ABSTRACT

SHORING MONITORING - A CASE HISTORY OF SHORING DESIGN AND PERFORMANCE MONITORING AT A TORONTO WATERFRONT SITE

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The paper is presented in two parts: the first section provides a description of the typical shoring monitoring methods used in the Greater Toronto Area, while the second section presents a case history of the Pinnacle project and monitoring data. The case history highlights the vital link between design and performance monitoring of shoring systems. Emphasis is placed on the purpose and value of monitoring data in general and to the Pinnacle project specifically.

Toronto has experienced a recent intensification of waterfront area development on land reclaimed from Lake Ontario over a 100-year period. This land comprises a myriad of buried wharves and docks, generally of rock-filled timber cribs sitting on bedrock with hydraulic infill, and is saturated below 1.5 m.

Constructing underground basements is a challenge that is typically being solved with secant caisson cut-off walls supported by rock anchor tiebacks. The Pinnacle project is the most recent and is currently under construction. The Pinnacle project is distinguished from previous excavations because it incorporates five levels of underground parking, two of which are into shale bedrock. The shoring system supports adjacent streets and utilities, including the elevated Gardiner Expressway.

As part of the design process, excavation modeling was performed using the FLAC finite difference method. The modeling was used to confirm design assumptions, refine structural sizing and predict the influence of excavation upon neighboring structures. Mid-construction, the decision was made to extend the original 4 level excavation an extra level. Further modeling was conducted to investigate the performance of the shoring when extending the excavation below the toes of the soldier piles. Additional, deeper borehole inclinometers were installed to monitor the excavation below piles. Inclinometer monitoring provided vital information regarding shoring wall performance and verification of design parameters, including modeling predictions.

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Part 1: Monitoring of Shoring Performance

<u>Purpose of Shoring Performance Monitoring</u>

Monitoring the performance of shoring is an important component of the design and construction process. The shoring designer, contractor, owner and abutters can all benefit from the monitoring data.

Typically, the ground materials supported by shoring are not uniform. They are either created by nature in a random fashion or have been placed by man in a process that does not result in uniform properties. For design purposes, engineering parameters are chosen to model the overall behavior of these materials. There will always be some uncertainty in these parameters. We monitor the performance of the shoring to reduce the potential consequences of this uncertainty.

If the monitoring is well planned and executed, it allows confirmation of design assumptions during construction. Additionally, overall performance of the system can be monitored by measuring pile and wall movements. If the shoring designer has confidence that accurate monitoring will be undertaken, then more aggressive assumptions can be integrated into the design, resulting in reduced construction costs.

All parties involved benefit from reduced risk. Monitoring data can provide early warning when a shoring system is not performing as expected. This warning allows corrective actions to be taken prior to damaging abutting property or failure of the shoring system. Knowing how well the shoring system is performing enhances site safety and the safety of nearby structures, utilities or transportation systems.

Ideally, the monitoring program is designed to specifically address risk at potentially challenging locations. For example, if a sensitive structure or utility is very close to the excavation, frequent movement monitoring of the shoring and the structure itself can be performed to ensure that no damage occurs. Acceptable movement limits can typically be determined and alert levels defined at some percentage of the maximum acceptable movement. If movements reach alert levels, then the construction process is modified prior to there being more significant consequences.

The monitoring data is also valuable when addressing claims. Preconstruction condition surveys of abutting properties, vibration and movement data provide defense against nuisance claims.

During sensitive projects, the monitoring data can be used to keep stakeholders informed and thus reduce fears that the excavation process will result in damage to their structures.

All of these benefits represent cost savings and therefore, a well planned and executed shoring performance monitoring program will result in a less expensive project [(1)].

Components of a Shoring Monitoring Program

Pile Target Monitoring

The City of Toronto mandates that pile movements be monitored. This is typically achieved through some form of surveying. Our preference is to use three dimensional pile target monitoring with a total station.

We typically use reflective targets as both the reference points and survey targets. Targets are either installed at the tops of piles or at tie-back location to monitor tie-back performance. This method allows the displacement of each target to be determined in 3 dimensions to an accuracy of better than 3mm [(2)].

Precision Survey Monitoring

Sensitive structures, such as abutting buildings, vital utilities, roadway bridges and bents, railway and subway tunnels and stations in close proximity to the excavation can be monitored by precision survey techniques. Survey objectives are optically surveyed to determine the position of each target in 3-dimensions to an accuracy of than 2 mm [(2)].

Inclinometers

Inclinometers have been used for shoring monitoring in Toronto since the early seventies and provide the most useful data for accurately measuring ground movements [(3)].

An inclinometer casing is attached to a pile or grouted into a borehole. A wheeled probe is inserted into the casing that tracks machined alignment grooves which prevent the probe from rotating. The grooves are oriented to be aligned with the shoring wall. The probe measures tilt over its gauge length and the readings are summed to calculate the movement along the casing length. The casing must be installed to sufficient depth to ensure the bottom is a fixed point or the top of the casing surveyed to determine how much translational movement is occurring.

Electrolevel Beam Sensors

Beam sensors are tilt sensors that are mounted on a rigid beam with a defined gauge length, typically 1 to 2 meters long. Each end of the beam is anchored to the structure. The beam simplifies conversion of tilt to millimeters of movement (settlement, heave, convergence, or lateral displacement). Also, beam sensors can be linked end-to-end to monitor differential movements and provide absolute displacement and settlement profiles [(4)].

Strain Gages

Strain gages can be attached to struts, tie backs and walers to measure deformation, allowing calculation of loads [(4)].

SMART Cables

SMART cables are instrumented tie-backs capable of providing data regarding tie-back elongation versus depth. This data may be used to calculate the tie back load distribution versus depth.

Load Cells

A load cell is a transducer which converts force into a measurable electrical output. Although there are many varieties of load cells, strain gage based load cells are the most commonly used type. Center-hole load cells are designed to measure loads in tiebacks, rock bolts, and cables. These are used for proof testing and performance monitoring of tiebacks, rock bolts, and other anchor systems [4].

Pre-Construction Condition Survey or Defect Summary

A pre-construction condition survey is used to thoroughly document all visible defects on exterior and interior surfaces of adjacent structures within the zone influenced by the excavation. It provides valuable documentation in the event of construction impact on the adjacent structures and/or exaggerated or '"nonsense" claims by neighbors.

Vibration Monitoring

Traditionally vibration level measurement in the Toronto area has been carried out to monitor for potential damage to nearby structures. Recent projects have demonstrated that environmental effects, such as lost productivity at neighboring office buildings, must also be considered in selecting the construction methodology and monitoring programs. Vibration monitoring equipment consists of seismographs capable of measuring and recording the Peak Particle Velocities (PPV) in the transverse, longitudinal, and vertical directions.

Part 2: Case History

Historical Background

Toronto's waterfront is characterized by a flat area some 1.5 km wide by 6 km long comprising ground reclaimed from Lake Ontario. The original shore line remains visible as a sharp rise on all north-south streets leading up to Front Street which is some 5m higher in elevation than the reclaimed land.

The land was gradually reclaimed from the lake over a 100 year period beginning in the early 1800's. What began as a series of docks and wharfs was filled in order to create more space and to provide deeper anchorages for shipping. By 1950, this advance had reached the present shore line to the south, with the northern part comprising extensive railway sidings, and with slips, warehouses and grain elevators along the water's edge. In the 1950's, urban planners decided to relocate the commercial harbor further east and consolidate the railway to a corridor along the northern edge, thus freeing the lands for development. The elevated east-west Gardiner Expressway was constructed on drilled pier and driven pile foundations through the middle of the flat lands.

This was followed in the 1960's by new buildings near the lake: the Toronto Star building and the Harbour Castle Hotel, both founded on drilled caissons. Parking was provided in above-grade multi-storey garages. A number of condominium apartment buildings were then built along the north side of Queen's Quay at the edge of the lake, all without basements.

Within the 'railway lands' north of the Gardiner Expressway, the CN Tower, the world's tallest free-standing structure, was erected in 1974, and the Skydome Stadium, with the world's first retractable roof, was built in 1986.

The Admiral Hotel, built in 1984, was the first building to be designed with underground parking on the edge of the waterfront. At this site, just east of Jarvis Street, the underlying Dundas or Georgian Bay Shale bedrock is at a depth of 9m (30 feet) with overburden consisting predominantly of silty sand hydraulically placed fill and saturated below 1.5m (5 feet). Three levels of underground parking provided a convenient depth for spread footing foundations on the bedrock. A secant caisson wall was selected to provide a permanent dam against ingress of lake water, and act as both temporary shoring and the permanent basement wall. It was found that a 1m (3 foot) toe into the rock was sufficient to form a permanent water seal, due to the tight nature of any vertical fissures in the horizontally layered shale. This project set the pattern for subsequent development, and some ten condominium apartment projects have since been built, generally with 3 levels of underground parking, using a similar secant wall technique.

The Pinnacle Project

The site, 180m long by 90m deep (600x300 ft), occupies a city block, bounded by Bay Street to the west, Yonge Street to the east, Harbour Street along the south and by the elevated Gardiner Expressway along the north. The project comprises four 35-storey residential towers with five levels of podium. Originally, there were to be four levels of underground parking. It was planned to be constructed in two phases with the easterly 60% of the underground structure built in the first phase.

The ground surface is relatively flat at elevation 77.2m. The depth to rock falls about 1m from north (8m) to south (9m). The fourth basement, with a maximum excavation depth of 13m, was therefore to be entirely in shale bedrock.

The main structure of the Gardiner Expressway, founded on drilled caissons on the rock, is located some 20m north of the site, with two east-bound ramps, the Bay Street on-ramp and the Jarvis Street off-ramp, located between the main structure and the site. These ramp structures are founded on precast concrete piles driven to rock. The nearest ramp, the Jarvis Street off-ramp, has nine piers bordering the site, six of which are adjacent to phase 1. The retaining wall structures at the base of the other ramp, located in phase 2, are founded on driven timber piles.

Overburden soils are predominantly hydraulically placed sand fills, saturated below 1.7m and contain a number of known abandoned dock structures.

Shoring Geometry

The chosen shoring system comprised a drilled secant caisson wall that doubled as a groundwater cut-off barrier. A number of alternative layouts were examined based on both the consultant's and specialist shoring contractor's previous experience of neighboring projects. The selected layout comprised 1067mm (42") diameter caissons at 867mm (34") centers, allowing a 200mm (8") nominal interlock. Every third caisson was reinforced with a W610 (W24) structural steel beam (soldier pile), and these caissons were drilled into the rock to one meter below final excavation level, resulting in rock sockets some 5 meters deep.

Lateral bracing was provided by one level of rock anchors inclined at 45 degrees located near the top of wall in order to penetrate the wall above the water table, and by a rock anchor within the rock, designed to reduce deformation and minimize the steel beam weight.

The intermediate unreinforced caissons were terminated 1.8m (6ft) below rock surface, estimated to be the minimum for assured sealing against groundwater, thus ending about 2m to 3m above final excavation.

A modified geometry was employed at the Gardiner Expressway piers, where greater stiffness was desired, and where the existing battered foundation piles obstructed the use of tiebacks. Similar caisson size and spacing was employed but with every second caisson reinforced, and intermediate caissons drilled to final excavation level. In Phase 1, the existing piers are evenly spaced at 17.25m (71ft 3in), allowing use of a repeating pattern comprising groups of 6 of the closer spaced piles at each pier with 5 bays of standard shoring in between. The group of six piles at each pier was braced by a continuous steel waler anchored by tiebacks at each end attached to the outer two soldier piles. We chose to bury the waler behind the wall rather than have it exposed on the face in the conventional position.

Temporary shoring along the west face of Phase 1 was provided by a steel sheet pile wall, with the toe driven into the top of the shale bedrock. Lateral support was provided by one row of rock anchor tiebacks connected to a continuous waler, and by toe pins in every second in-pan. A generous bench of bedrock was left between the phase line structural wall and the sheet piling location.

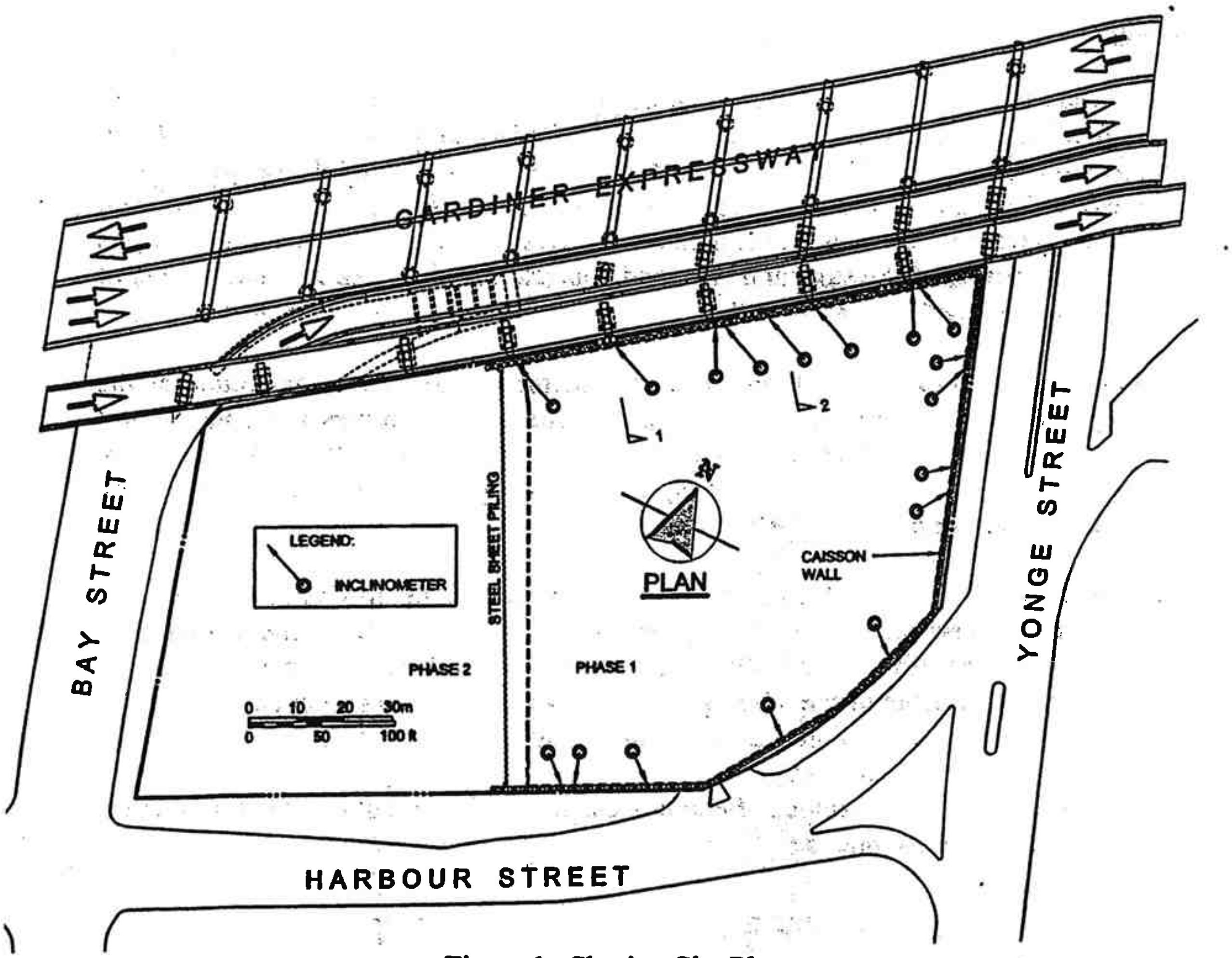


Figure 1: Shoring Site Plan.

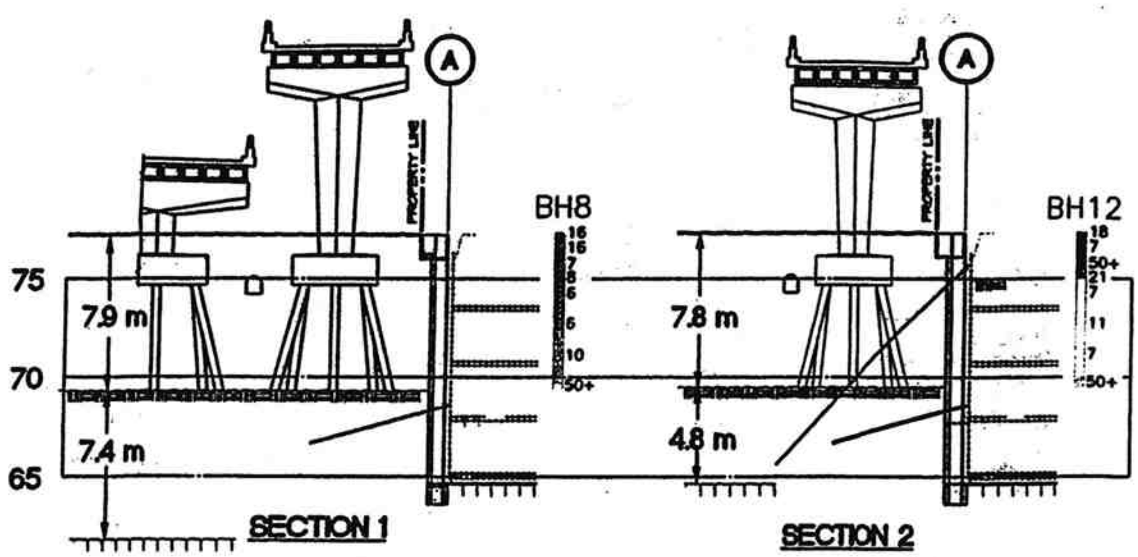


Figure 2: Sections showing four basement excavation at the Gardiner Expressway.

Design

The caisson wall was located to act as an outer form for the poured concrete basement walls after being trimmed back to a straight line at the face of the soldier piles. A 28 day concrete strength of 4 MPa was specified based on previous experience with water-tightness and trimming of similar walls. Structural design to select tieback loads and steel beam sizes was based on conventional earth pressure loading diagrams, and analysis of composite action of the structural steel and concrete based on deformation records of several nearby projects.

Two-dimensional FLAC[5] analyses were also performed for a typical section at the Gardiner Expressway piers and for two typical street sections representing the highest and lowest bedrock surface elevations. FLAC analysis enables examination of interaction of the structure and soil system, and provides estimates of structural performance and overall deflections.

The process of advanced modeling (using finite element or finite difference solutions of a stiffness matrix describing the geometry) permits the 'testing' of the design assumptions with the added benefit of parametric analysis for worst case or sensitivity testing. Estimates of movements of the shoring and neighboring structures may be made which allow comparison with actual movements recorded by the monitoring process.

The modeling indicated maximum lateral movements of 12 mm out of site at the pile top and 2 mm into site at the pile toes.

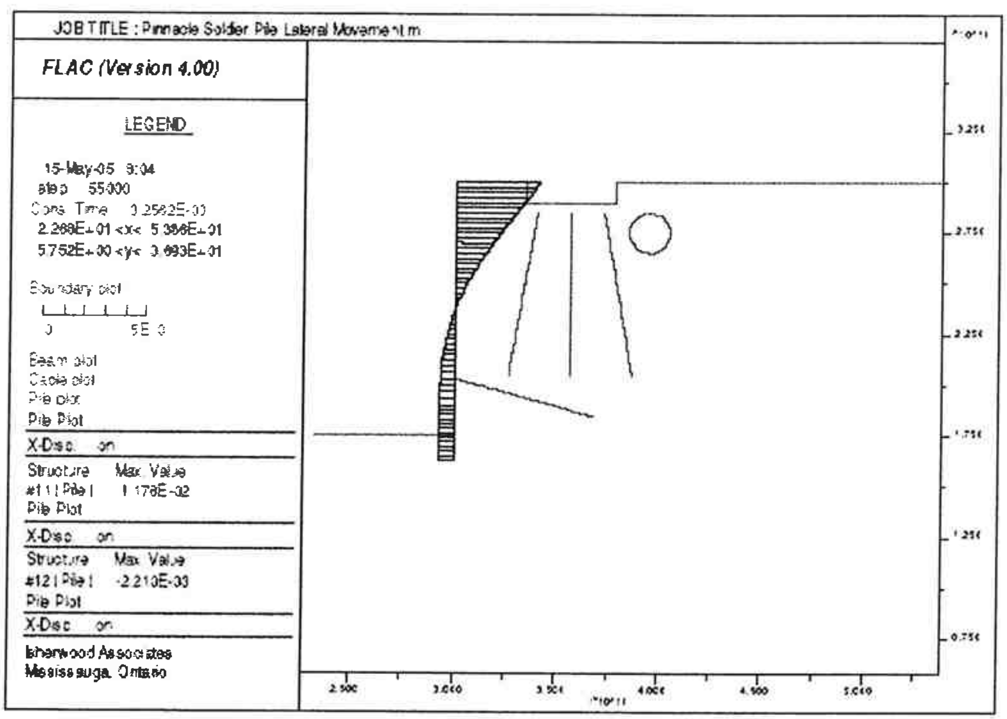


Figure 3: Pile movement at original final excavation level (four basements).

Monitoring

Inclinometers attached to the soldier piles provided the principal monitoring of the shoring system. These were installed in front of each of the six piers and at six other representative wall locations.

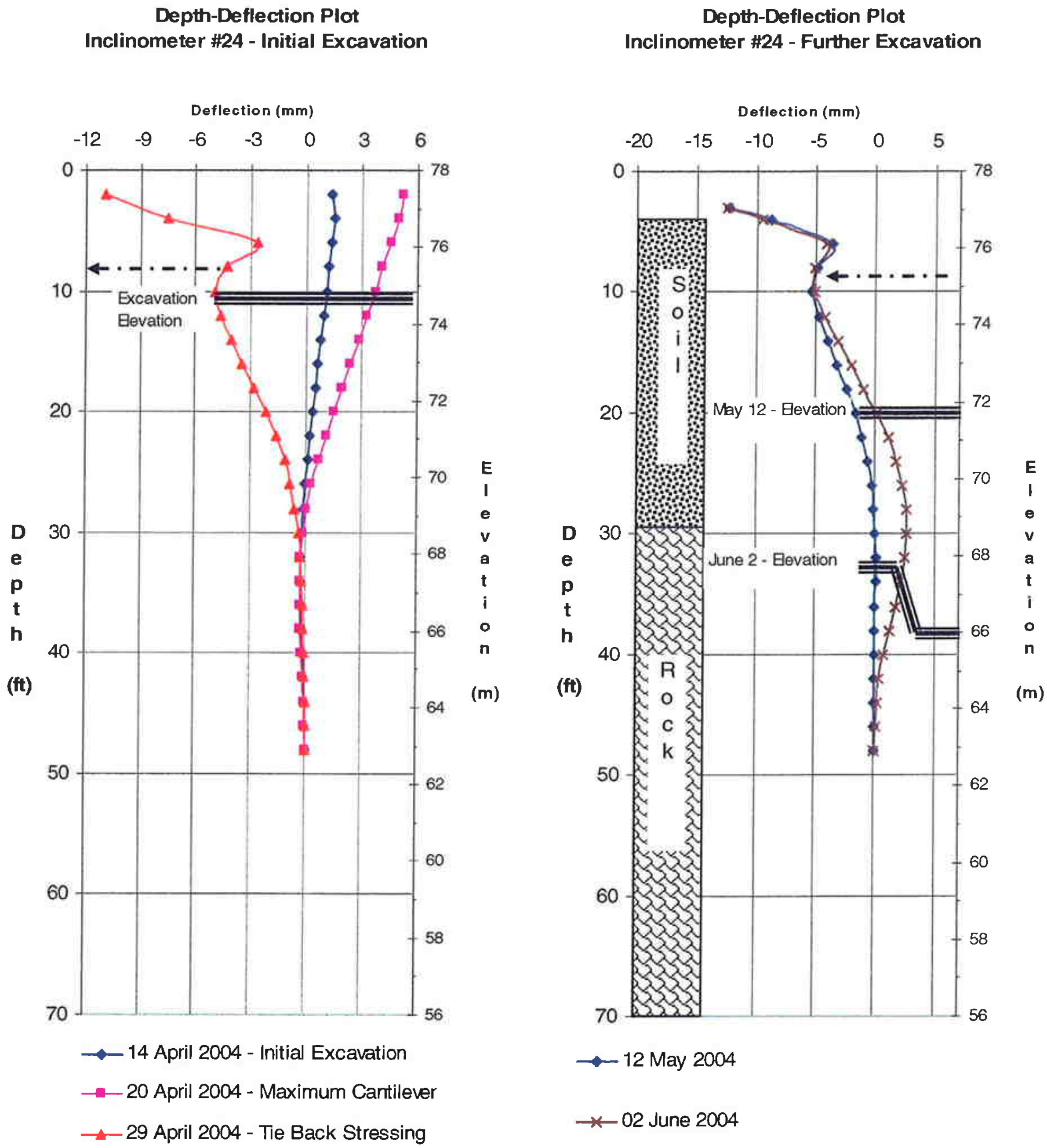
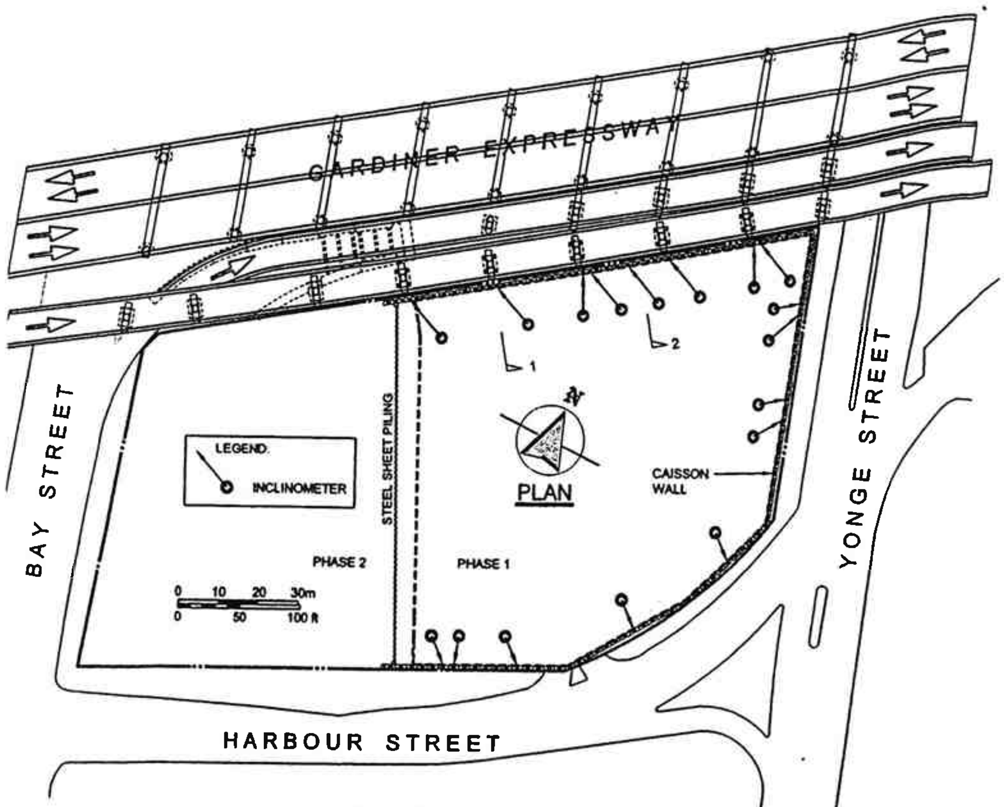


Figure 4: Four Basement Level Inclinometer Plots



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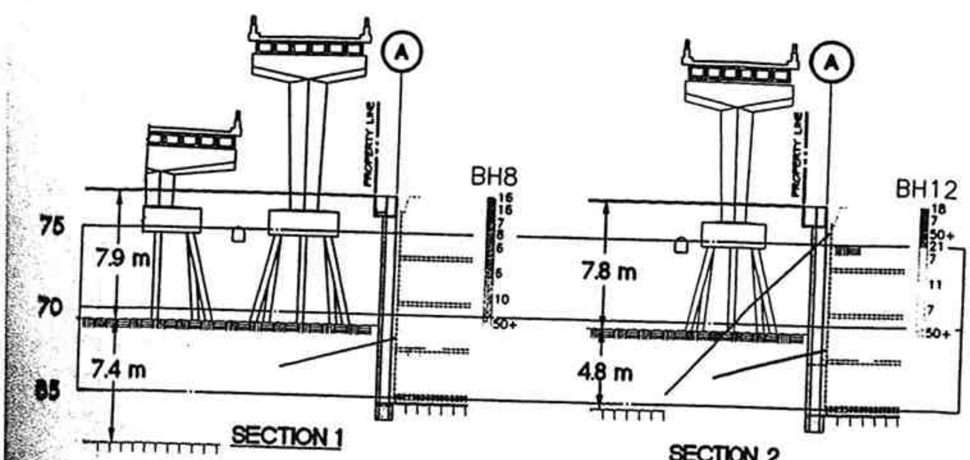

Figure 2: Sections showing four basement excavation at the Gardiner Expressway.

All inclinometers provided similar patterns of movement related to stages of the excavation and tieback stressing. The initial excavation to 2m to allow tieback installation resulted in cantilever movements of generally 10mm at the streets, with movement extending down to the rock. In two locations this was as high as 20mm. This deflection was recovered by tieback stressing with top of wall moving out of site to a maximum of 20mm.

The stiffer shoring at the Gardiner piers showed cantilever movements of 5mm or less with recovery during stressing to generally 5mm out of site at the bracing elevation as illustrated by the following plots. As excavation advanced a classical bowing inwards of the wall occurred below the brace, with a marked increase in deformation as excavation penetrated the rock. The net inward movement at each of the piers reached 2 to 3mm.

Fifth Basement

After the secant caisson and sheet pile walls for phase one were installed and the excavation well advanced, the client raised the question of adding an extra basement that would take the excavation well below the toes of the existing shoring.

Complicating this was the phenomenon of locked-in horizontal stresses in the rock[6]. This was discovered some forty years ago during water intake tunneling under the lake, where cracking of the inside of poured-in place concrete linings occurred at the spring line rather than at the crown where conventional theory would predict. The phenomenon has been studied at several subsequent excavations deep into the rock.

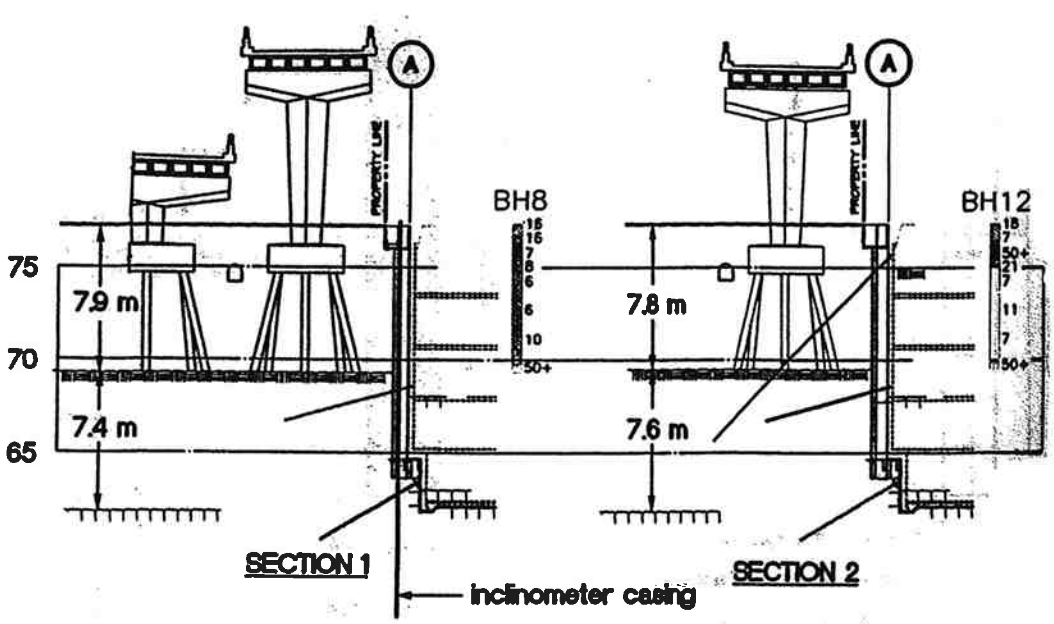

Figure 5: Sections showing five basement excavation at the Gardiner Expressway.

Several methods of underpinning the caisson wall shoring were studied and priced. The eventual method adopted was to move the fifth basement perimeter in by 750mm (30 in) leaving a rock ledge to protect the original toes of the wall. Pre-shearing by line drilling was specified, together with an additional level of rock anchors drilled immediately below the toes of the existing soldier piles. This took the form of a 62mm (2 ¼") Dywidag bar anchored to a 450mm (18") diameter circular steel plate recessed into the pre-sheared face. At the sheet pile wall, the rock ledge was reinforced with rock pins, shotcrete and mesh to secure the additional height.

As part of the scheme additional inclinometers were installed at four soldier piles, located immediately adjacent to existing inclinometers, by drilling vertically through the caisson wall concrete and then into the underlying bedrock to a depth of 22m (72 ft). These served to monitor rock expansion as the deeper excavation relieved the locked-in stresses in the rock.

Additional FLAC modeling was conducted to predict the behaviour of the shoring system with the additional excavation. The analysis assumed locked-in rock lateral stresses[6] of 200 kPa per m of depth (1 MPa per 5 m). Modeling of the increased excavation depth with the additional rock anchor indicated pile base movement of 7 mm into the sit

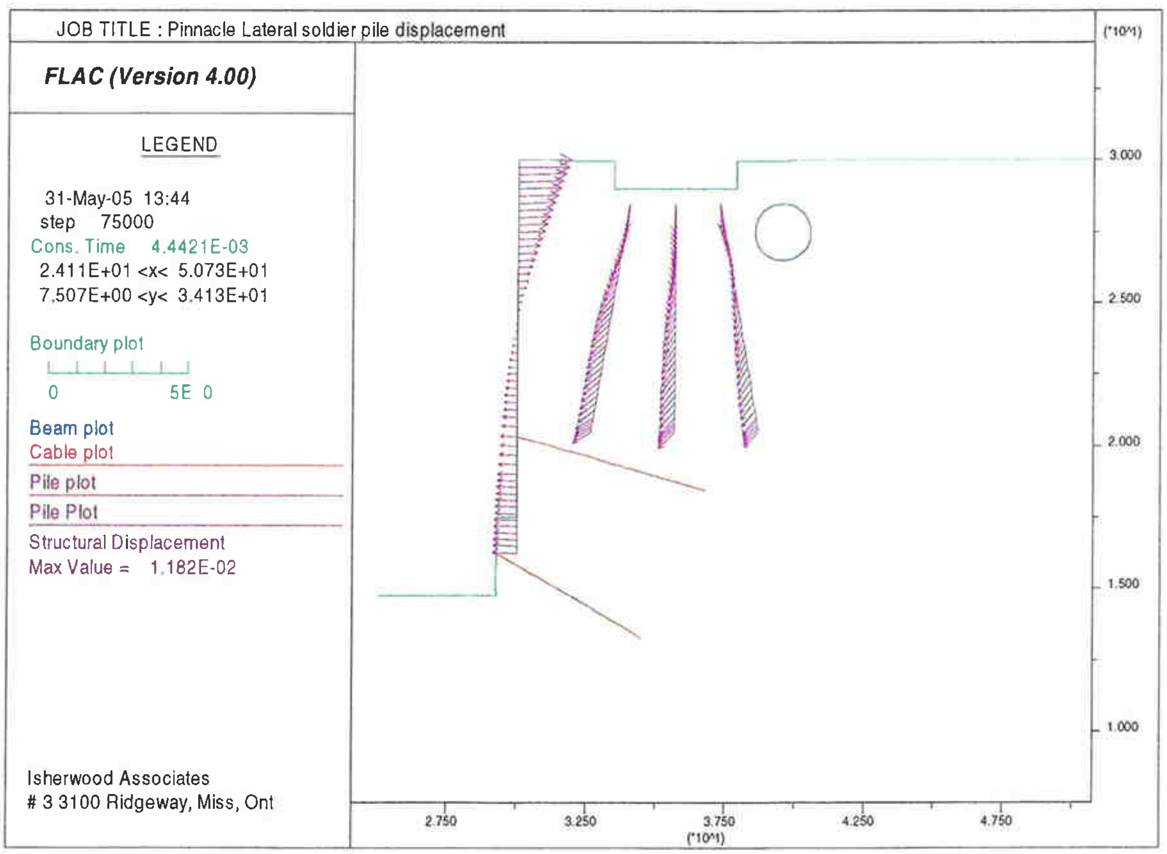

Figure 6: Soldier pile lateral movement at extended excavation depth.

The predicted soil and rock mass movements are provided in Figure 7 below. The caisson wall and rock anchors appear to translate towards the excavation, without substantial rotation.

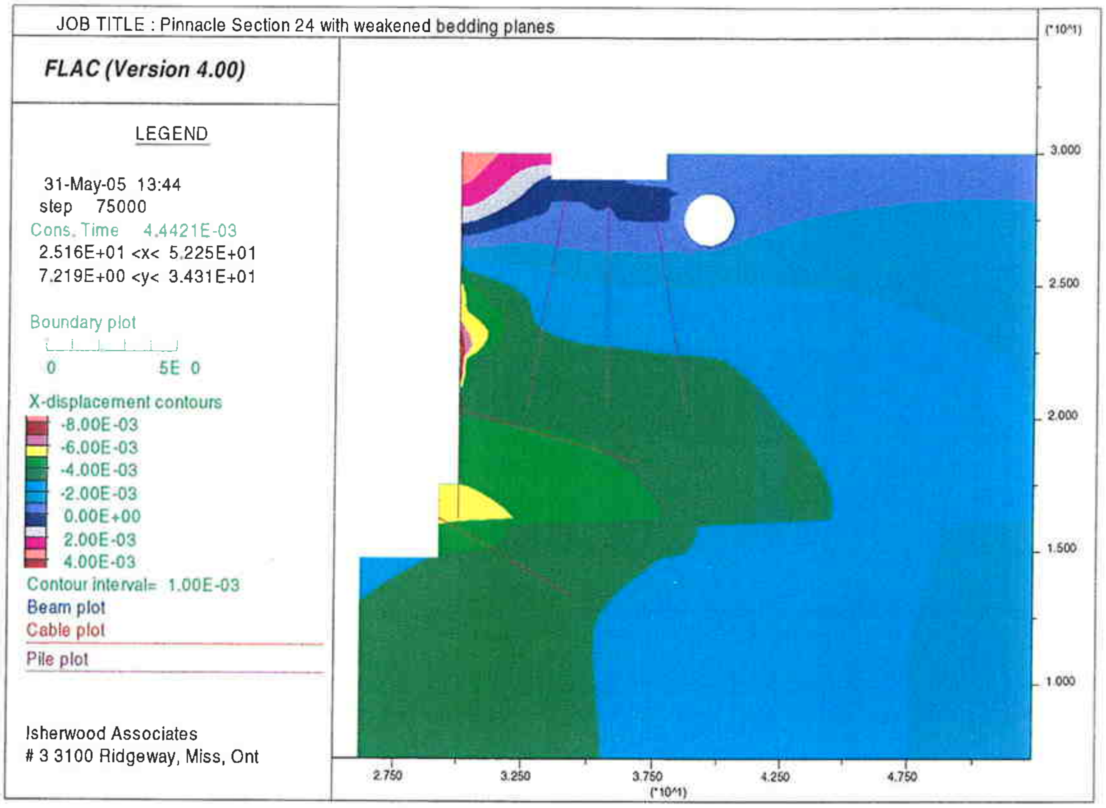

Figure 7: Lateral soil/rock displacement (1mm contour).

Monitoring

As the excavation advanced into the rock, the original inclinometers at the piers indicated the top of the wall was progressively moving out. However, the deeper adjacent inclinometers showed that this was an illusion, as in reality there was a pronounced movement inwards of 3 to 7mm at the base of the wall. The rock is shale with well defined horizontal jointing and it appeared the rock expansion was causing sliding at bedding planes near the base of the excavation. If one looks closely at these plots, other slippages of about 1mm are apparent at other levels in the rock below the excavation.

The original inclinometers still provide a reliable indication of the deformation profile over the depth of the wall, except that they could not pick up translation of the toe of the wall. On the other hand, the deeper ones provide the deformation picture below the toes, whilst reflecting only recent changes above that level. A more accurate overall picture is obtained by combining the two as illustrated in the following plots.

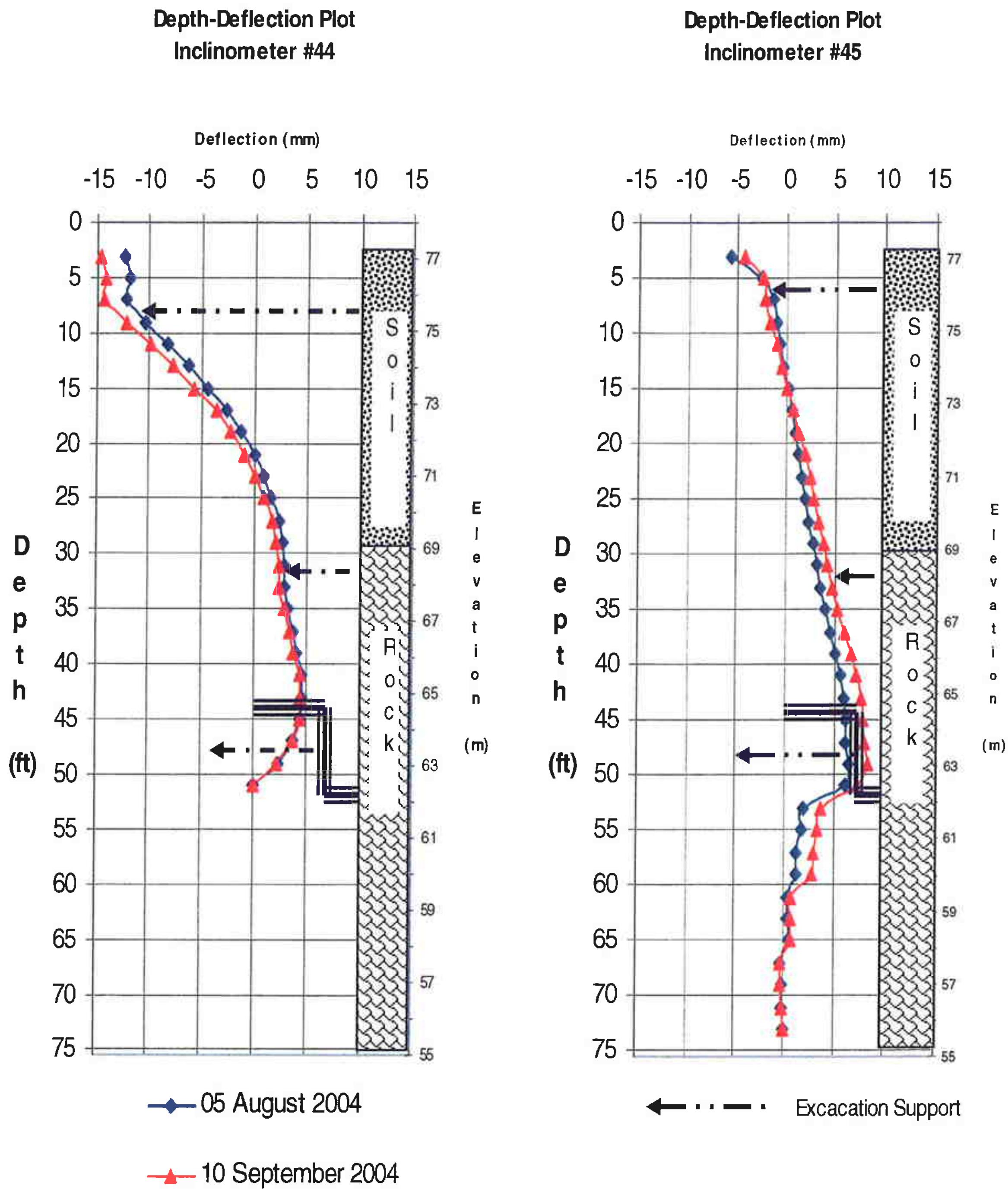

Figure 8: Five Basement Level Inclinometer Plots – Shallow and Deep Inclinometers.

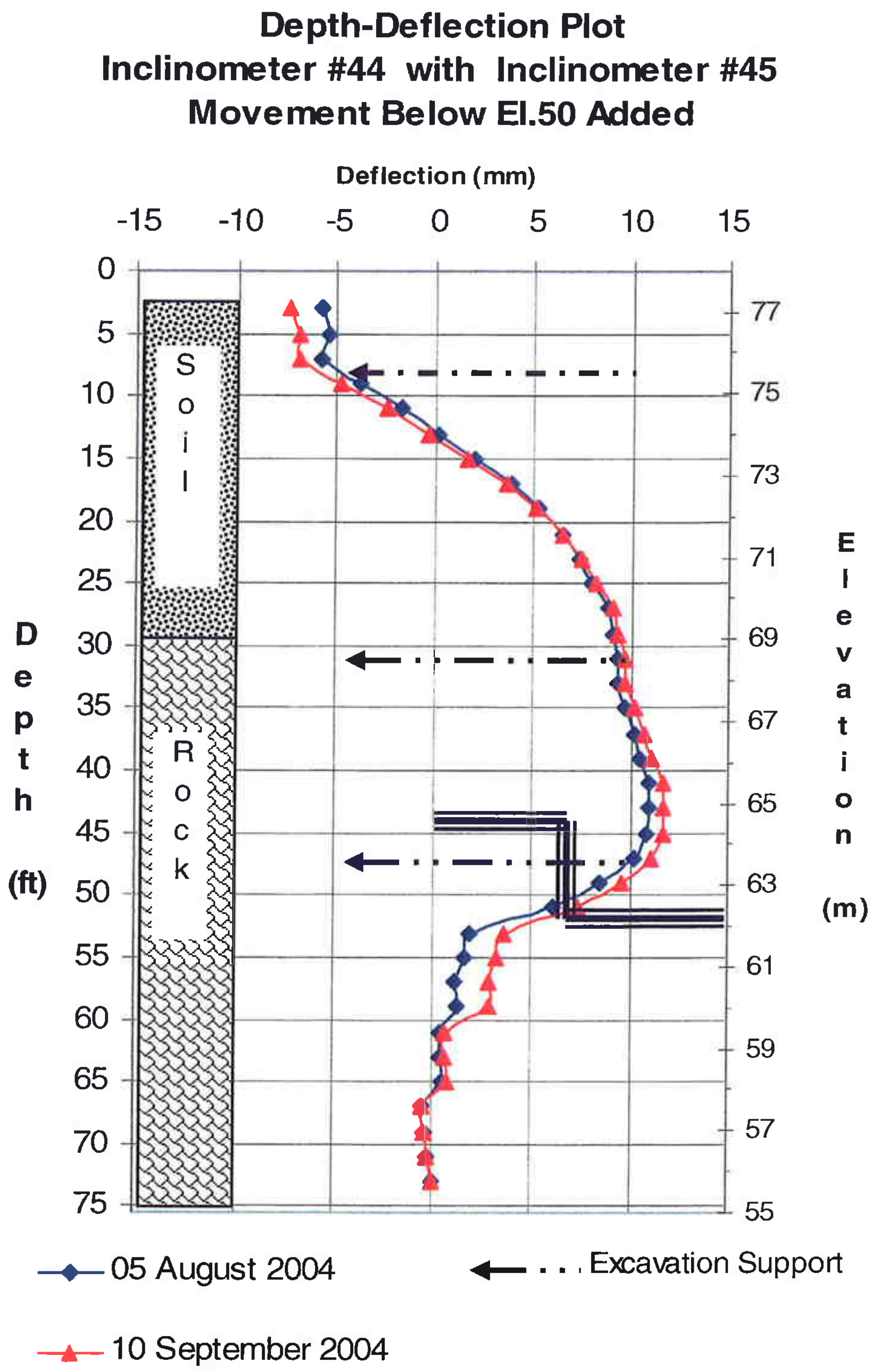

Figure 9: Adjusted Data Combining Deep Movement to Shallow Movements.

Conclusion

The FLAC analysis predicted lateral translation due to the increase in excavation depth below the pile toes. Incorporating the locked-in lateral rock stresses into the model resulted in approximately doubling of this translation. Yet the observed pile and rock translation exceeded the FLAC predictions by a factor of 2. The nature of the observed movement indicates shifts are occurring at distinct layers within the rock near the base of the excavation.

The effect of the deep rock movements on the Gardiner Expressway piers is not significant since differential movement in the rock mass is distributed back some distance, as indicated by the smaller translation experienced in the tieback anchorages centered some 8.5 m behind face of wall. The overburden and structures within it experience a mild lateral expansion spread over a distance of several meters.

The inclinometers gave valuable performance information at every stage of the excavation, providing confidence that preload values were appropriate, and that adjacent streets and structures were not being adversely affected by the excavation.

The deep inclinometers provided a useful picture of movement in the rock mass as the locked-in stresses were relieved.

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