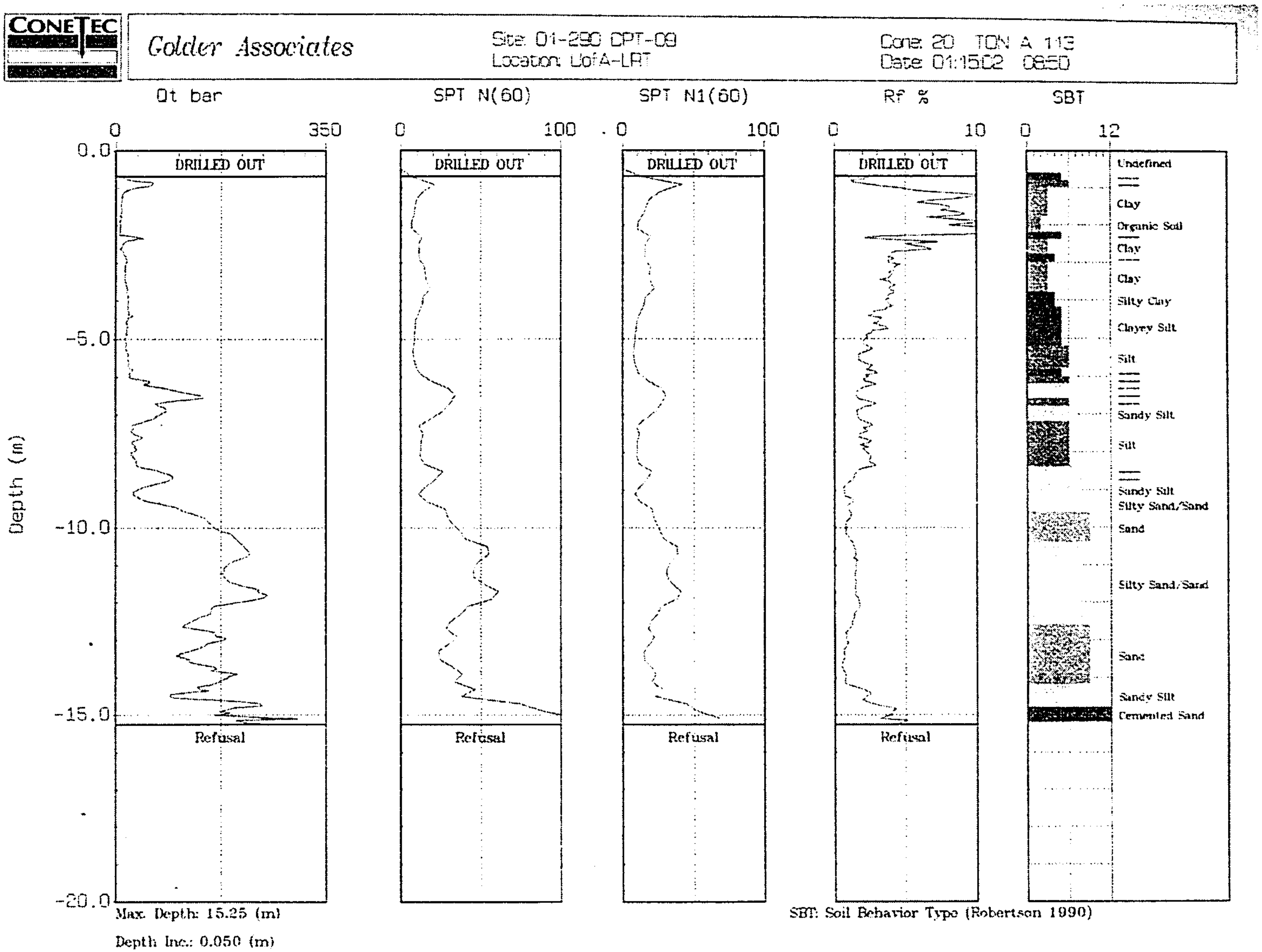

Figure B- Results obtained from CPT-09 (Golders).

Appendix C – Outwash Sands and Silts

This Appendix provides a series of Figures of logs from one borehole and three CPT tests in the region from the portal to the Education Car Park where the TBM face was predominantly Outwash Sands and Silts. These Figures are provided as examples of the soil classification, and stratigraphic variation present in the Outwash Sands and Silts. The logs were recorded by Golders.

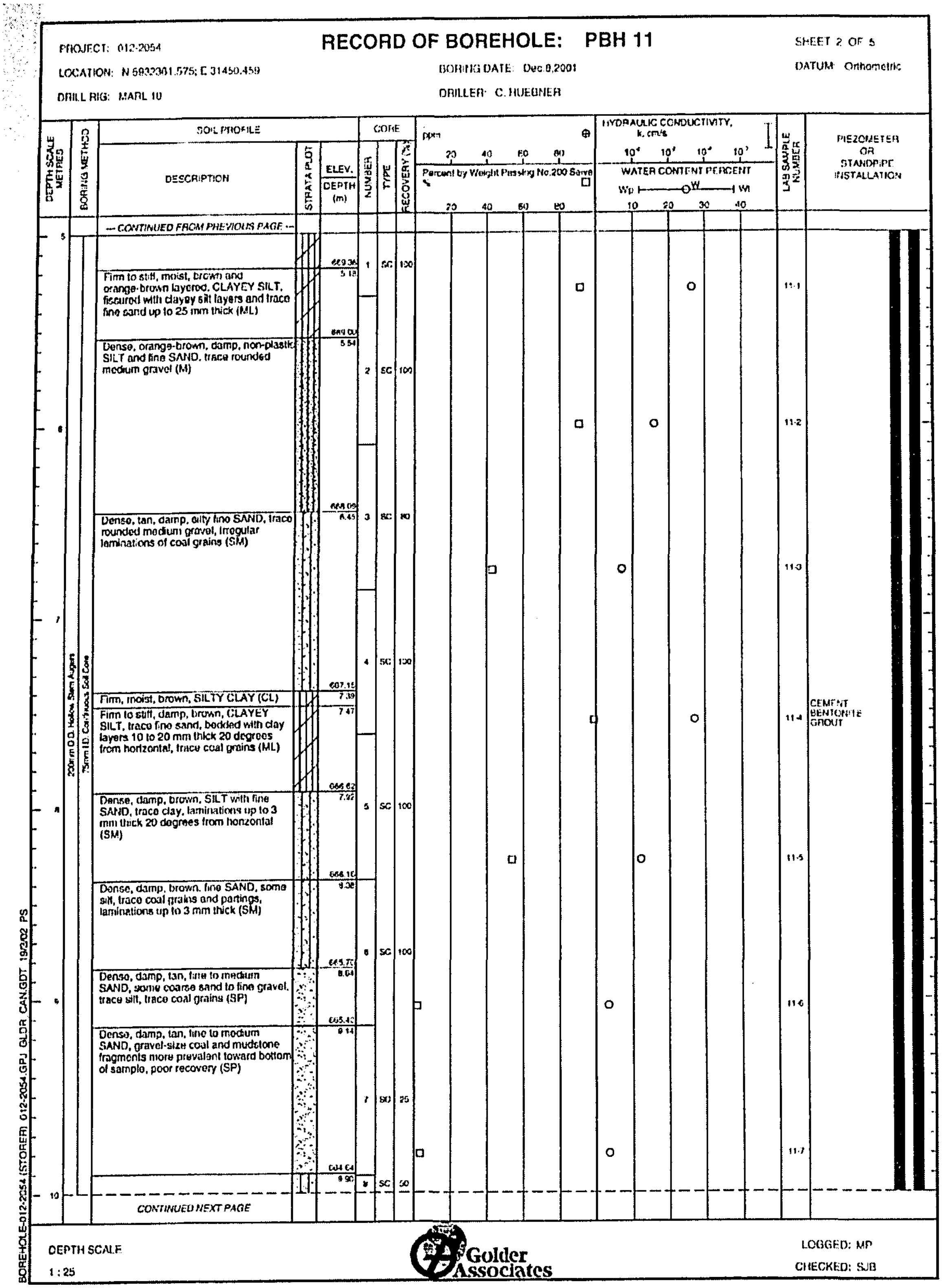

Figure C-1 A selection of the borehole log for Borehole PBH-11 focusing on the Outwash Sands and Silts (Golders).

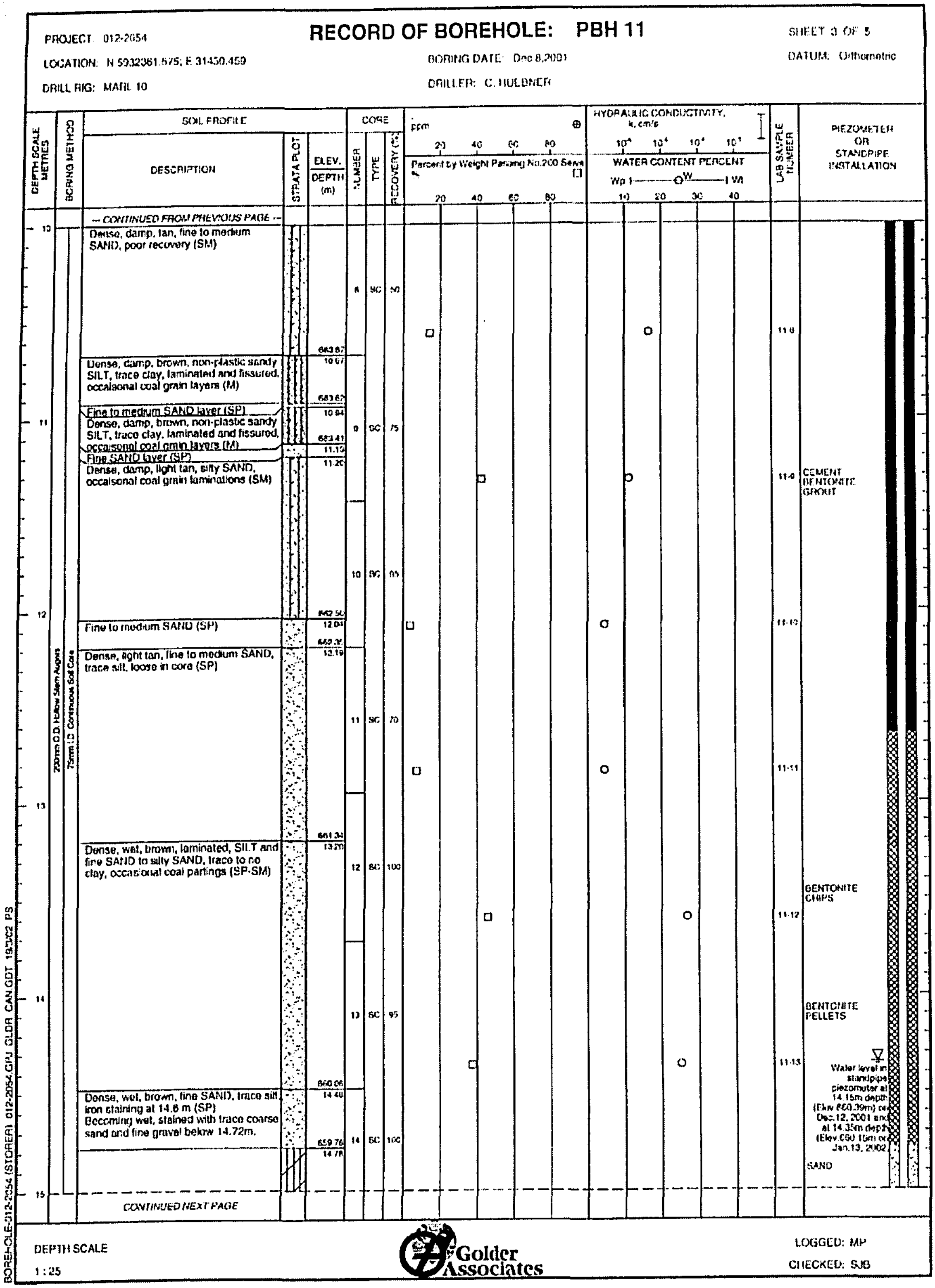

Figure C-2 A selection of the borehole log for Borehole PBH-11 focusing on the Outwash Sands and Silts (Golders).

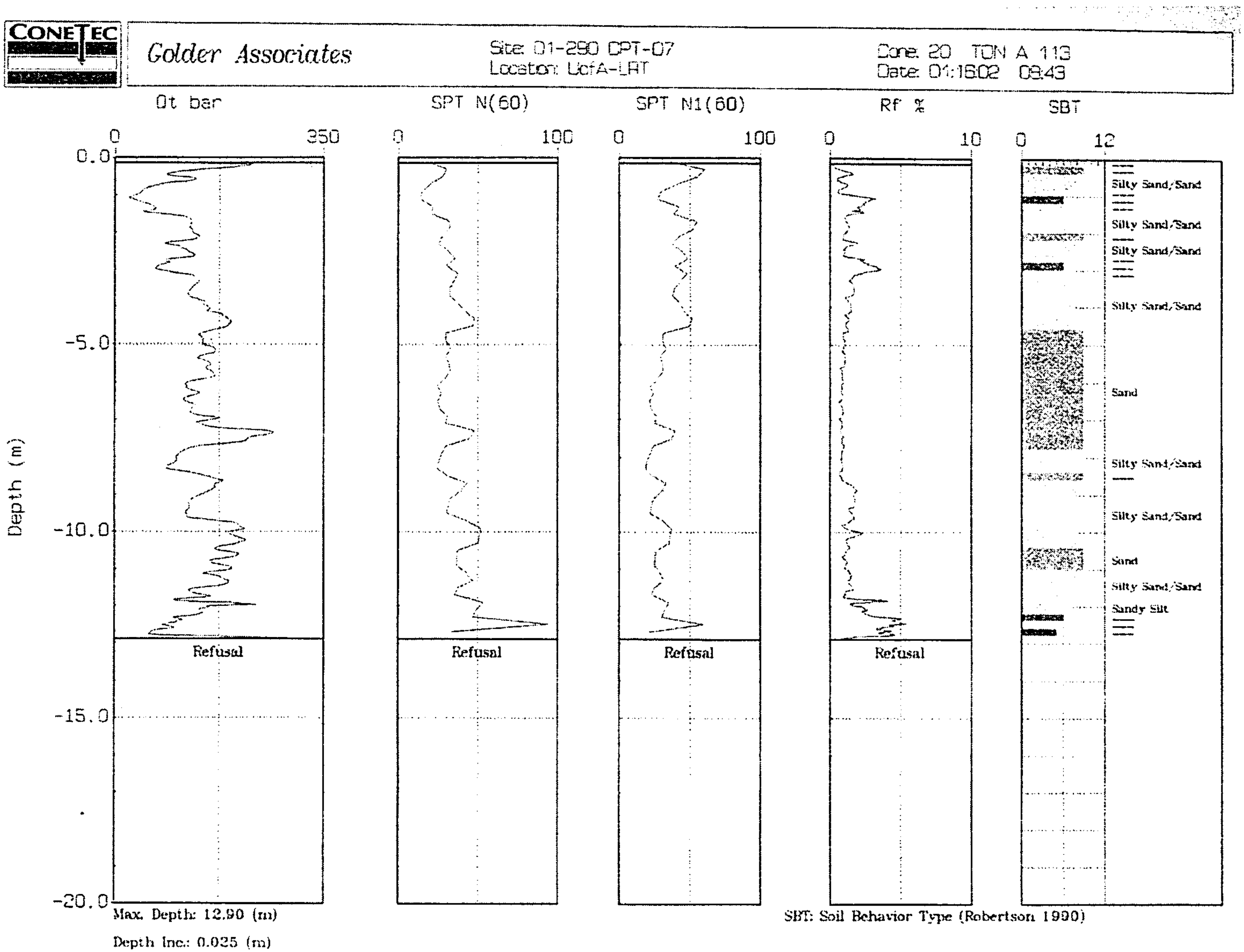

Figure C-3 Results obtained from CPT-07 (Golders).

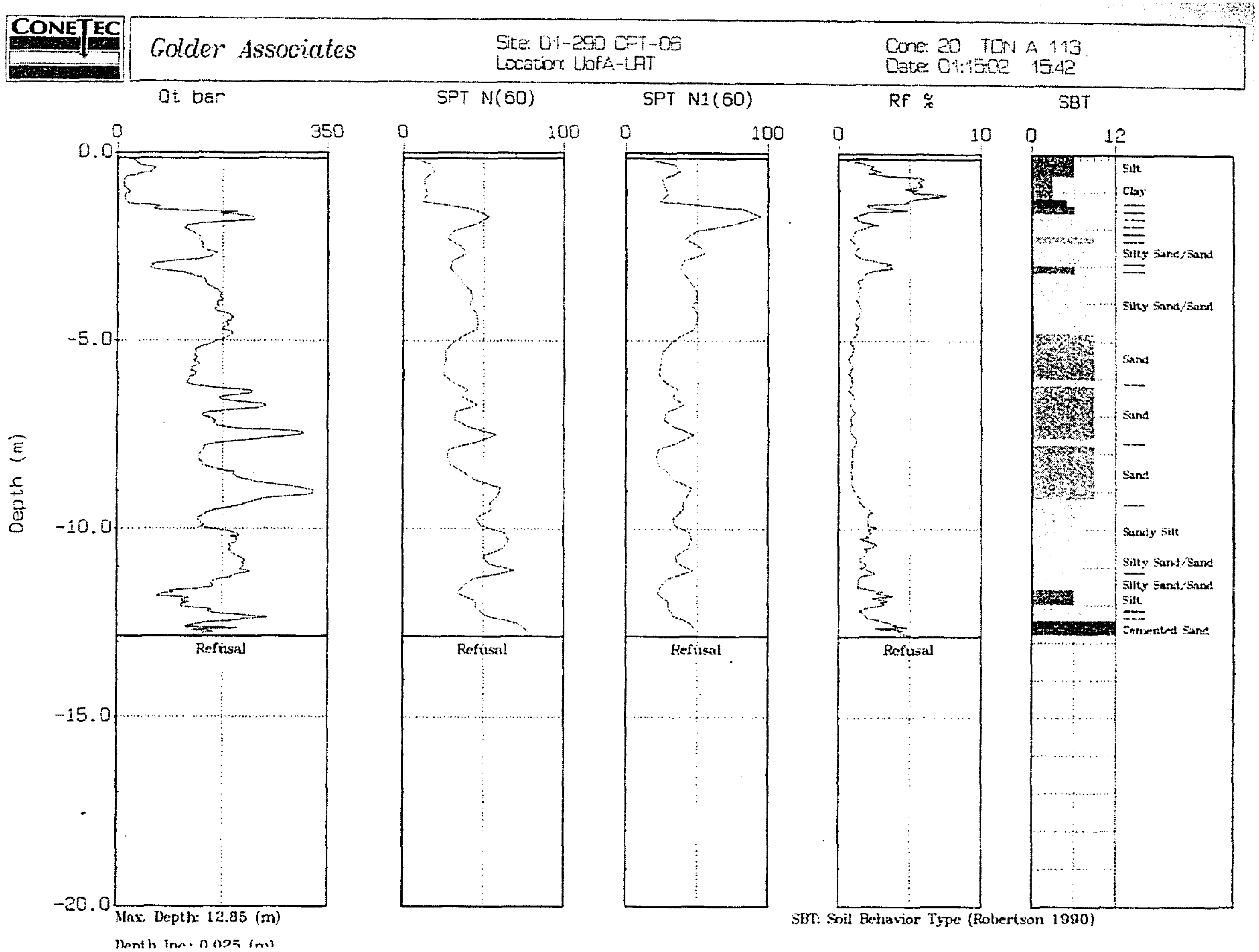

Figure C-4 Results obtained from CPT-06 (Golders).

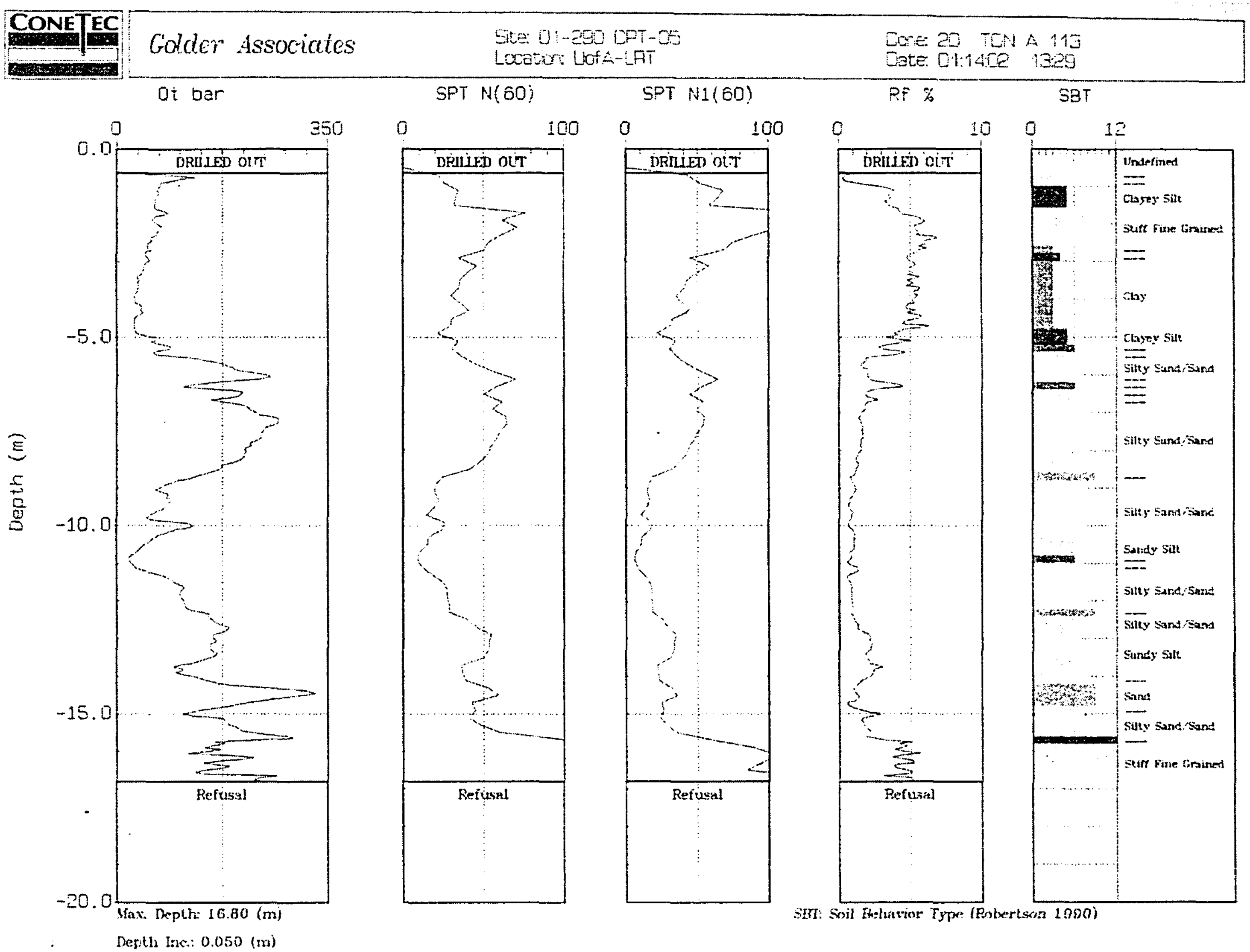

Figure C-5 Results obtained from CPT-05 (Golders).

Appendix D – Glacial Till

This Appendix provides a series of Figures of logs from two boreholes in the region after the Education Car Park where the TBM face was predominantly Glacial Till. These Figures are provided as examples of the soil classification and stratigraphic variation present in the Glacial Till. The logs were recorded by Golders.

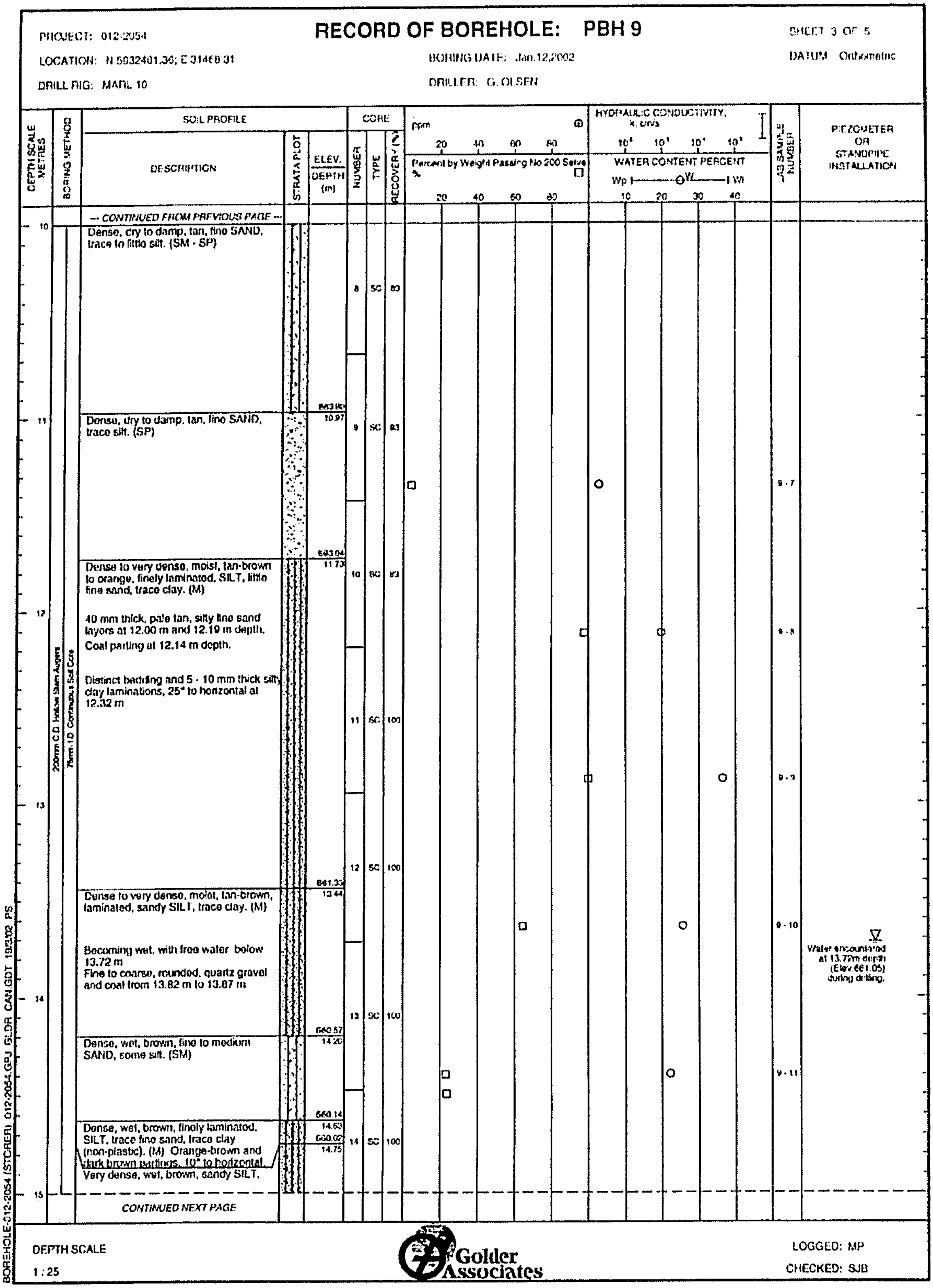

Figure D-1 A selection of the borehole log for Borehole PBH-9 focusing on the Glacial Till (Golders).

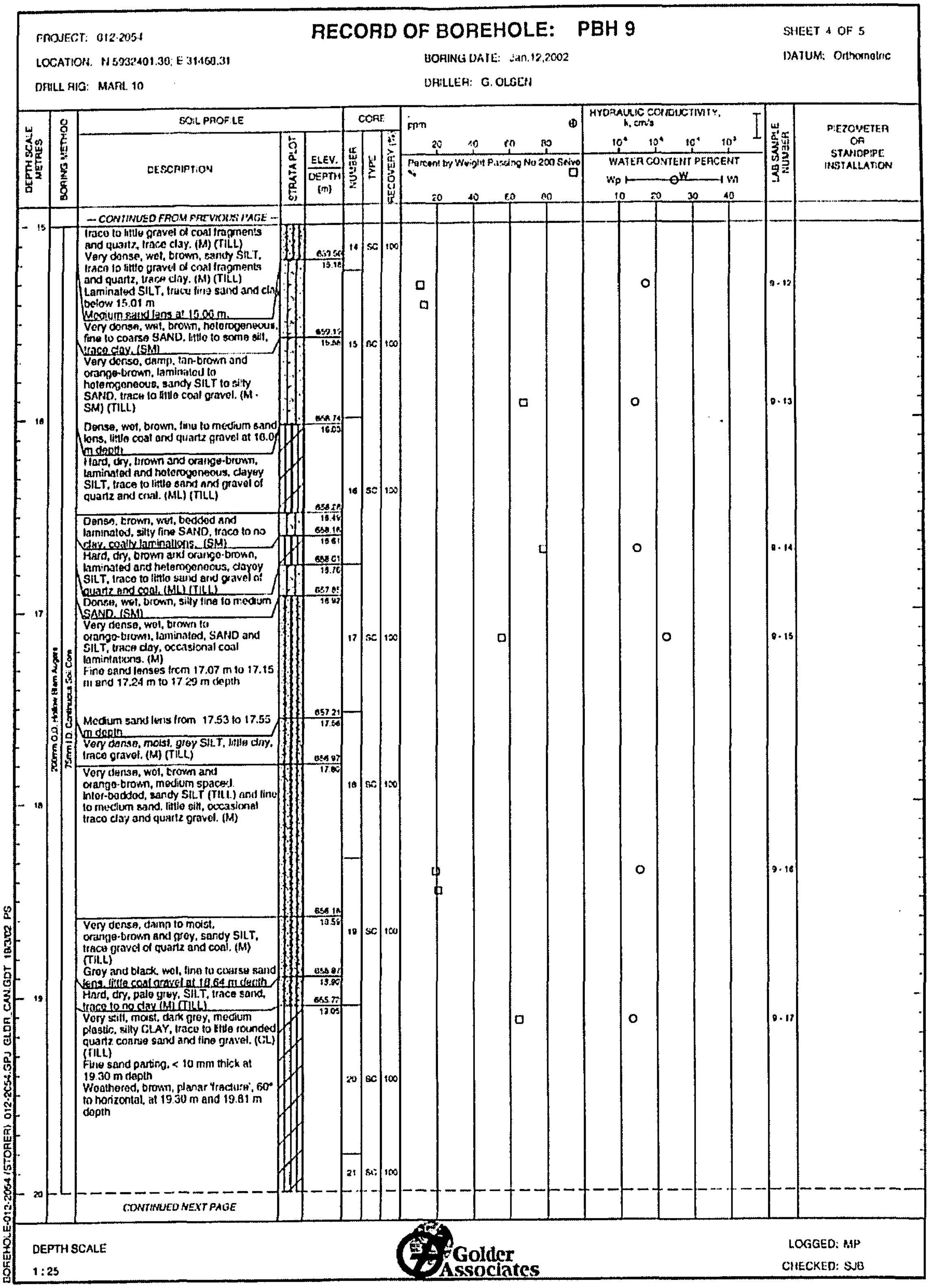

Figure D-2 A selection of the borehole log for Borehole PBH-9 focusing on the Glacial Till (Golders).

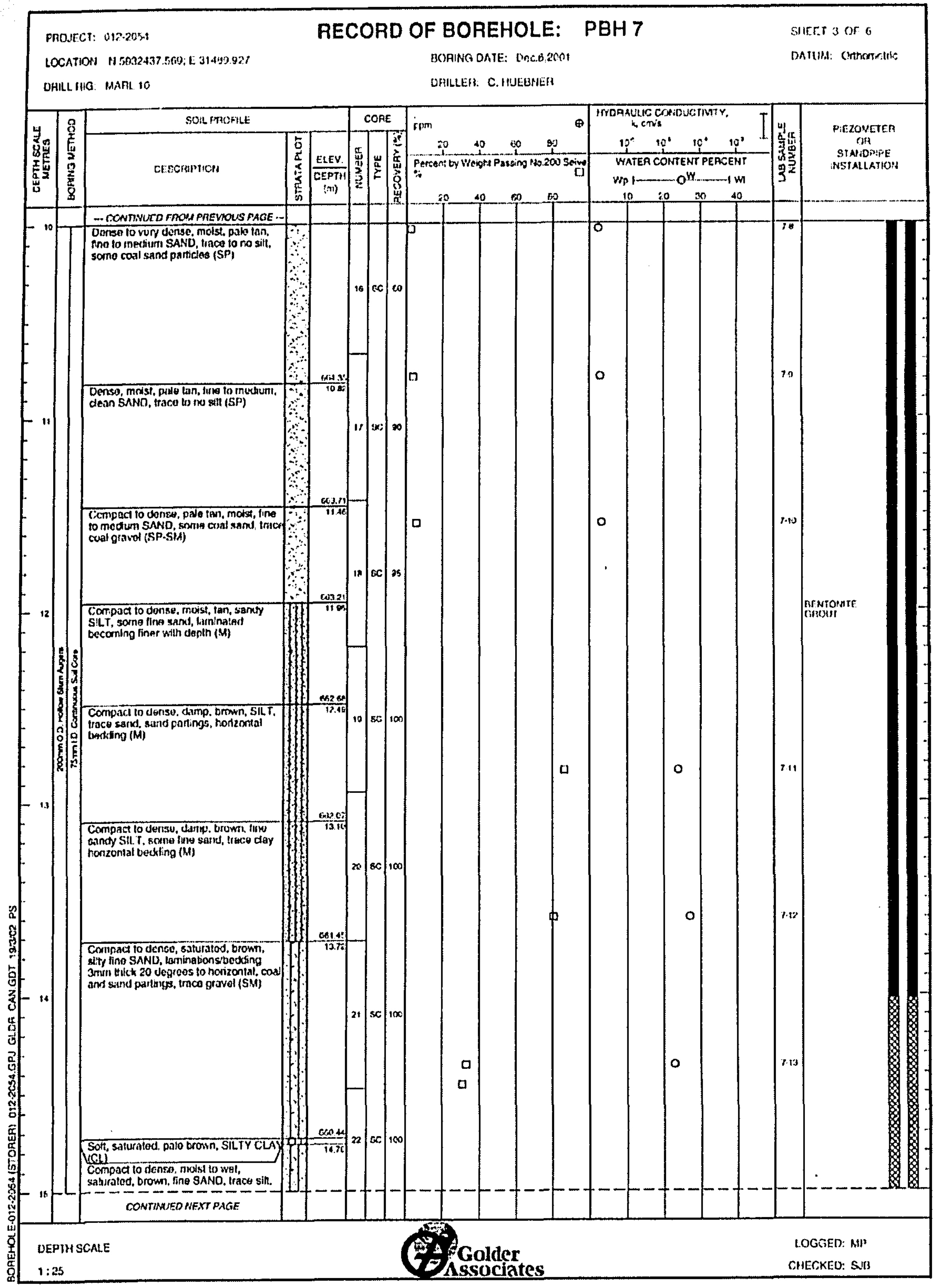

Figure D-3 A selection of the borehole log for Borehole PBH-7 focusing on the Glacial Till (Golders).

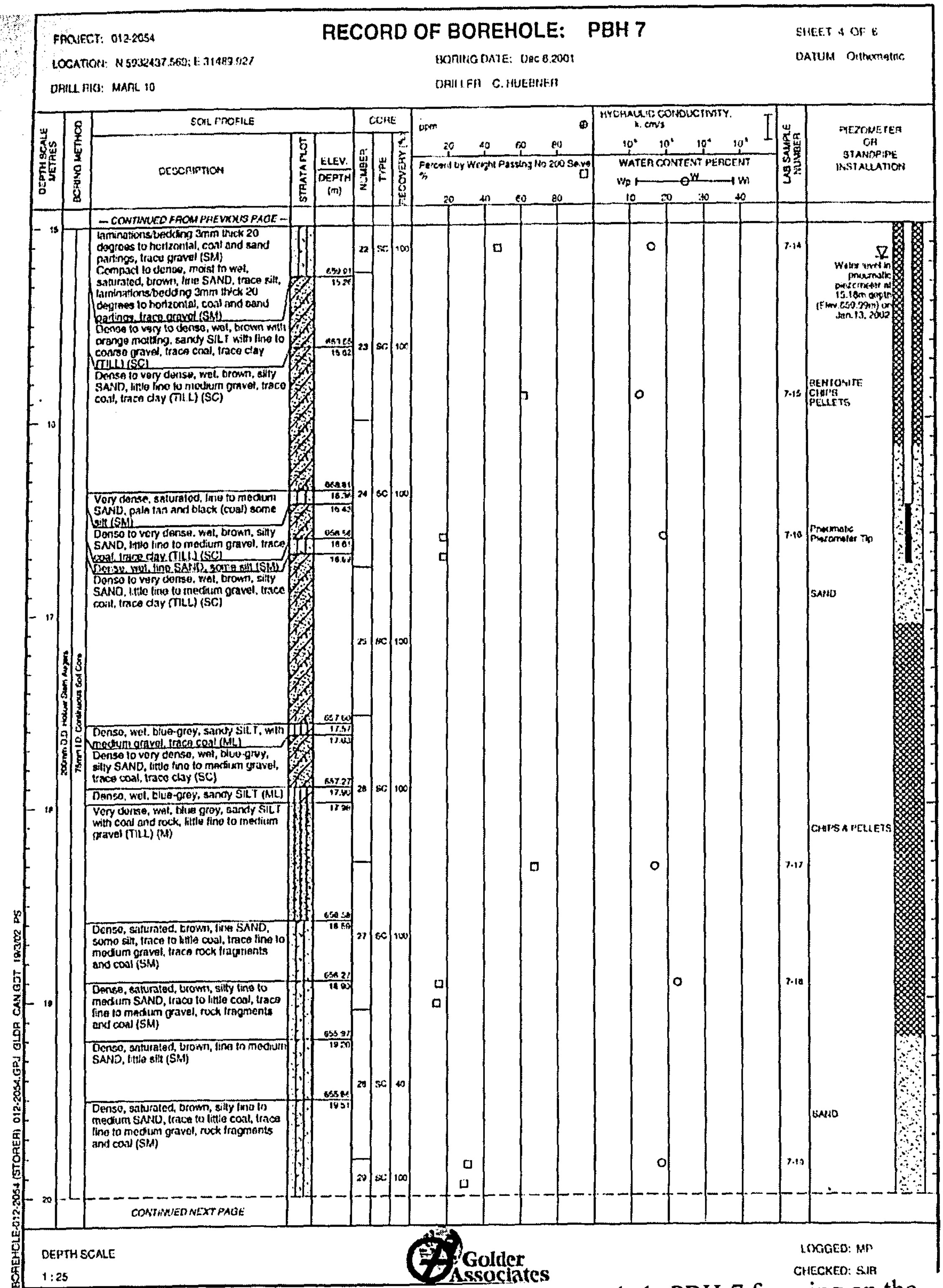

Figure D-4 A selection of the borehole log for Borehole PBH-7 focusing on the Glacial Till (Golders).

Appendix E – Bedrock

This Appendix provides a series of Figures of logs from three boreholes in the region approaching the removal shaft where operation took place in mixed face conditions of Glacial Till and Bedrock. These Figures are provided as examples of the soil classifications and stratigraphic variation present in the Bedrock. The logs were recorded by AMEC and Golders.

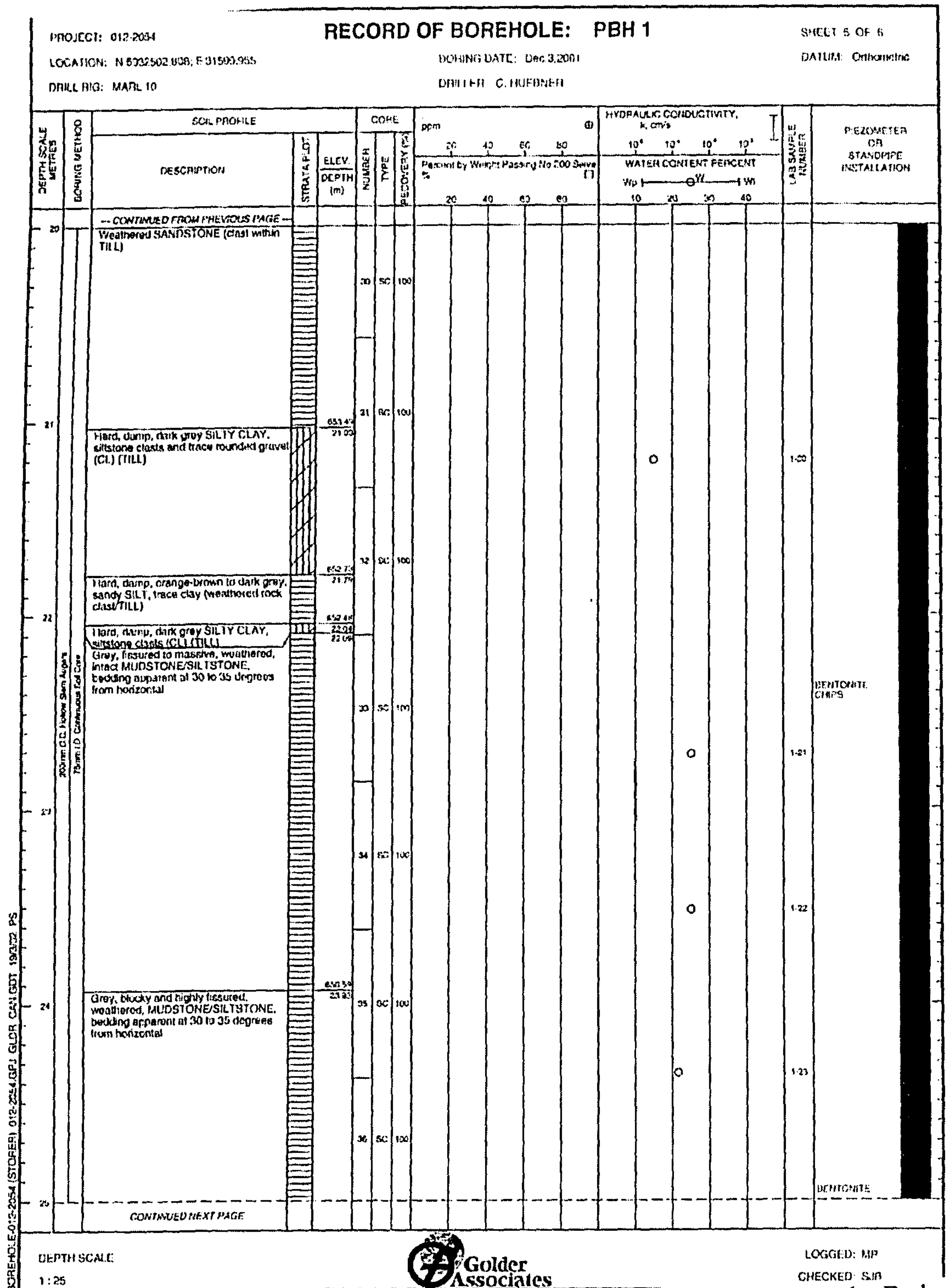

Figure E-1 A selection of the borehole log for Borehole PBH-1 focusing on the Bedrock (Golders).

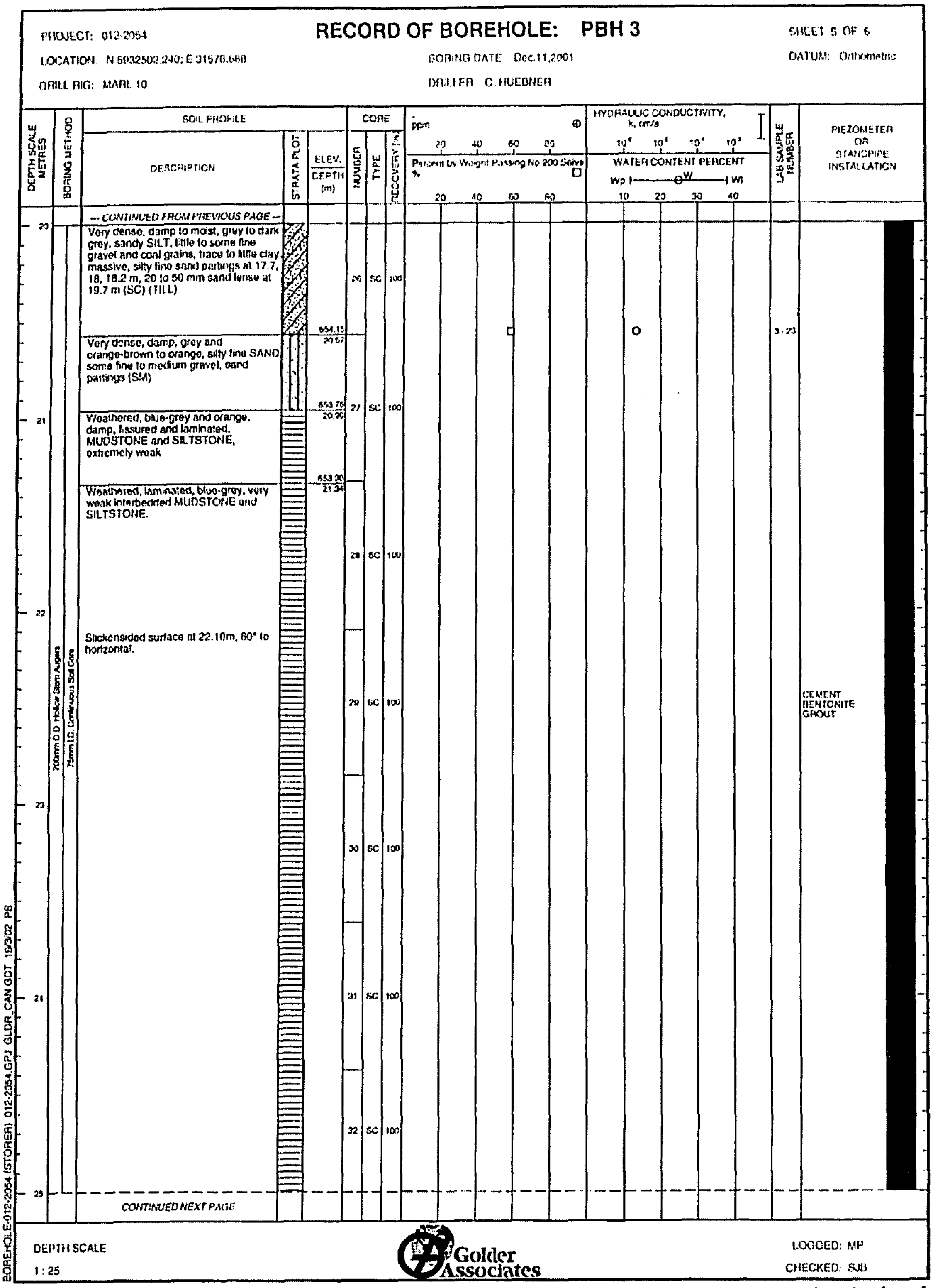

Figure E-2 A selection of the borehole log for Borehole PBH-3 focusing on the Bedrock (Golders).

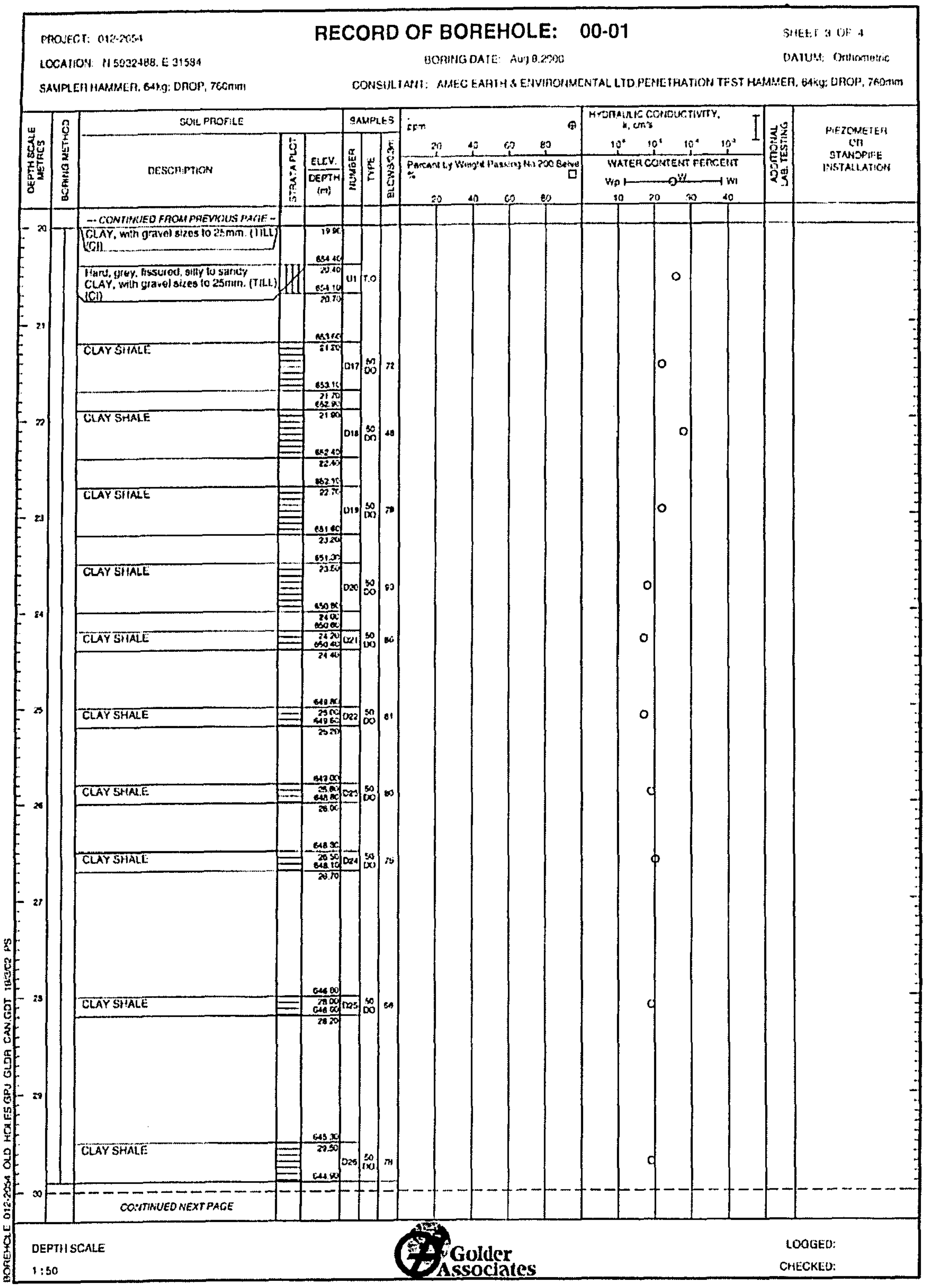

Figure E-3 A selection of the borehole log for Borehole 00-01 focusing on the Bedrock (AMEC).

Appendix F – Grouting Procedures for Edmonton SLRT Construction

This Appendix contains "Method Statement MS-006, Revision A", the revised grouting procedures implemented for the construction of the Edmonton SLRT. This revised grouting procedure was implemented before construction passed under the Education Car Park during the construction of the Southbound Tunnel and remained in place for the rest of construction.

EDMONTON SOUTH LRT

Method Statement MS-006 Rev A

(Specification Reference 02351 1.5.2.9 & 02353 1.4)

TUNNEL GROUTING PROCEDURE

(Revisions in bold italics)

SCOPE

The TBM excavates a bore of 6539 mm to 6564 mm depending upon the wear of the cutting tools, the rings which are constructed within this void are 6300 mm. Consequently an annulus of up to 132 mm exists between the back of the rings and the ground. To prevent any ground settlement and to maintain the shape of the ring, it is necessary to fill this void as soon as practical after the ring has been constructed. This backfill is done by grouting as the TBM advances and can be carried out either through the tailskin, utilising purpose designed grout tubes, or through the segments of the rings themselves.
This method covers both techniques of grouting, including materials, plant, equipment and the quality control check to assure complete grouting.

DEFINITIONS

Grout Mix
Cementitious grout containing cement, sand, fly ash and bentonite. Other additives such as accelerators, anti-wash out etc will be used as required.

Preferred mixes
Design mixes will be developed as works progress. For launching the machine an initial mix has been developed.

Mix designs are given in terms of proportions in kilograms to produce one cubic metre of grout and in pounds to produce one cubic yard.

Design mixes will be varied from time to time. The initial mix is included as an appendix to this method.
It is not intended to amend this method should the mix change, only the design mix sheet. It is therefore imperative that the revision status of the design mix is checked prior to use. Changes to the design mix will be forwarded to the Engineer.

Grout Gel Time, Buoyancy
The grout placed behind a completed liner ring must achieve sufficient cohesion (gel strength), within 45 minutes of grout placement (or, prior to full extrusion of the ring from the TBM tail can), to counteract the buoyant force acting on that ring. Grout cohesion will be assessed using a shear vane. Correlation will be made between the development of cohesion in the gelling grout and the time to mobilization of full resistance to buoyancy (as assessed using a free-floating, low-weight cylinder immersed in grout).

Anti wash out
The inclusion of the water reducers and bentonite provides the grout with anti washout properties. This will mitigate the dilution, and consequently the effectiveness of the grout, by ground water. Additional anti washout agents will be sourced, such that the mix can be modified of required.

Grout Set Time.
There is no specific requirement for early stiffness, apart from ensuring the rings remain in tolerance. To demonstrate sufficient stiffness gel testing will be carried out using a shear vane. Gel testing will be carried out for one ring each shift at 45 minutes, 1 hour, and 4 hours. A test will be done at 6.5 hours if required. The set time can be greatly enhanced using accelerators added to the grout at the pint of injection. The most effective of these being Sodium Silicate.

Strength Tests.
One-day strength of 1.5 MPa and 28-day strengths of 10 MPa are required to meet the contract specification. Samples will be taken from the grout; 100mm test cubes will be cast and crushed to confirm strengths.

Tailskin Grouting.
Injection of grout will be through 30 mm by 60 mm oval section tubes installed into the TBM tailskin. ***Four*** pairs of tubes are provided around the circumference of the tailskin, however the grouting equipment allows only four ports to be used at the same time.

Grout injected from the tubes emerges at the rear of the tailskin; consequently grout will always be present at the rear of the tail seal brushes. ***To prevent blockage of tail tubes and contamination of the wire brushes, which may occur during grouting, it is proposed to use an non cementicious material (e.g. Bentonite) for the last 300 mm advance of the last ring of each shift.***

Segment Grouting.
Should the grout tubes within the tailcan become blocked, it may be necessary to grout through the segments. Injection of grout will be carried out through grout holes through the concrete segmental lining. Again it is proposed to use four ports only at shoulder and knee position on the rings on both sides.
For segment grouting, an accelerated grout will be preferable, as it is not possible to grout the annulus until the grout holes have been pushed clear of the tailskin.

EQUIPMENT

Grout will be batched off site at a ready mix concrete batching plant.

Grout Cars
Muhlhauser Cars with built in 600v 10HP agitators. Each car has a 100mm outlet at each end. The grout car nominal capacity is four cubic metres. The total calculated grout car volume is five cubic metres.

Grout Pumps
2 No Schwing KSP12 pumps each with 2 independent feeds. 600v, 40HP with PLC controller. Operating pressures variable up to 30 Bar.

Grout Injection nozzles.
Fabricated guillotine valves complete with pressure gauges and manual bleed valves. Attaches to grout line by 2" NPT threads into each the tail tubes.

Pressure gauges.
Manually read gauges with electronic sensors linked backed to the TBM PCL.
Pressure measured will be at the pump. An interpolation will be carried out to assess the actual injection pressure at the annular void.

PROCEDURES

1) Grout will be batched off site at a local ready mix batching plan. Grout will be delivered using ready mix trucks.

2) ***Ready mix trucks will arrive on site and discharge directly into the grout cars using a surface mounted hopper and drop pipe. The pipe-work will be lagged in areas where it may be affected by the weather.***

3) On arrival of the grout car within the shaft, the flexible hose will be moved over the grout car. Grout cars are supplied with electrical agitators, which will be activated during filling. The flexible hose will be installed into the filling hatch and the valve at the base of the wet tank will be opened allowing grout to flow through the 150 mm feed pipe. The grout car will be the last car in the train to be serviced, thus minimizing the time grout remains in the skip. The grout car will be washed out if required in the shaft prior to refilling. Should this be carried out, a silt trap will be installed into the shaft sump, which will allow the removal of fines on a daily basis.

4) The required volume of grout for a ring is 3.2 cubic metres. The full volume of the grout car is five cubic meters. This differential equates to a 525 mm dip from the top of grout car, to top of grout in the skip. Note that this is the theoretical maximum grout take. Should tool wear occur, less grout would be required. ***Experiences on the South tunnel indicated a grout take of 4m3 for each ring.***

5) On completion of reloading the skip, the train will be transported through the tunnel to the rear of the TBM. To prevent spillage of the grout through the tunnel, fabricated hatches have been added to the top of the skip.

6) At the TBM, the segments will be unloaded before pulling the train back to locate the grout car at the pumps. The grout car will then be uncoupled from the last muck car allowing the train to pull under the discharge of the belt. The grout cart will be situated between the Schwing pumps allowing a separate pump on each of the two skip outlets.

7) The grout car will be coupled up to the pumps, and pumps will be coupled to their respective injection ports before giving the all clear to the TBM operator. Grouting will then commence concurrent with the mining of the face.

8) ***Grout injection rate will adjusted manually to match the mining advance rates.*** The pumping capacity of the grout pumps is up to 20 cubic metres per hour. With 3.2 m^3, this gives a variable grouting time down to 9.6 minutes. Mucking rates will be variable, however are expected to average around 25 minutes. ***Grout will be pumped into the annulus to maintain a pressure at the point of injection of 30 psi. Should the pressure fall below 10 psi, The TBM advance will be slowed or stopped to allow the grout pressure to build up. Should the pressure rise above 60 psi, the gout pump stroke rates will be slowed down to allow a reduction of pressure. Pressures will be initially measured at the point of injection with parallel readings at the pump. This will enable the pressure drop within the hose to be assessed and allow pumping to be controlled from the gauges mounted on the Schwing pumps.***

9) For grouting through the tail tubes a 50 mm flexible grout hose will be coupled from the respective pump to the injection tube in the tailskin. A valve will be installed between the hose and the tail tube to allow pressure relief should the tube block. All tubes not in use will be pre-filled with tail seal grease to prevent blockage from back flow.

10) In the event of a blockage during grouting, the respective hose will be uncoupled and will be joined onto the adjacent tube. The blocked tube will be fully flushed out. **Once clear the tube will be purged with bentonite and the entry valve will be sealed off.**

11) During normal mining grout will remain within the hoses and tubes between cycles, however, at times of slow production, such as during the launch, the grout hoses will be flushed out after each ring. ***To achieve this a short mix of grout will be batched (approximately 3m3). This will be pumped as normal and should be fully injected by the time the TBM has advanced 900 mm. At this point 1m3 of bentonite will be added to the grout car form a holding tank on the TBM. This will be pumped into the annulus during the final 300 mm advance of the TBM.***

12) ***At the end of each shift or at any time where extended stoppage is expected the above procedure will be carried out and all hoses and tubes will be fully flushed, and the pump will be cleaned out.***

13) For segmental grouting it will be necessary to install a valve between the injection nozzle and the grout hole. The grout holes do not have non-return valves and consequently a valve is required to enable hose disconnection. A series of 12 valves will be used in rotation allowing 4 in use for the current ring, 4 in use for the previous ring and 4 being cleaned out.

Each segment will incorporate plastic caps to seal grout and lifting holes.

14) On completion of grouting each ring, as well as clearing hoses etc. any spillage will be cleaned from the rings, all grout plugs will be checked and a record will be made as to volumes and pressures used. Copies of grouting records will be forwarded to the Engineer. ***Information with respect to grout volumes, pressures etc will also be recorded by the TBM Data logger, which will also display real time data to surface computer terminals within Aecon's and the Engineers offices.***

15) Secondary grouting is not envisaged, with a fixed volume of grout injected into a known annulus, voids are considered highly unlikely. The specification does however require proof holes to be drilled 20m back from the tail seals in every other ring for the first 20 rings or until such time as confidence is established. After this drilling is to be done every 10 rings, or as otherwise directed.

16) Proof holes will be drilled from the gantry adjacent to the ventilation system. Using an 18 mm diameter drill bit attached to an impact drill, a 500 mm deep hole will be drilled through the grout hole and into the grout, to investigate the grout. The results will be recorded and forwarded to the Engineer.

17) For stoppages, it is necessary to ensure the annulus around the TBM is fully filled, whilst keeping the grout from setting around the TBM and the tail brushes. To achieve this, a bentonite mix will be pumped prior to stopping.

REFERENCES

- Contract specification section 2351 and 02353.
- Method Statement MS 001 Normal Tunnel Construction.

Appendix G – Deep Proximity and Time Settlements

This Appendix contains the Figures showing the deep settlement points along the Northbound and Southbound Tunnel. The Figures present the information in terms of proximity to the TBM face and over the course of time.

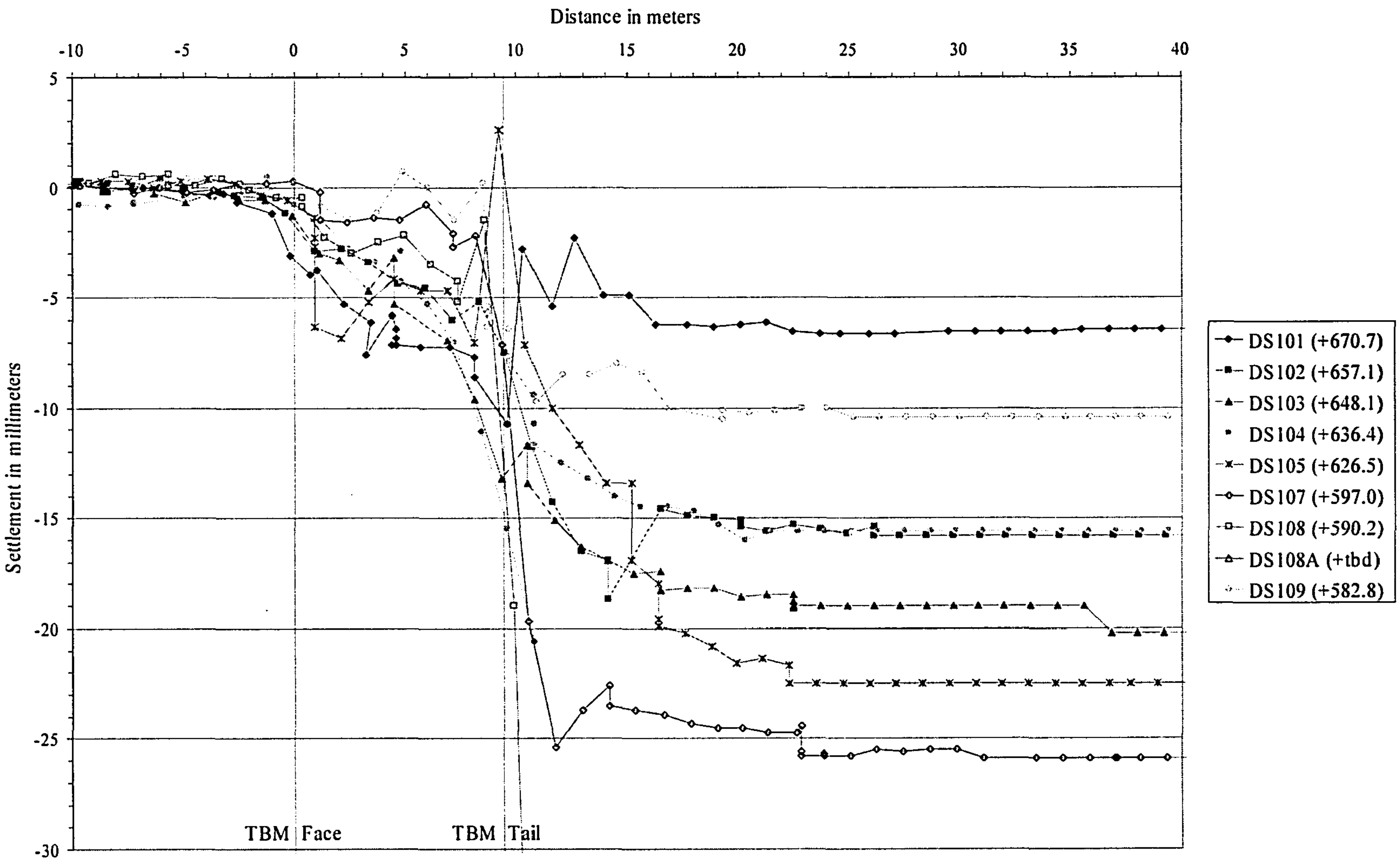

Figure G-1 Observed deep settlements during construction of Southbound Tunnel with proximity to TBM face, from Portal to Education Car Park.

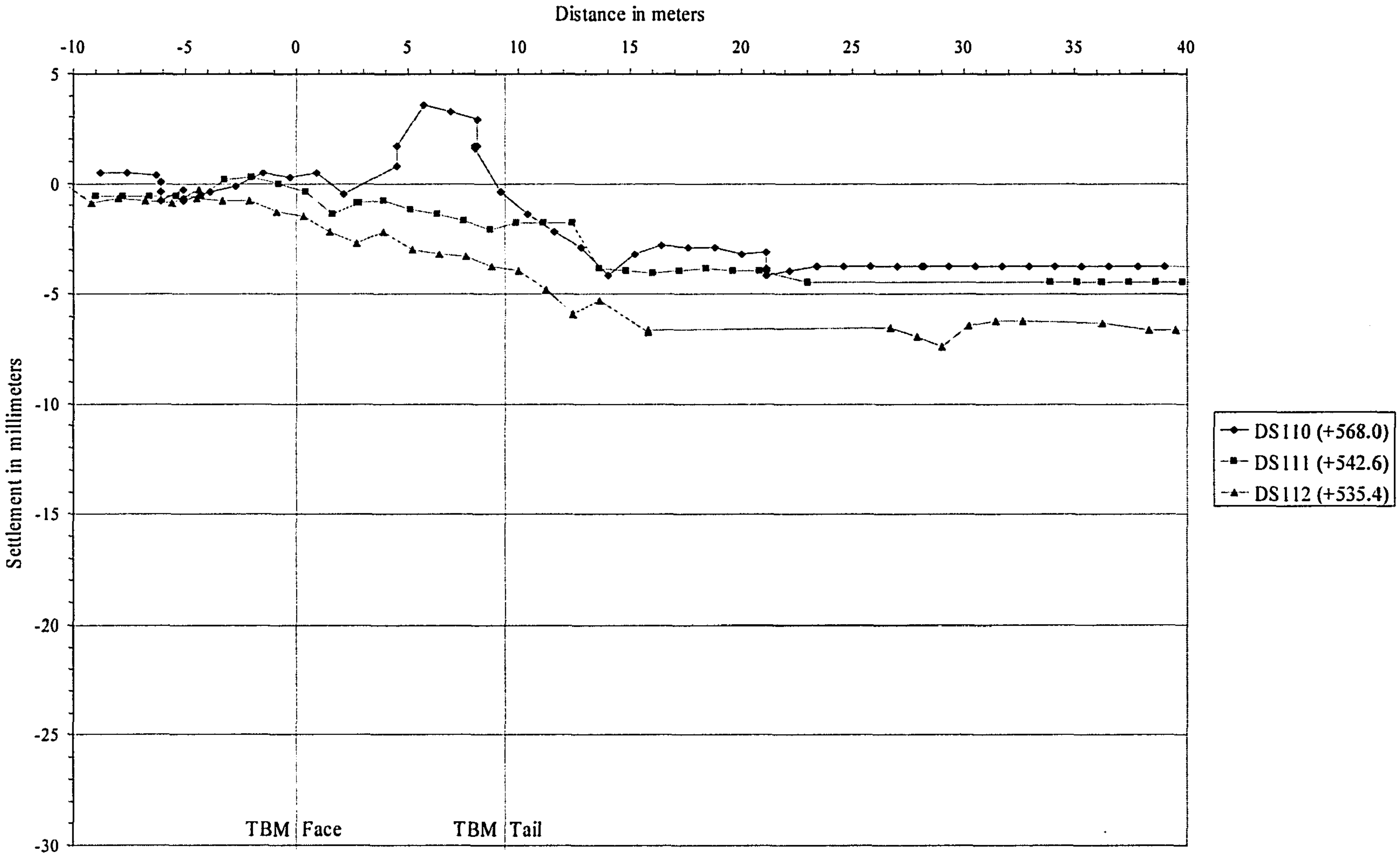

Figure G-2 Observed Deep settlements during construction of Southbound Tunnel with proximity to TBM face, Education Car Park.

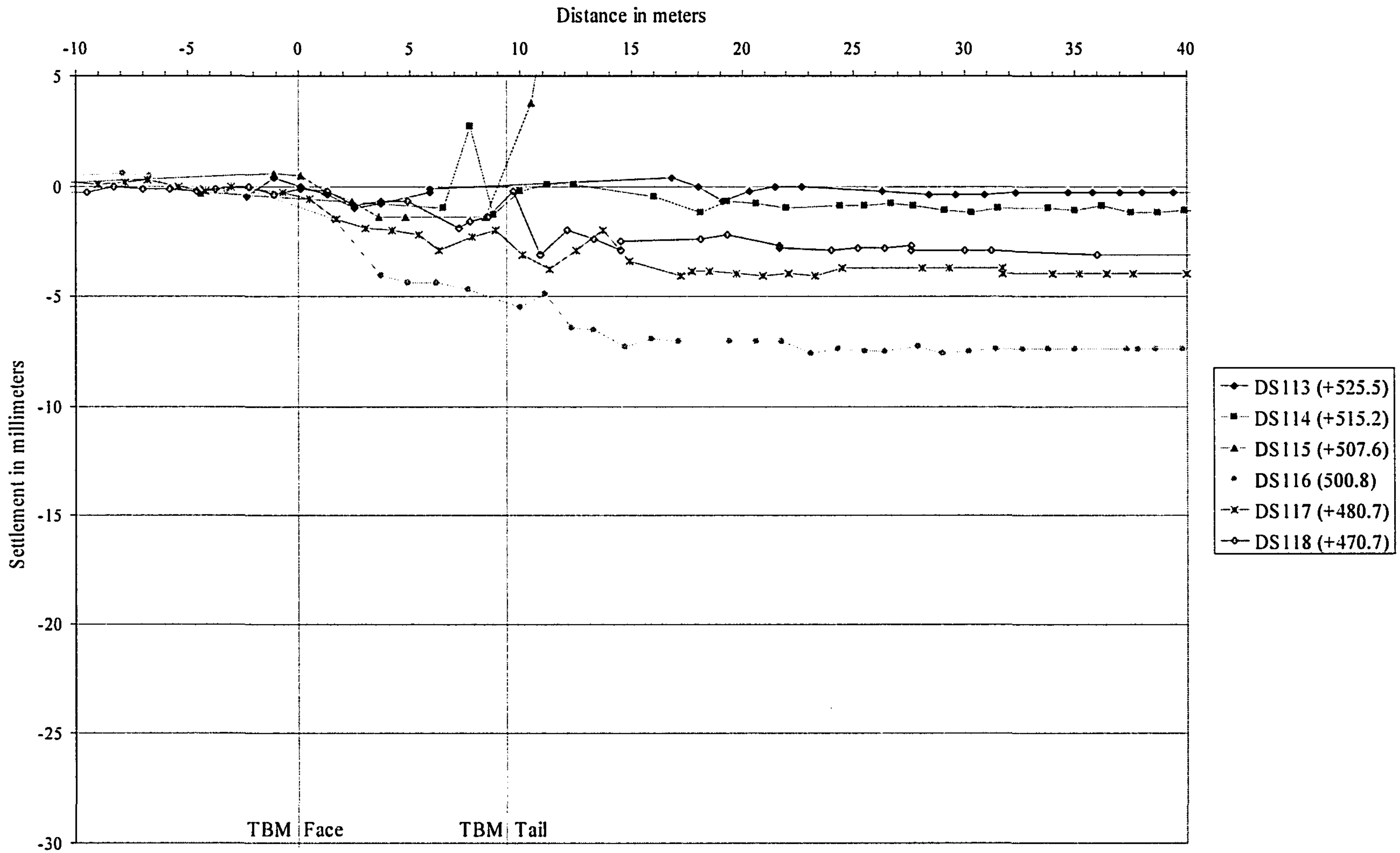

Figure G-3 Observed deep settlements during construction of Southbound Tunnel with proximity to TBM face, Education Car Park to St. Joseph's College.

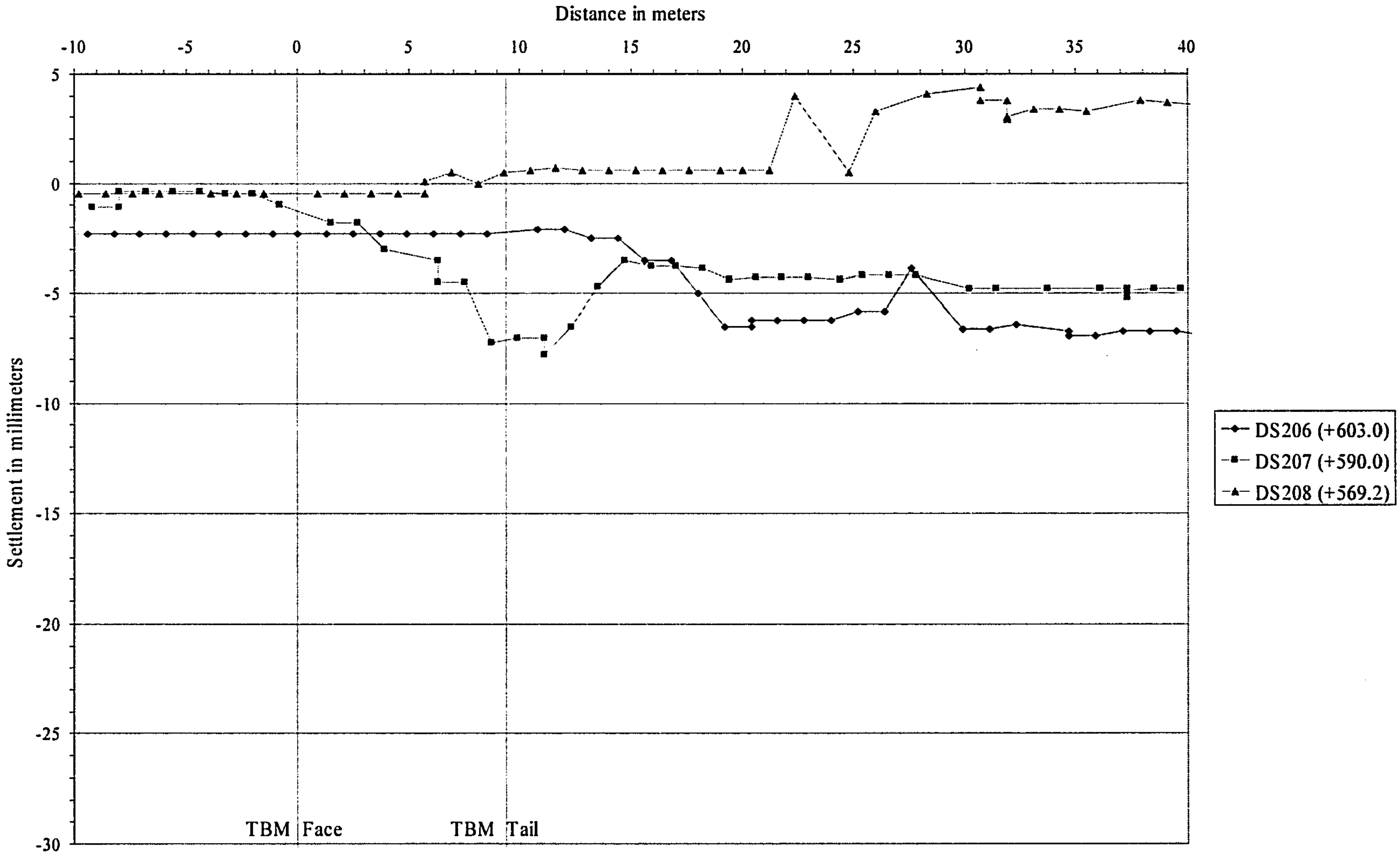

Figure G-4 Observed deep settlements during construction of Northbound Tunnel with proximity to TBM face, from Portal to Education Car Park.

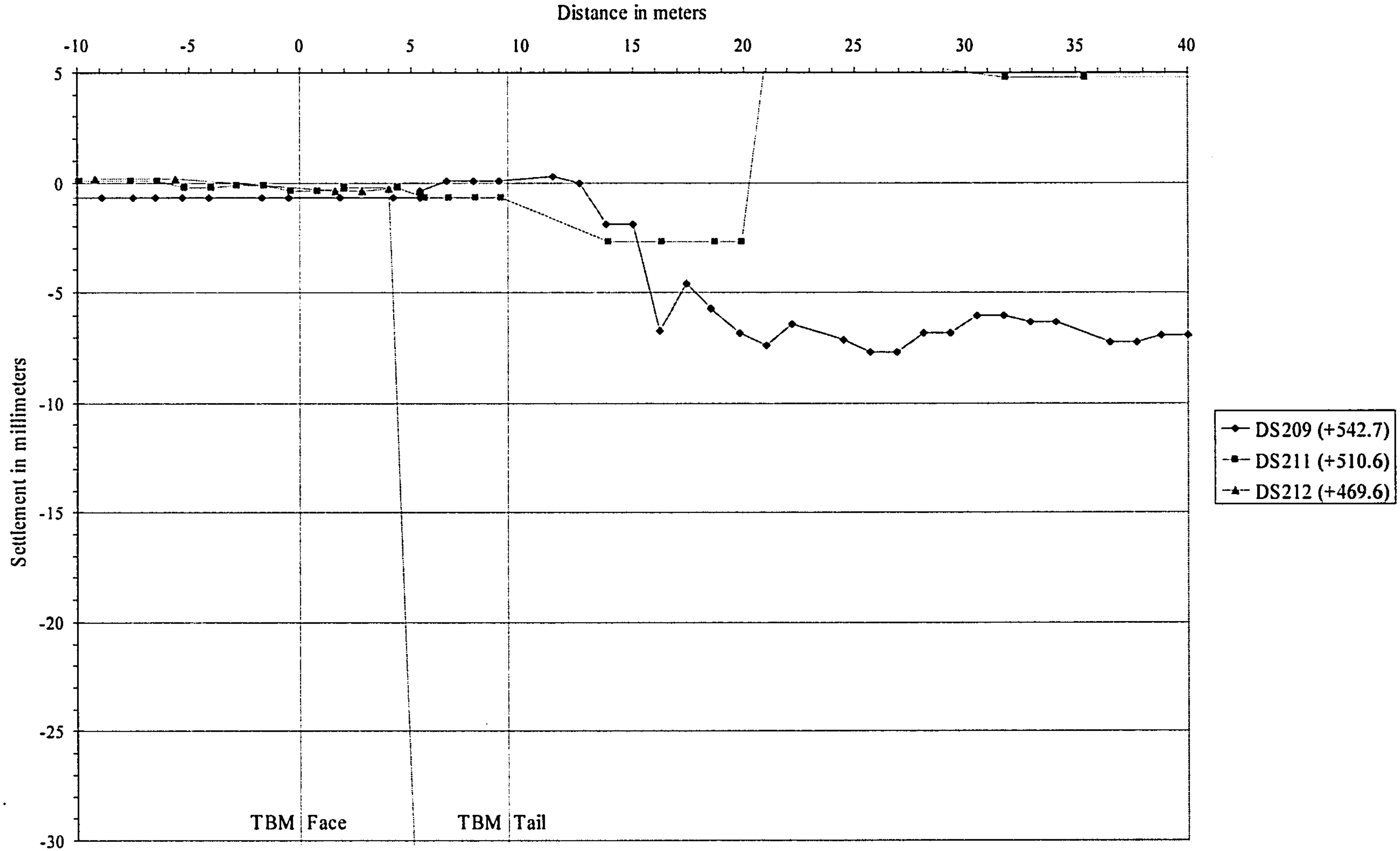

Figure G-5 Observed deep settlements during construction of Northbound Tunnel with proximity to TBM face, from Education Car Park to breakthrough.

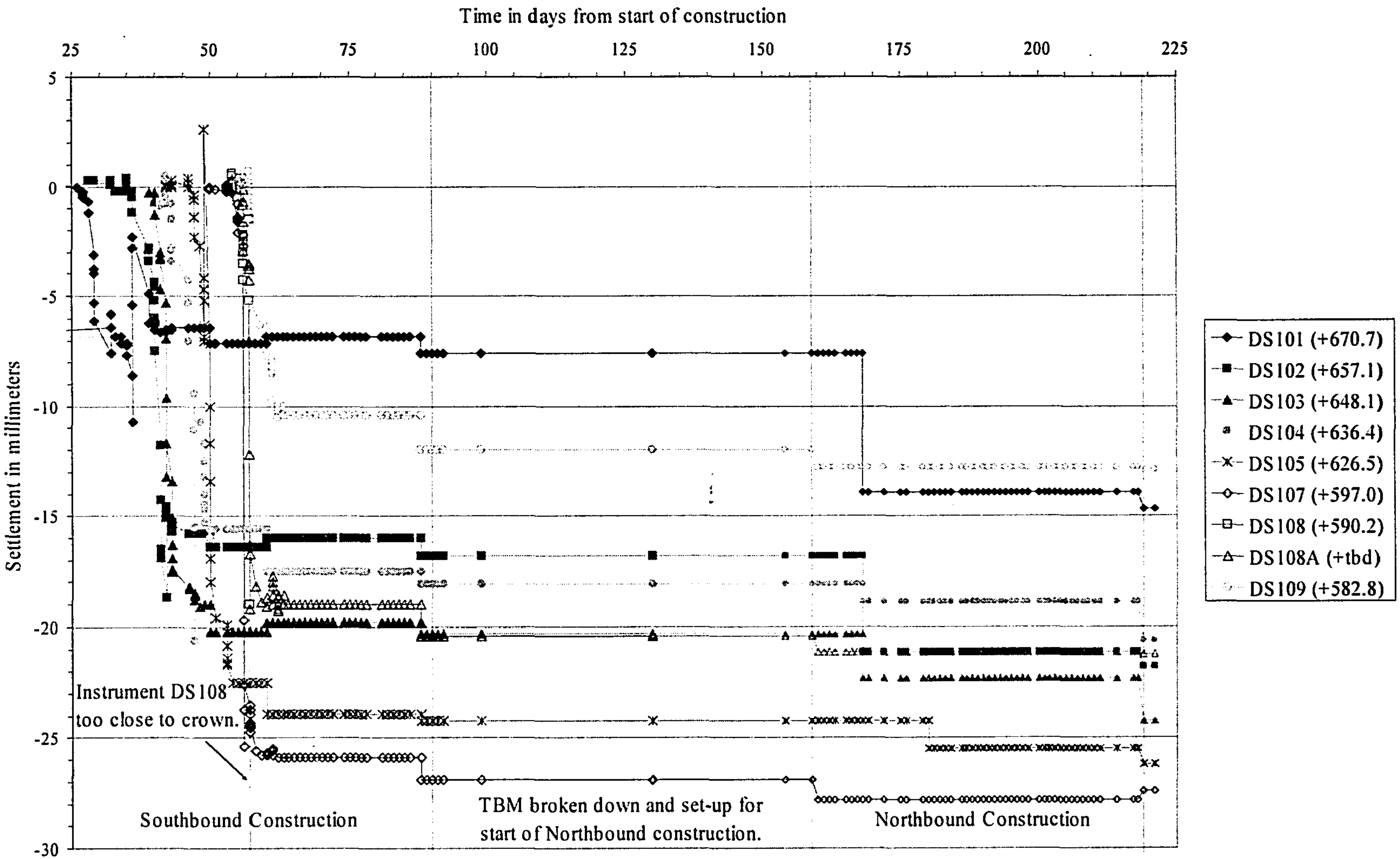

Figure G-6 Observed deep settlements in Southbound alignment instruments over the course of construction with respect to time, from Portal to Education Car Park.

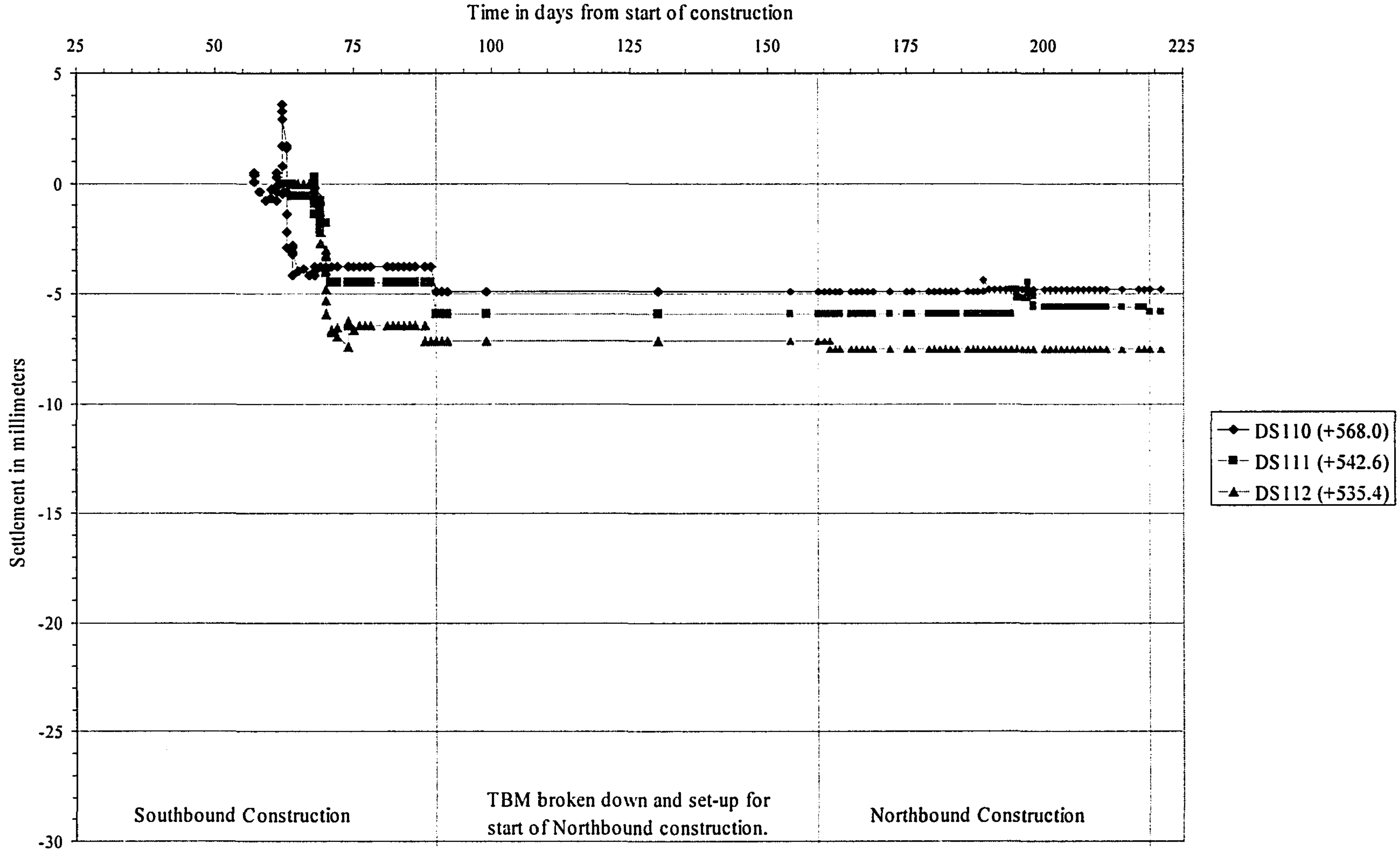

Figure G-7 Observed deep settlements in Southbound alignment instruments over the course of construction with respect to time, Education Car Park.

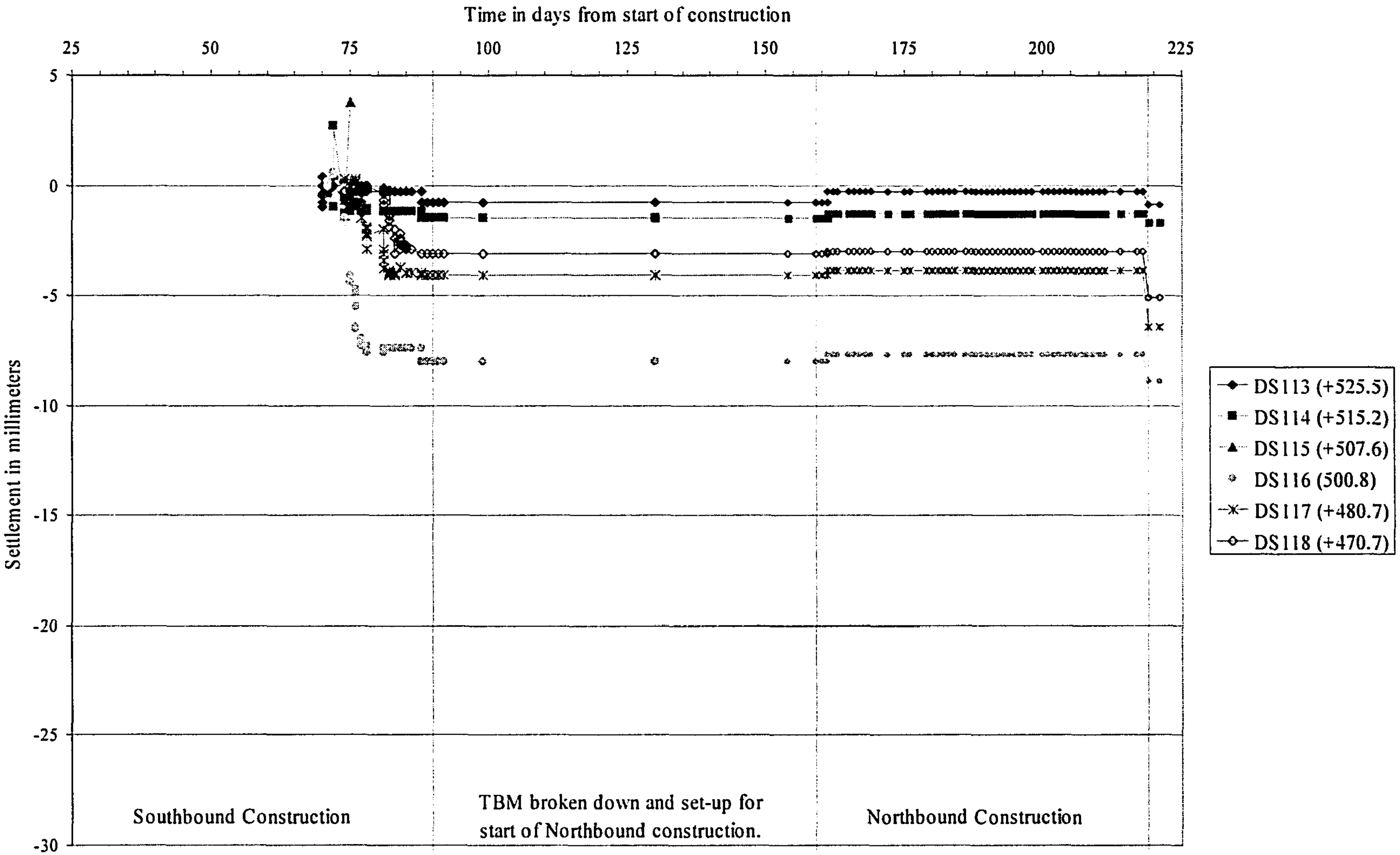

Figure G-8 Observed deep settlements in Southbound alignment instruments over the course of construction with respect to time, from Education Car Park to St. Joseph's College.

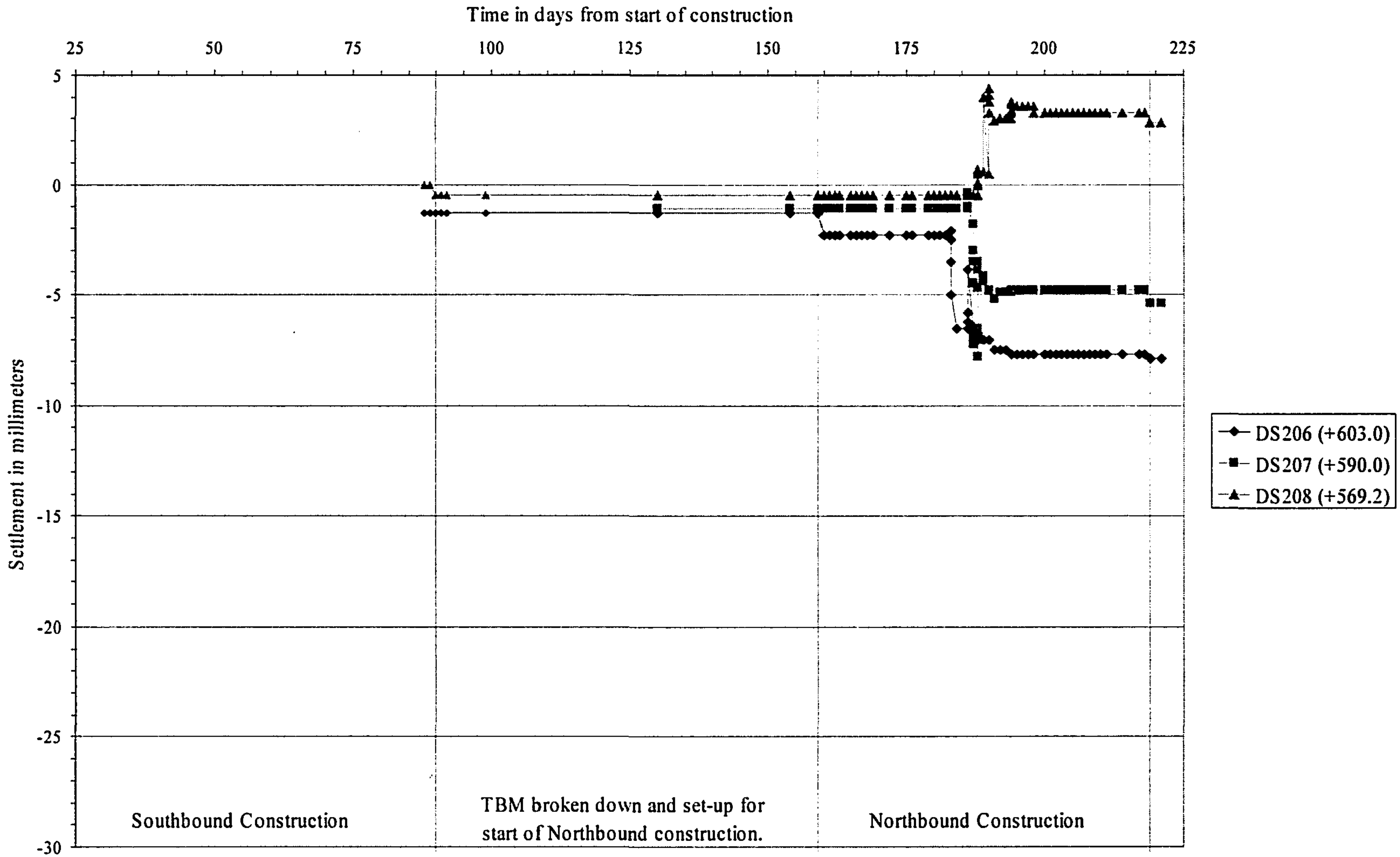

Figure G-9 Observed deep settlements in Northbound alignment instruments over the course of construction with respect to time, from Portal through Education Car Park.

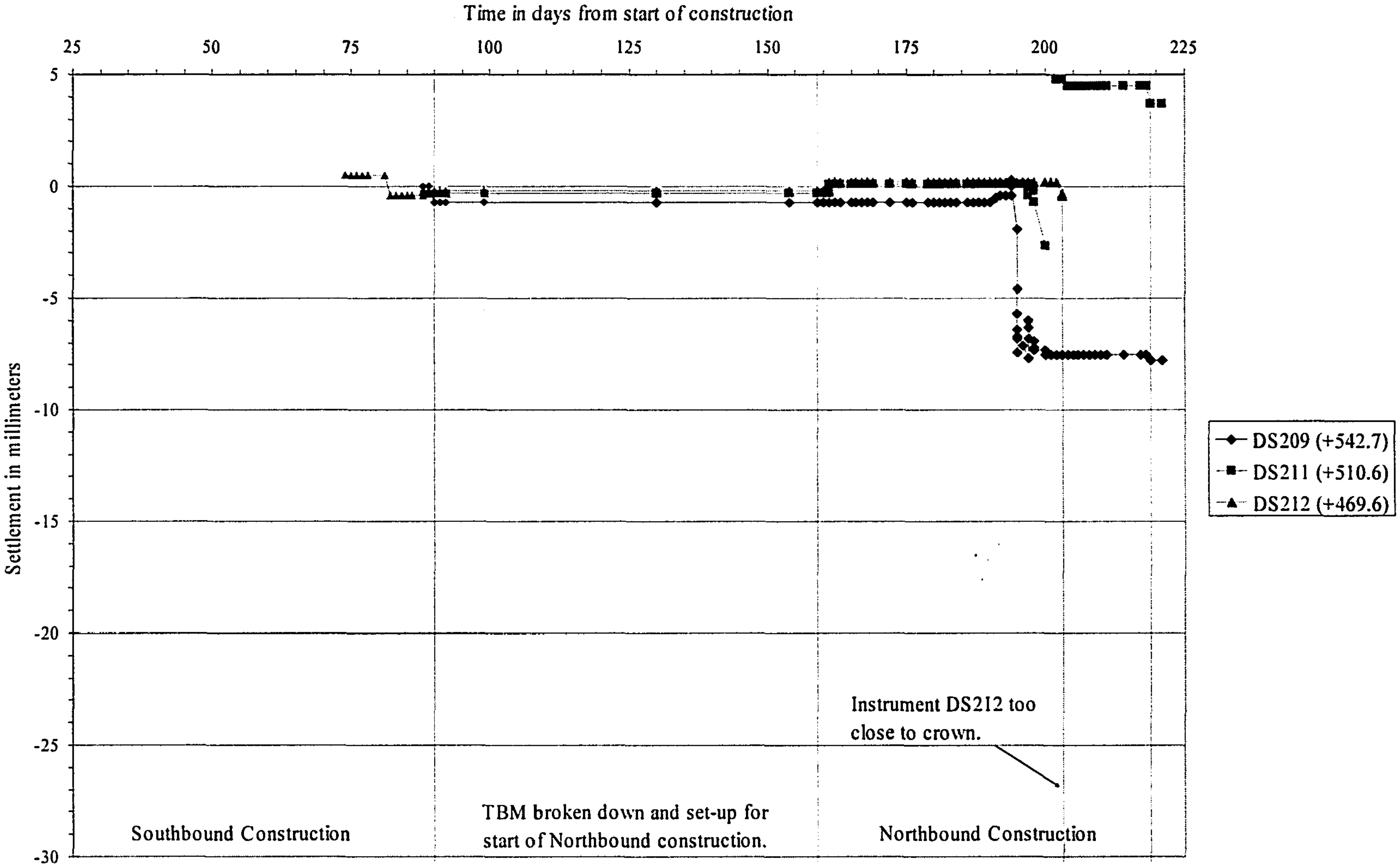

Figure G-10 Observed deep settlements in Northbound alignment instruments over the course of construction with respect to time, from Education Car Park to breakthrough.

Appendix H – Deep Array Settlement

This Appendix contains the Figures for the observed deep array settlements at Array A, B, C, and D for the construction of the Southbound and Northbound Tunnels.

The arrays show the settlement perpendicular to the tunnel alignment. The elevation of the settlement gauges are reported over an interval of 50m. From when the TBM is 10m from reaching the gauge (-10m), to when the TBM has passed 40m beyond the settlement gauge (40m).

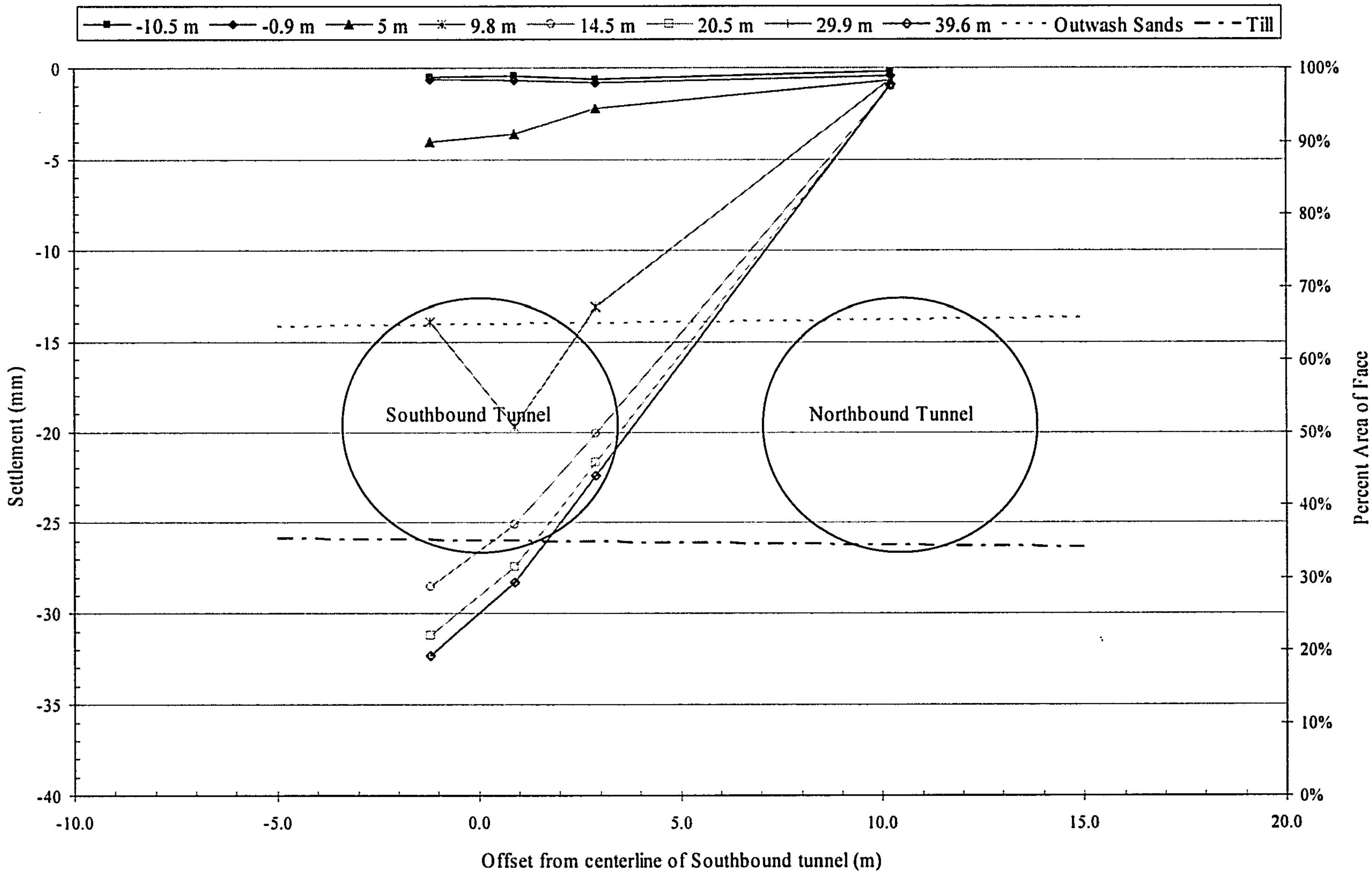

Figure H-1 Observed deep settlement at Array D during construction of Southbound Tunnel.

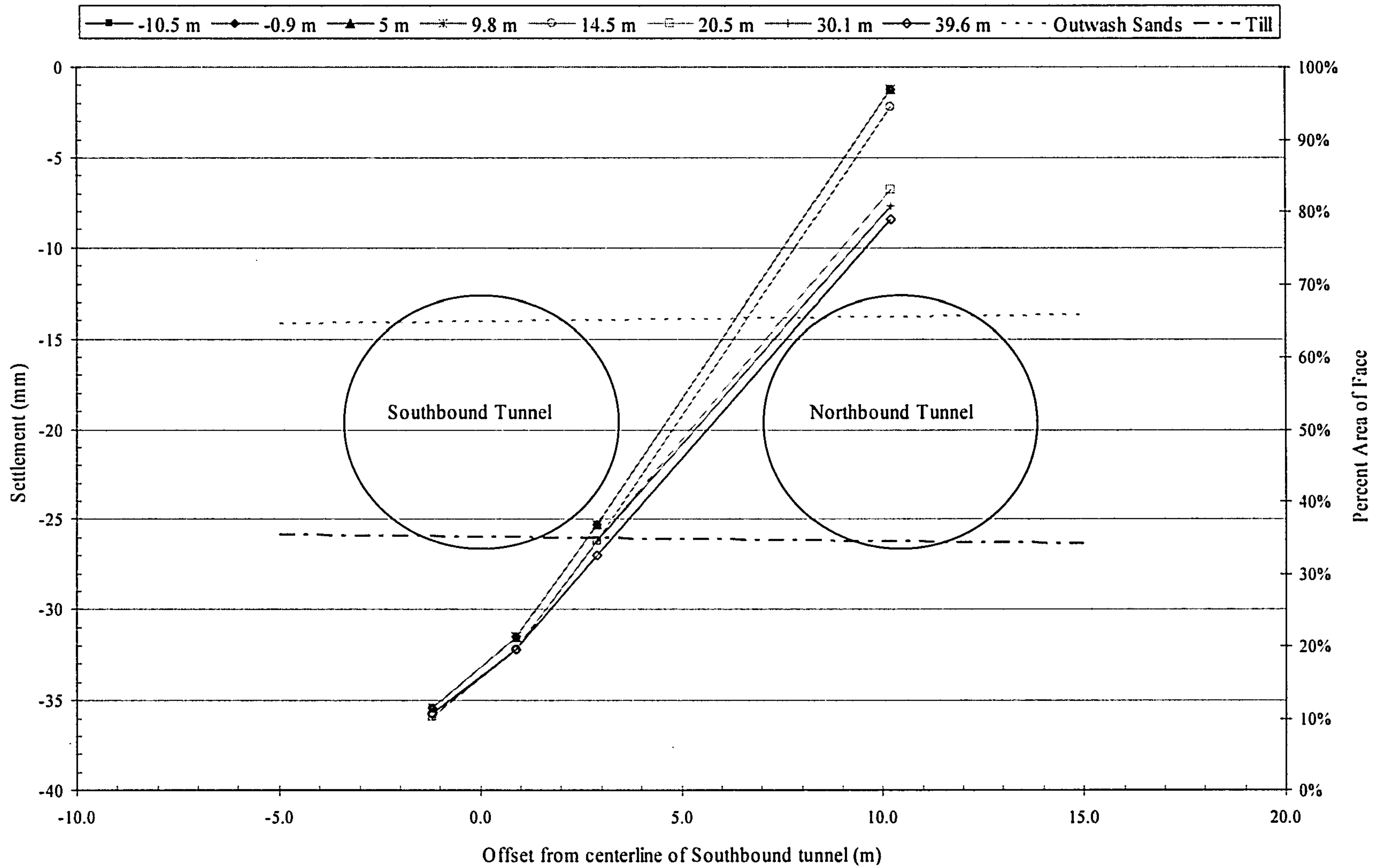

Figure H-2 Observed deep settlement at Array D during construction of Northbound Tunnel.

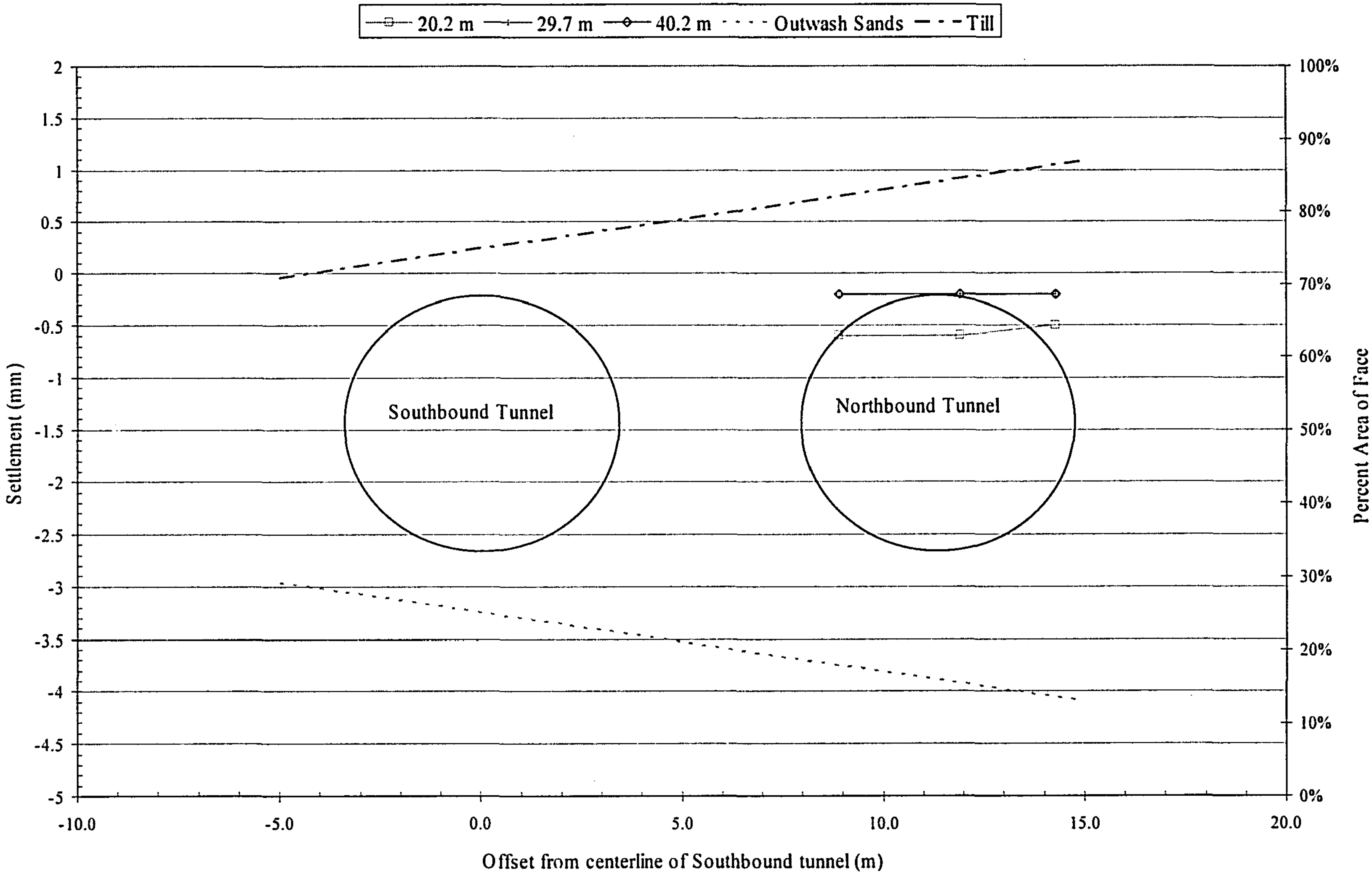

Figure H-3 Observed deep settlement at Array C during construction of Southbound Tunnel.

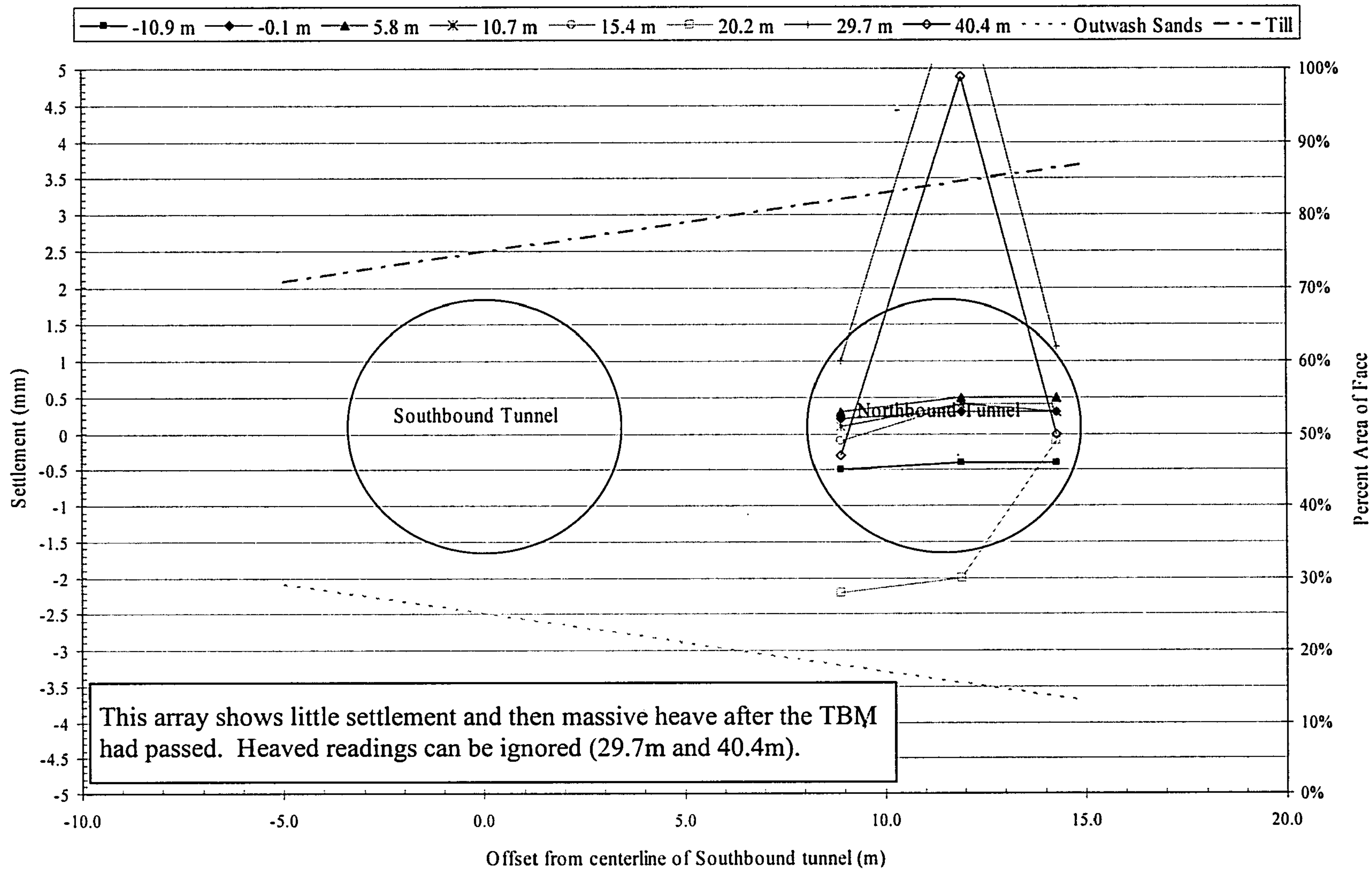

Figure H-4 Observed deep settlement at Array C during construction of Northbound Tunnel.

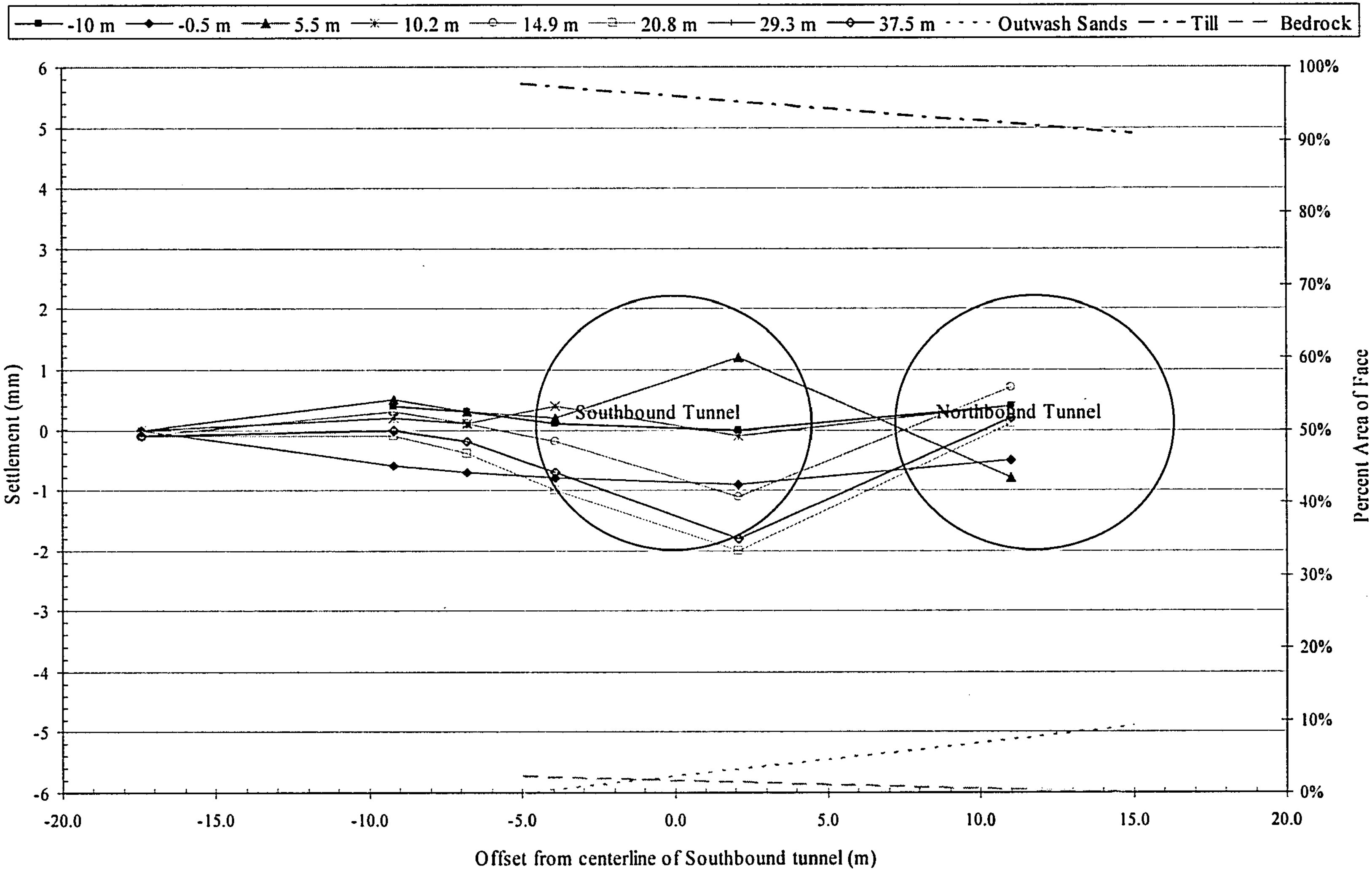

Figure H-5 Observed deep settlement at Array B during construction of Southbound Tunnel.

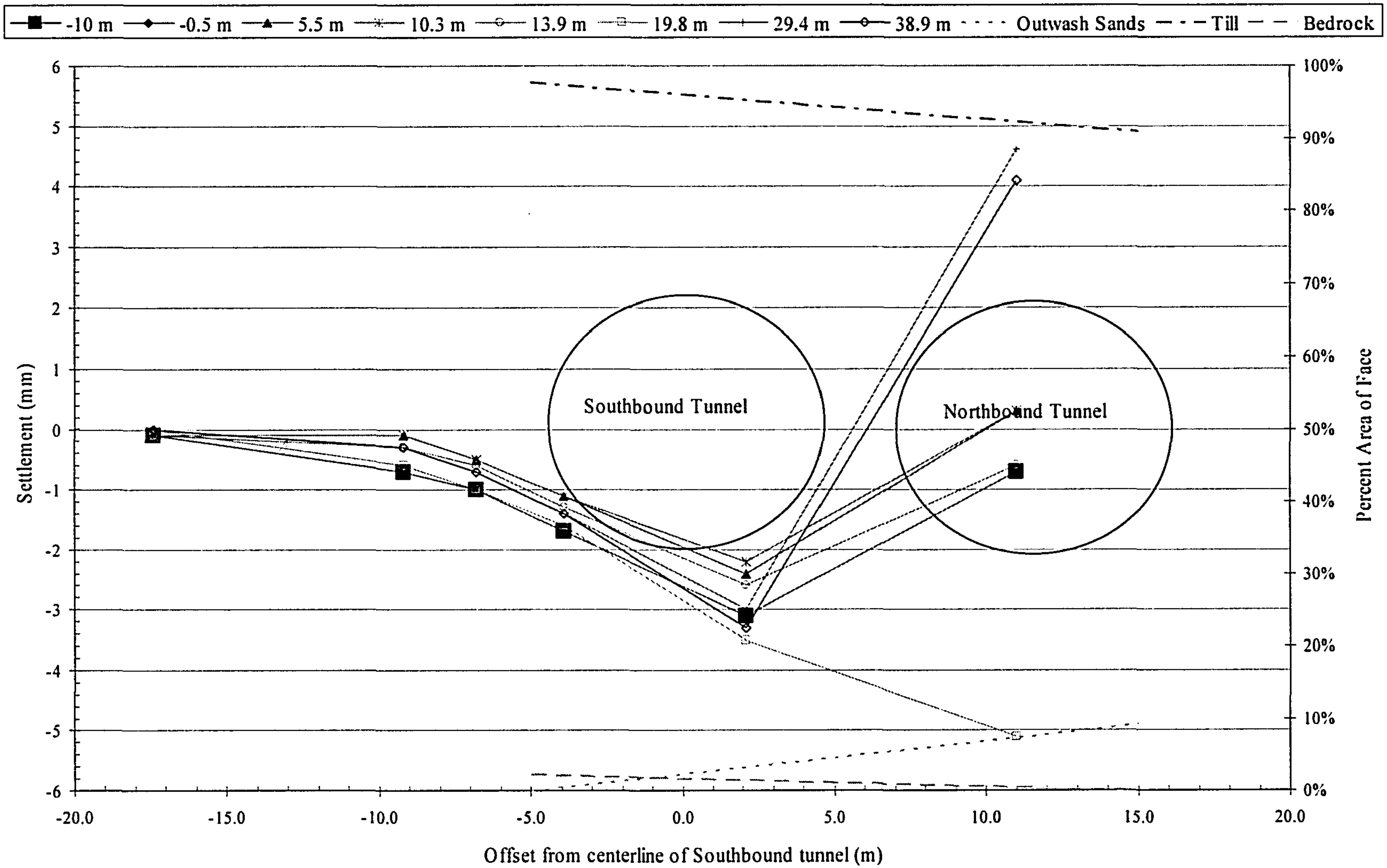

Figure H-6 Observed deep settlement at Array B during construction of Northbound Tunnel.

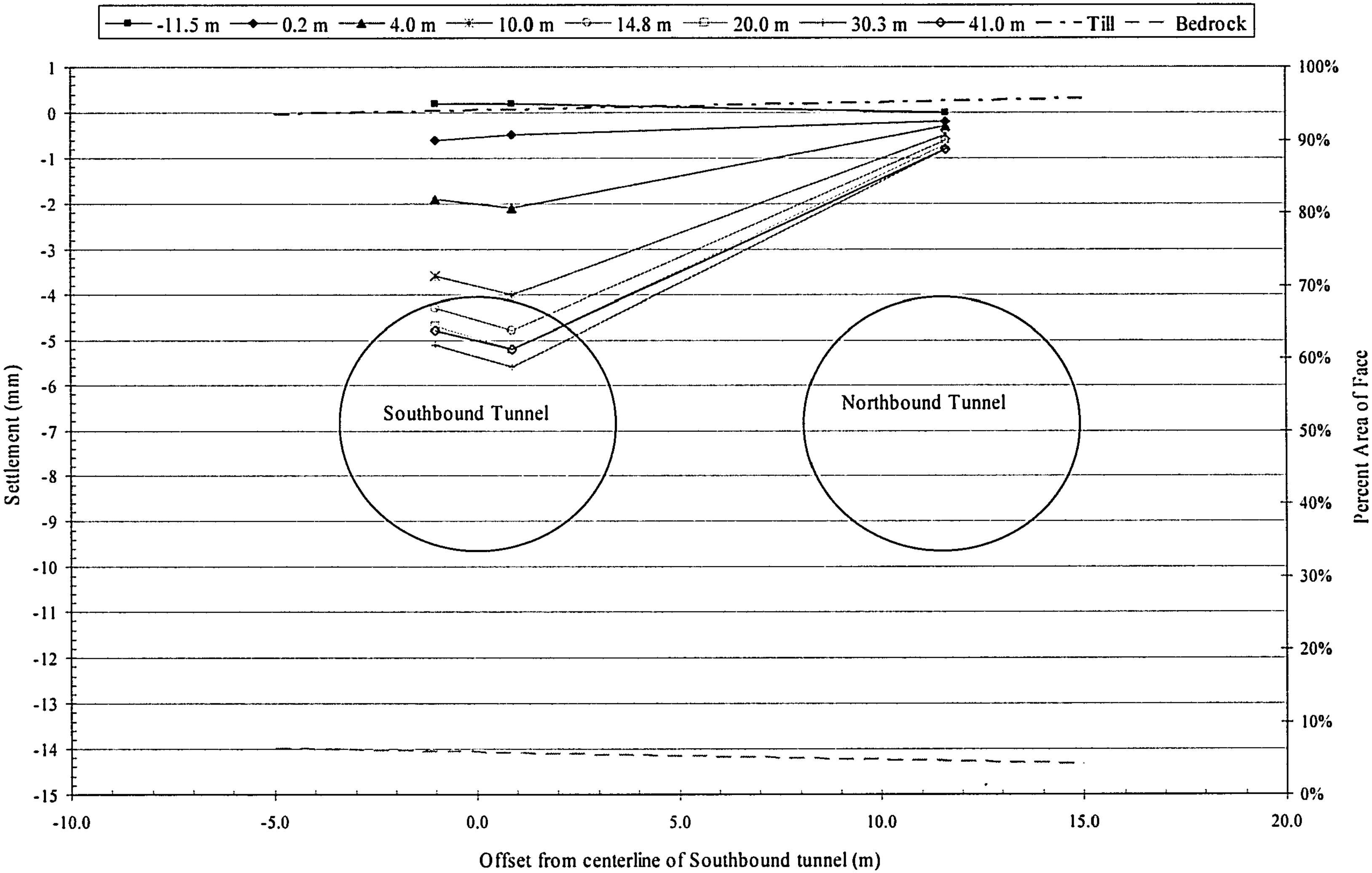

Figure H-7 Observed deep settlement at Array A during construction of Southbound Tunnel.

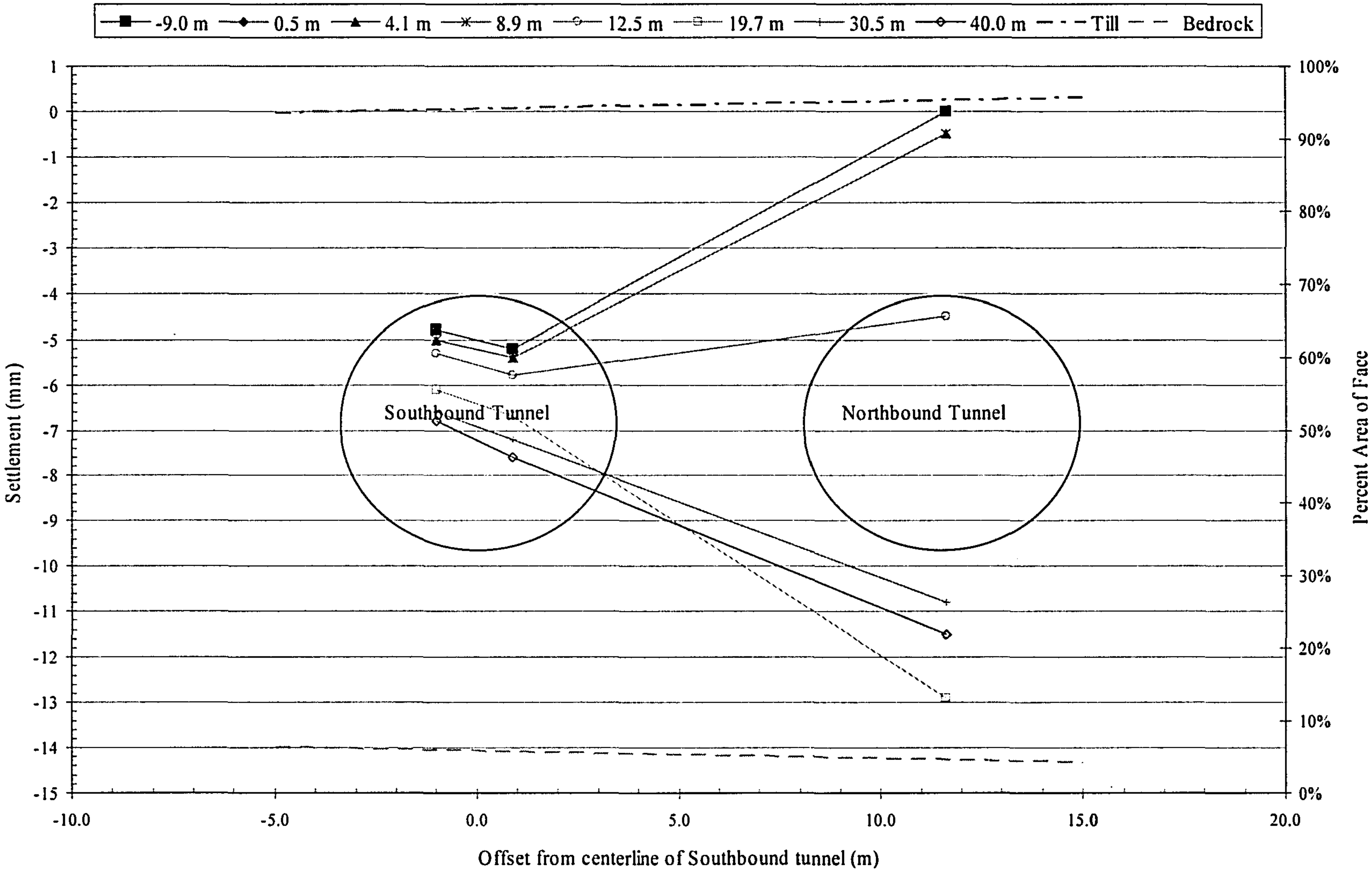

Figure H-8 Observed deep settlement at Array A during construction of Northbound Tunnel.

Appendix I – Building Array Settlement

This Appendix contains the building settlement Figures for the West Wall, and North Wall for the construction of the Southbound and Northbound Tunnels, and the Utility Corridor for the construction of the Northbound Tunnel.

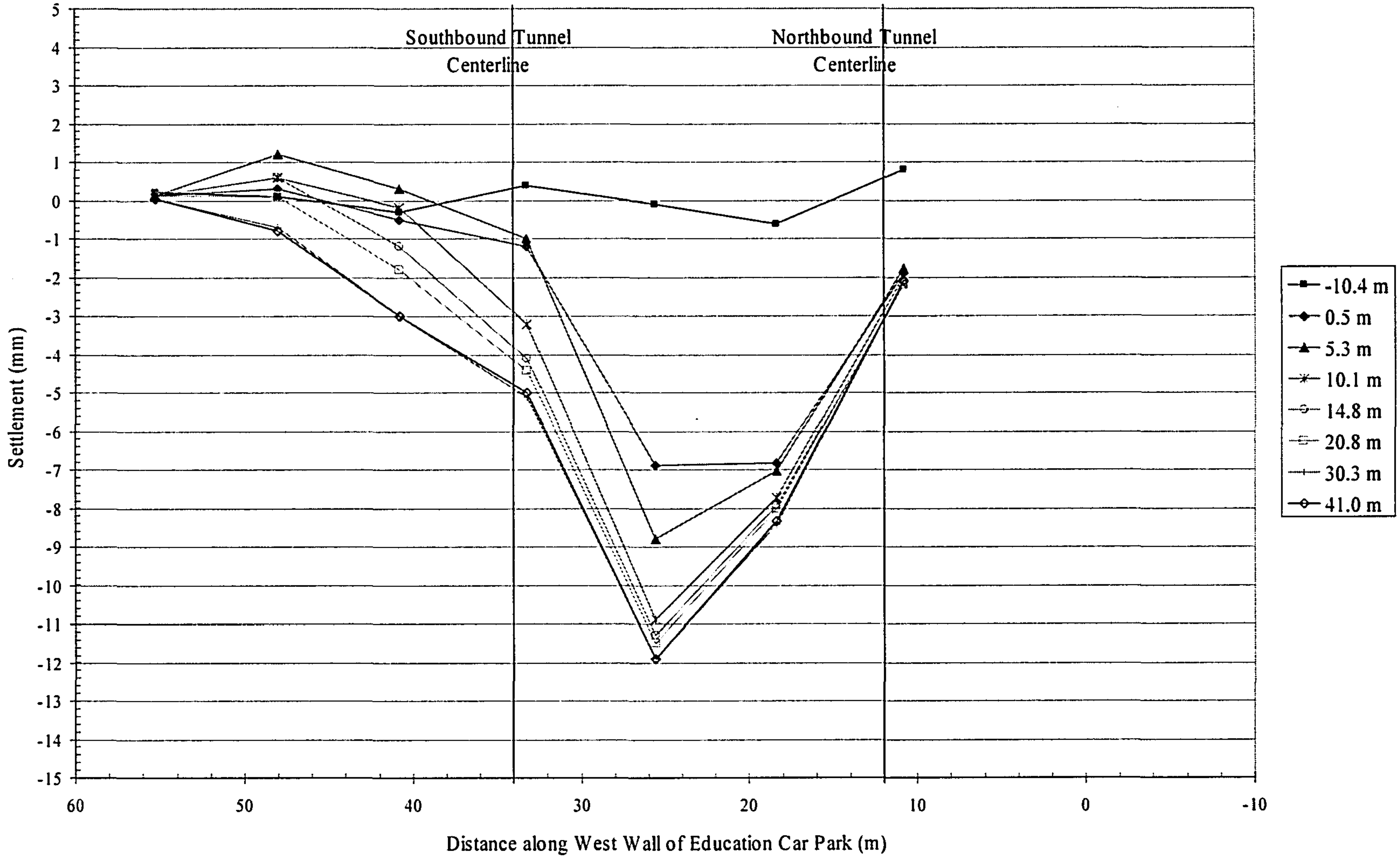

Figure I-1 Observed settlements along West Wall of Education Car Park during construction of Southbound Tunnel.

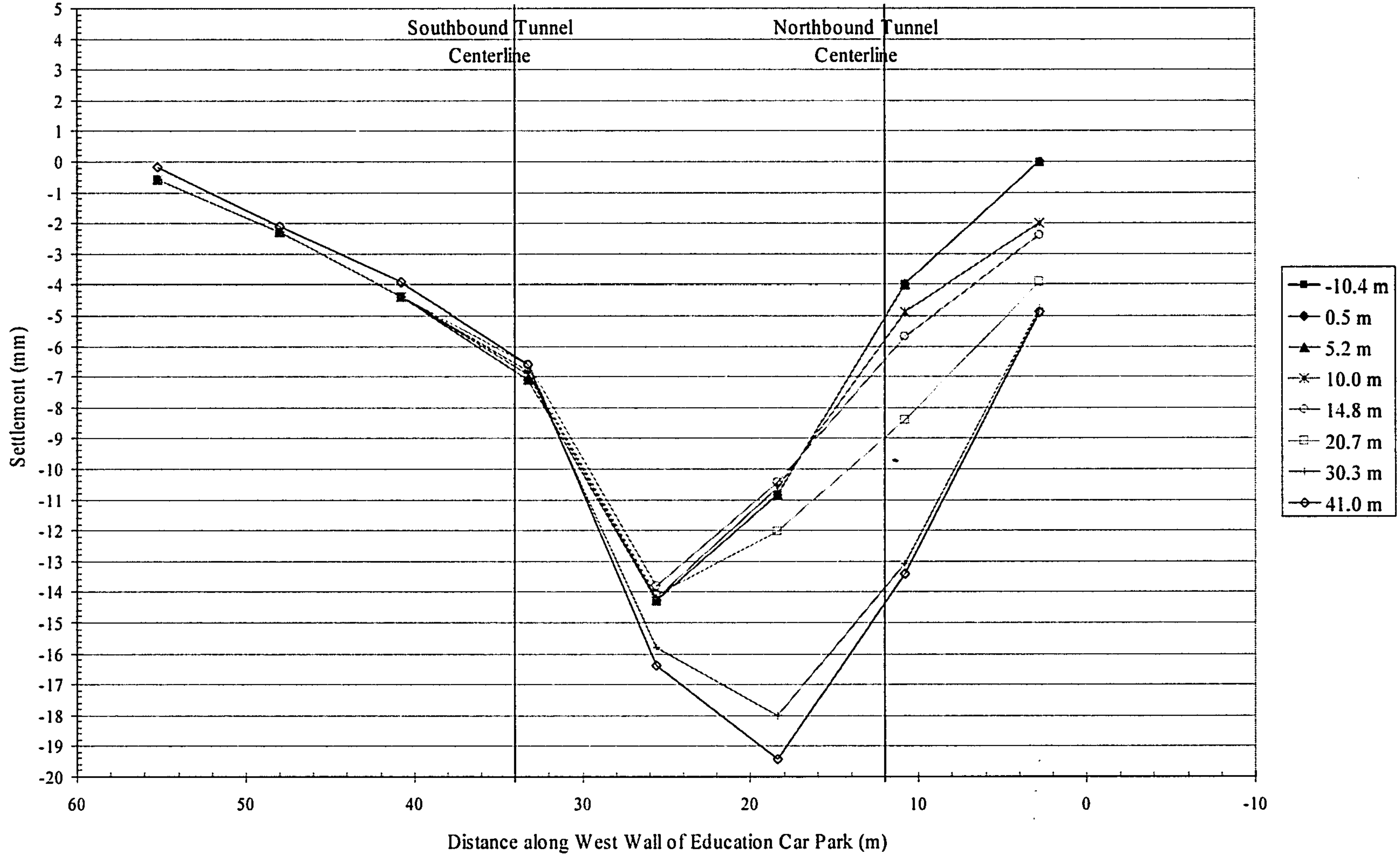

Figure I-2 Observed settlements along West Wall of Education Car Park during construction of Northbound Tunnel.

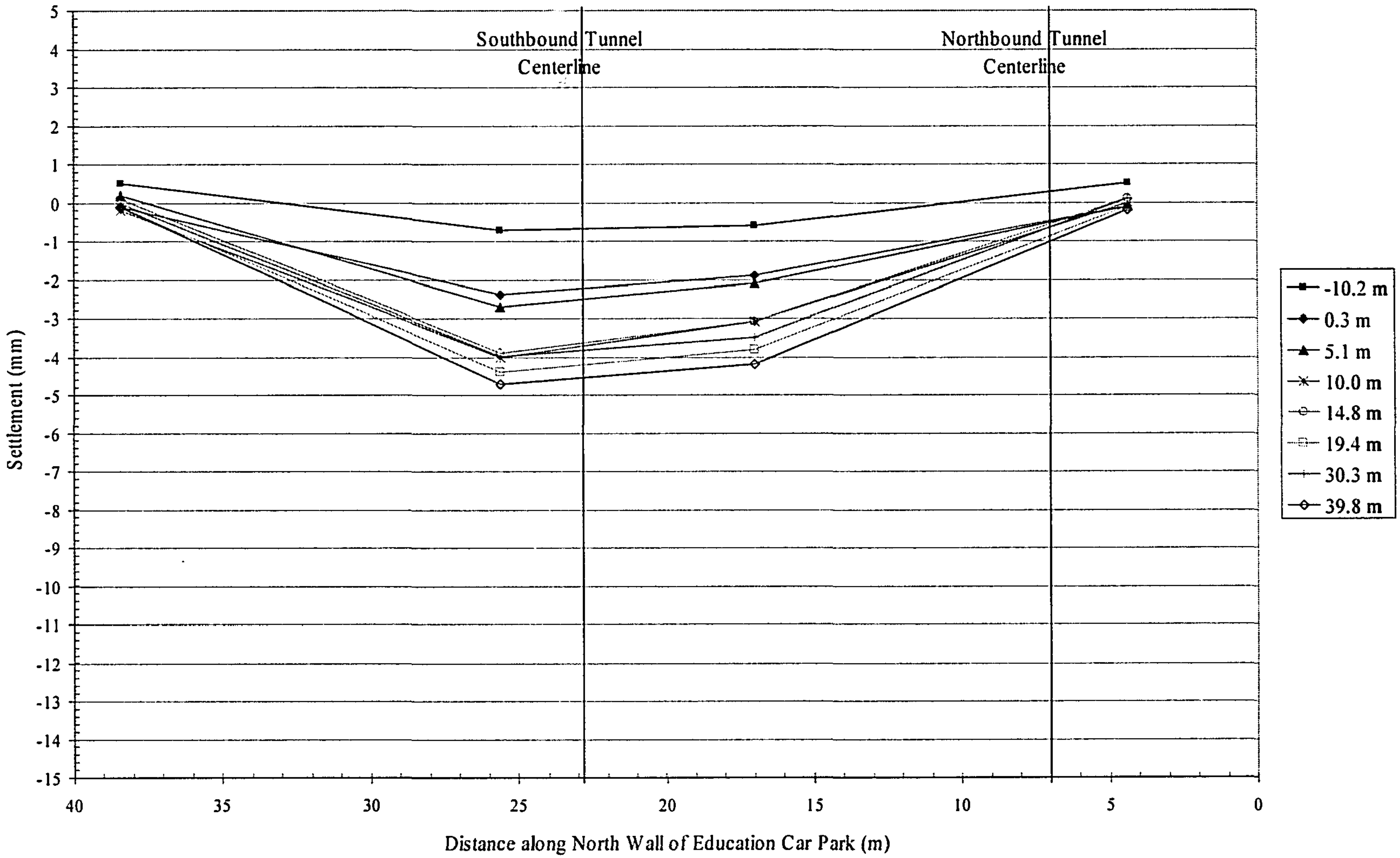

Figure I-3 Observed settlements along North Wall of Education Car Park during construction of Southbound Tunnel.

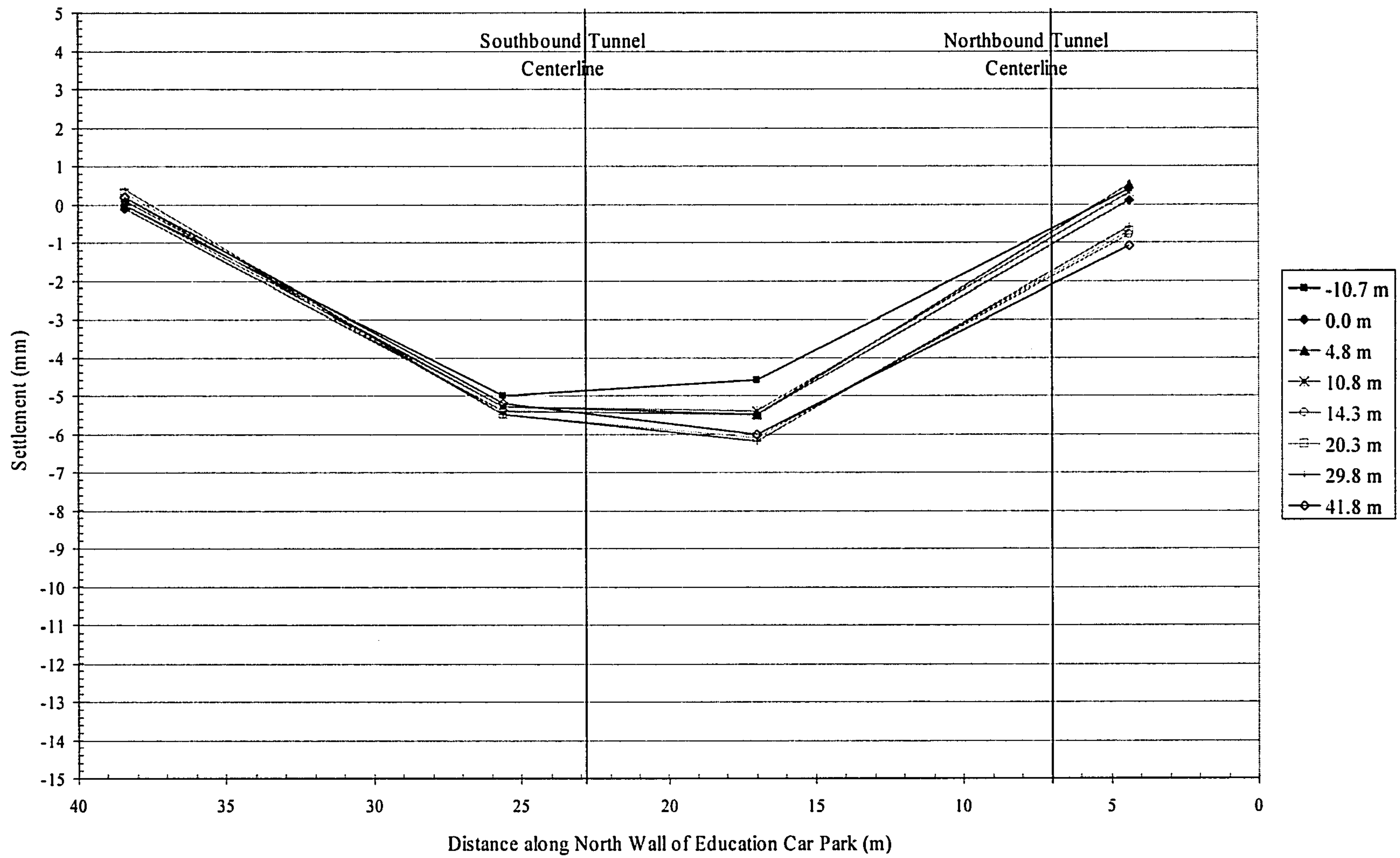

Figure I-4 Observed settlements along North Wall of Education Car Park during construction of Northbound Tunnel.

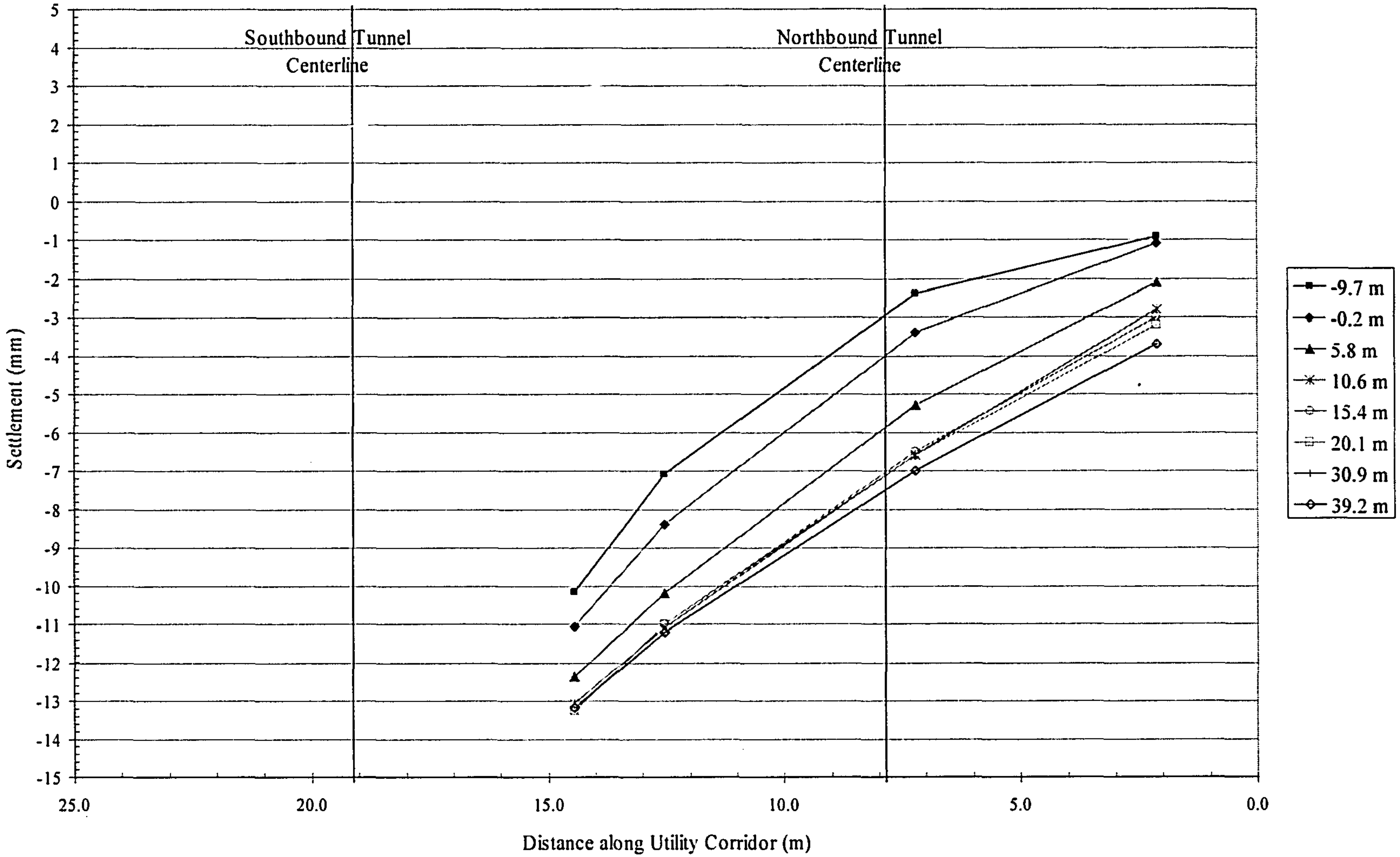

Figure I-5 Observed settlements along Utility Corridor on the approaches to Education Car Park during construction of Northbound Tunnel.