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Belshaw, D. J.; Palmer, J. H. L.

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RESULTS OF A PROGRAM OF INSTRUMENTATION INVOLVING A PRECAST SEGMENTED CONCRETE-LINED TUNNEL IN CLAY

by D. J. Belshaw and J. H. L. Palmer

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Results of a program of instrumentation involving a precast segmented concrete-lined tunnel in clay

D. J. BELSHAW

Morton, Dodds & Partners Limited, Consulting Geotechnical and Geological Engineering, 50 Galaxy Boulevard, Rexdale, Ont. Canada M9W 4Y5

AND

J. H. L. PALMER

Geotechnical Section, Division of Building Research, National Research Council of Canada, Ottawa, Ont., Canada K1A 0R6

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In Thunder Bay, Ontario, a 2.16 m diameter tunnel is nearing completion in a soft to firm clay using a full-face tunnel boring machine and an unbolted precast segmented concrete lining. The lining consists of rings composed of four trapezoidal sections of unreinforced concrete, 1 m in length and 11 cm thick. Each ring is assembled within the protection of the tailpiece of the tunnel boring machine and clay grout is injected behind the ring as the machine advances by thrusting on the completed lining.

Because of the uniqueness of this method of construction, an extensive field instrumentation program was undertaken to evaluate the tunnelling procedure and the performance of the completed tunnel. The first phase of instrumentation involved a 60 m length of the tunnel and included observations of ground surface deformation, soil deformation (as measured by slope indicators, magnetic extensometers, and deep settlement points), total pressure on the lining, and deformation of the lining. The information also included tunnelling procedure and weight of spoil removed as the tunnel boring machine passed through the instrumented area.

These observations are presented and discussed. Some problems of instrumentation are considered, particularly as they related to the second array of instrumentation which was designed to provide detailed information on spatial deformations and pore pressure changes.

A Thunder Bay, Ontario, un tunnel de 2.16 m de diamètre est en voie d'être complété dans une argile molle à raide au moyen d'un appareil de forage de tunnel à pleine section et d'un blindage constitué de segments de béton préfabriqués et non boulonnés. Le blindage est constitué d'anneaux formés de quatre sections trapézoïdales de béton non armé et d'une longueur de 1 m par 11 cm d'épaisseur. Chaque anneau est assemblé à l'abri du bouclier et un coulis argileux est injecté à l'arrière de l'anneau alors que l'appareil de forage avance par poussée sur le blindage en place.

Par suite de la nouveauté de cette méthode de construction, un programme élaboré d'instrumentation a été entrepris pour évaluer la procédure d'excavation et la performance du tunnel complété. La première phase de l'instrumentation implique une longueur de 60 m de tunnel et comprend des observations des déformations de sol (tel que mesurées au moyen d'inclinomètres, d'extensomètres magnétiques, de points de tassement en profondeur), de la pression totale sur le blindage et de la déformation du blindage. L'information incluait également la procédure d'excavation et le poids des matériaux de déblais enlevés alors que l'appareil d'excavation passait à travers la zone instrumentée.

Ces observations sont présentées et discutées. Des problèmes d'instrumentation sont considérés particulièrement en regard du deuxième réseau d'instrumentation qui avait été conçu pour fournir des informations détaillées sur les déformations dans les trois directions et sur les variations de la pression interstitielle.

[Traduit par la revue]

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Introduction

The city of Thunder Bay, Ontario, has undertaken a $40 million program to merge and modernize sewage treatment and collection facilities. As part of this program tenders were called for a 3.3 km section of a 2.16 m diameter sanitary trunk sewer. The successful low bid was based on the use of a tunnel boring machine together with a segmented precast concrete tunnel lining. As this was the first proposed use of such a technique in North America and as the subsoil conditions were known to be difficult for tunnelling, an extensive field instrumentation program was initiated to evaluate the tunnelling procedure and the performance of the completed tunnel.

Because of the unique nature of the project it was decided to instrument a section of tunnel as close to the beginning of construction as possible in order to document the performance of the tunnelling system. This decision meant that the

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lead time for the selection and installation of instrumentation would be very short. It was anticipated that the first array of instrumentation would provide valuable experience with the various instruments, some of which were being used under these field conditions for the first time, and would lead to a more successful second array planned for a later section of the tunnel. In fact, the first array of instrumentation was quite successful and very few instruments malfunctioned. Consequently, the second array was designed primarily to provide more detailed information on the spatial distribution of deformations and pore pressure with less emphasis on overall performance of the tunnelling system. These data will be presented and discussed separately.

Tunnelling Procedure

A detailed description of the tunnelling procedure has been given by Morton *et al.* (1977). The system employs a full-face tunnel boring machine together with an unbolted precast segmented concrete tunnel lining. Each ring of the lining is 1 m long and is composed of four trapezoidal-shaped segments of unreinforced concrete 11 cm thick. The lining is assembled within the protection of the tailpiece of the tunnelling machine and is designed to serve as both the primary and secondary lining. The mined diameter of the tunnel was 2.47 m and the outside diameter of the completed lining was 2.38 m.

The tunnelling machine is advanced by thrusting on the completed lining and, as the machine advances, a clay grout is injected into the tailpiece void. A key element in the system is an erector arm which has been specially designed to lift and erect the segments of lining. One ring can be assembled in 5–10 min and progress rates of 29 m of completed tunnel per 8 h shift have been achieved in soft to firm clay.

Subsoil Conditions

General

The tunnel is constructed within a deltaic plain formed at the confluence of the Neebing, McIntyre, and Kaministkwia Rivers in the city of Thunder Bay, Ontario (Fig. 1). The stratigraphy consists of post-glacial fluvial deposits of silt and sand underlain by late glacial lacustrine deposits of layered and varved clays which extend down to black shale bedrock of the Gunflint Iron Formation of late Precambrian age.

Site Conditions

The stratigraphy at the site of the first array of instrumentation, as indicated in Fig. 2, consists of about 3 m of silty sand underlain by 3.1 m of silt and then 17.7 m of silty clay and clay which extend to bedrock.

The silty sand and silt deposits are very loose to loose with '*N*' values in the order of 2–8 blows. The deposits were saturated, with the natural groundwater table at a depth of about 1.5 m at the time of sampling.

The cohesive deposits to a depth of 7.5 m and below a depth of 12.1 m may be classified as silty clay with a liquid limit of about 40% and a plasticity index of about 20%. Between 7.5 m and 12.1 m the deposit is clay with a liquid limit of about 75–80% and a plasticity index of about 60%. The consistency of the deposit increases with depth from soft above the tunnel to firm below the elevation of the tunnel invert. The stratum is slightly overconsolidated. The amount of layering increases with depth and occasional thin silt or sand seams occur. Typical samples are illustrated in Fig. 3 where the depths, indicated in feet, are approximately equivalent to the depths to the crown and invert of the tunnel.

Instrumented Section

General

A section of tunnel 60 m in length commencing about 90 m from the first shaft was selected for the first array of instrumentation. A plan of the instrumentation installed from the surface is shown in Fig. 4 and a cross section through the centre of the array is presented in Fig. 5. This instrumentation was designed to provide data on surface deformation along the 60 m length of tunnel as well as vertical and horizontal deformation, and pore water pressure changes in the vicinity of the tunnel at one cross section. In addition to these instruments, pressure cells and stressmeters were mounted in one ring of the tunnel lining installed near the centre of the test section.

Surface Settlement

Fifty-three settlement augers were installed at a depth of about 1.5 m which was below the anticipated depth of frost penetration. The design and installation of the augers are fully described by Bozozuk (1968). The settlement was measured using standard surveying techniques and was referenced to a deep bench mark seated on bedrock. The accuracy was about ± 2 mm.

Vertical Soil Deformation

As illustrated in Fig. 5, two deep settlement augers were established at a depth of 6 m and three lines of magnetic extensometers (a total of 16 points) were installed. Four points were

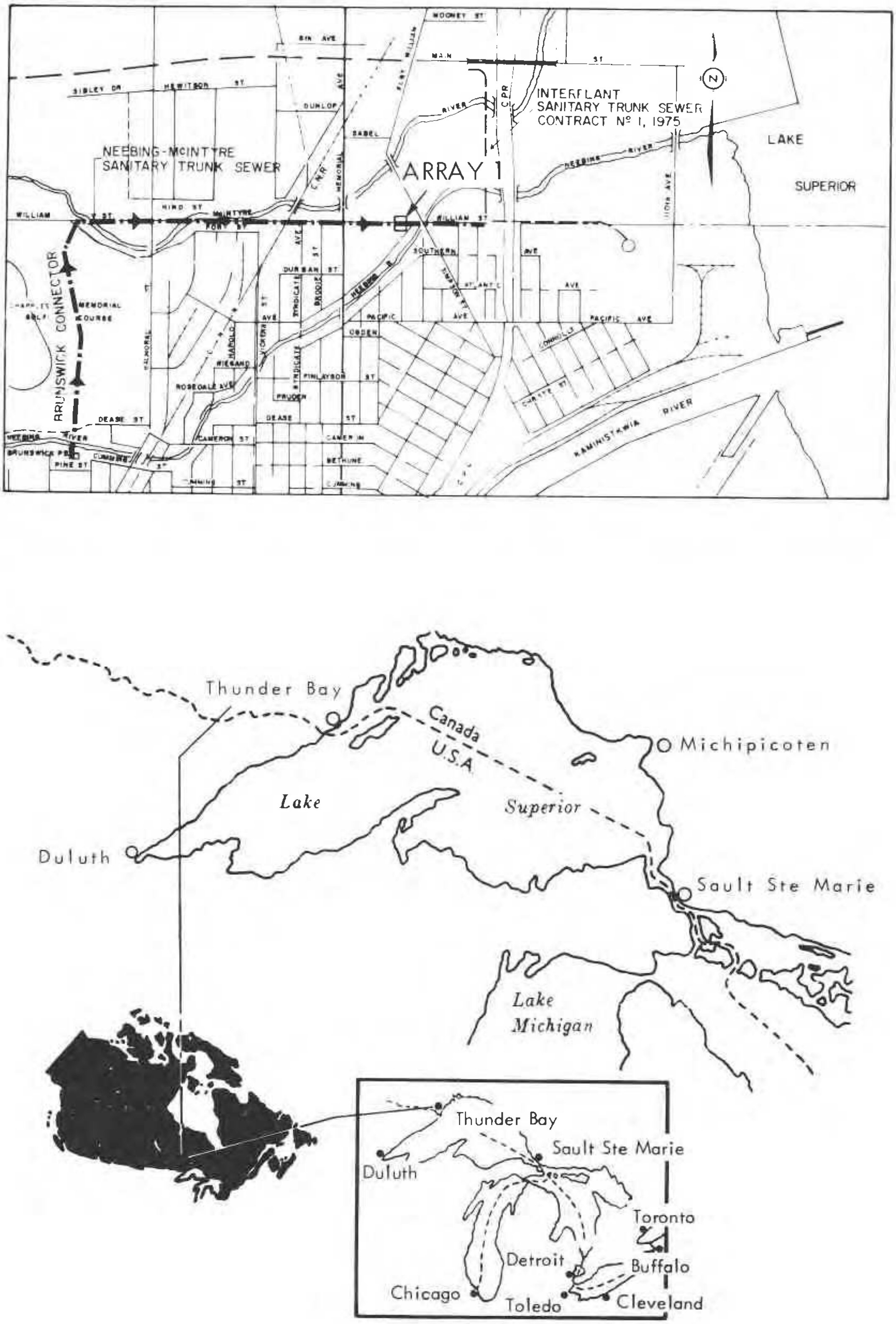

FIG. 1. Tunnel location.

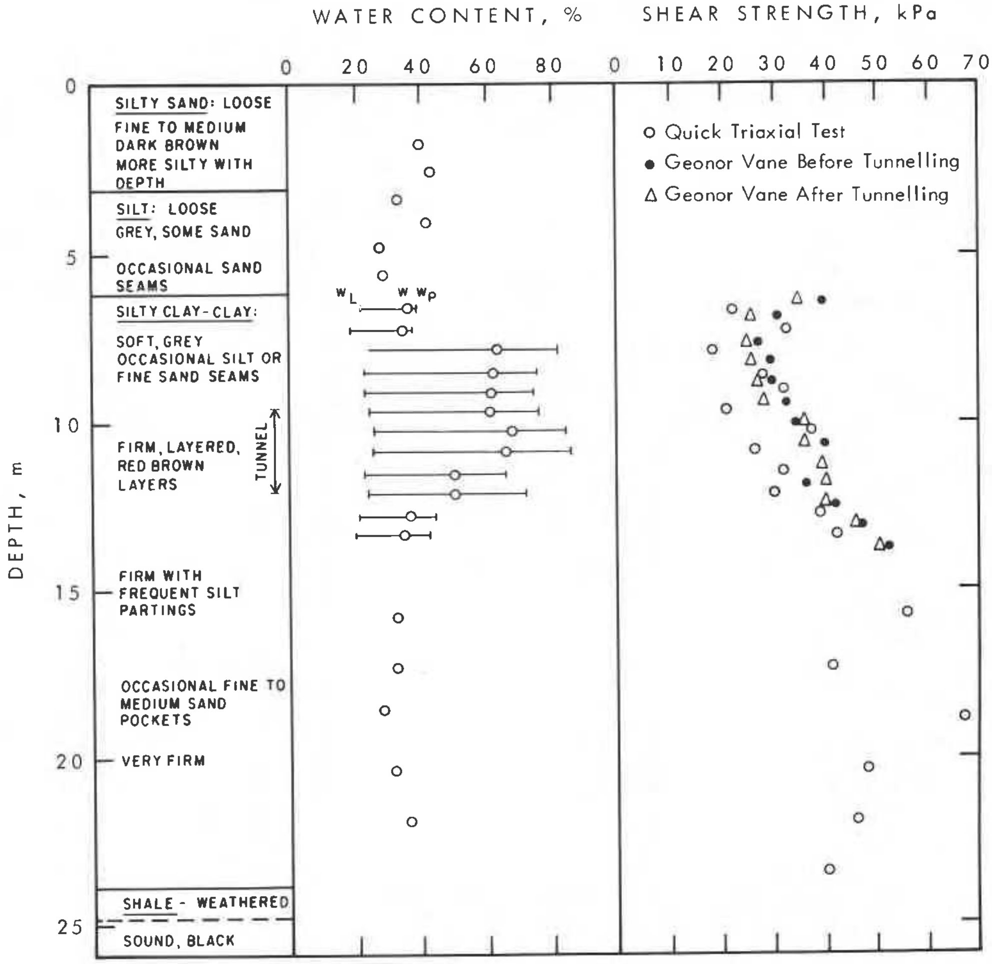

FIG. 2. Soil section and summary of test results.

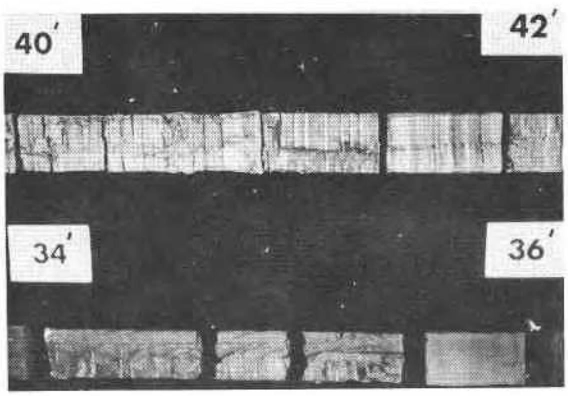

FIG. 3. Typical soil samples.

directly above the centre line of the tunnel and six points on either side of the tunnel. The vertical location of these points could be read to an accuracy of ± 1 mm and was referenced to the top of the casing which in turn was surveyed for elevation. The readout equipment and the magnetic settlement points were supplied by Solinst Canada Ltd., Burlington, Ont.

Lateral Soil Deformation

Four slope indicator casings were installed as indicated in Fig. 5. The three deep casings were seated in bedrock but, as a check on the accuracy of the results, the top of each casing was surveyed for position as was the casing over the centre of the tunnel. In addition, any settlement of the casings was observed as part of the settlement observations. A Digitilt slope indicator (Slope Indicator Company, Seattle, Washington) was used to determine the lateral displacement of the casings. The system is designed to provide an accuracy of 0.025 m/100 m of casing.

Water Pressure Changes

Seven pneumatic piezometers (Thor Instrument Company Ltd., Vancouver, B.C.) were installed

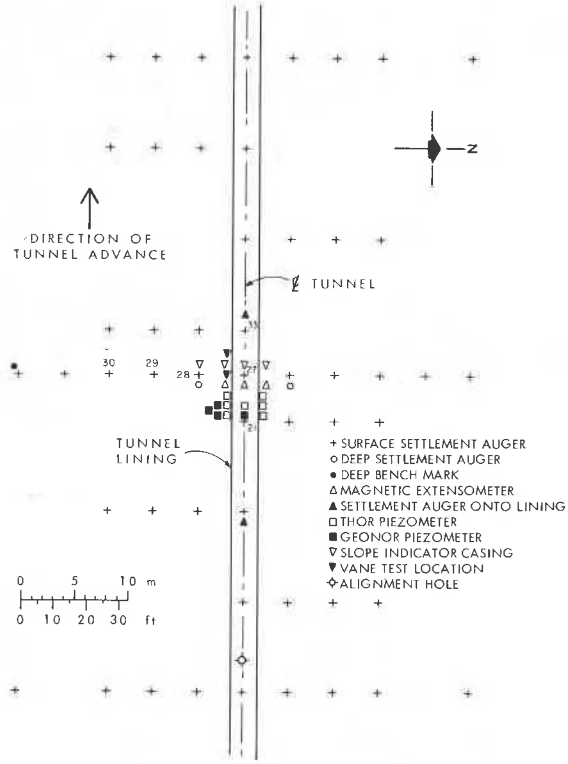

FIG. 4. Plan of instrumentation.

around the tunnel location (Fig. 5). In addition, four standard Geonor piezometers were installed: two provided a duplication of the pneumatic piezometers, one reflected changes in the sand layer, and one permitted observation of pore water pressure changes at depth.

Total Pressure Cells

Twelve total pressure cells were installed in one ring of the lining at the locations illustrated in Fig. 6*a*. The cells (Fig. 6*c*) were designed and manufactured at the National Research Council of Canada. Recesses for the cells were blocked out during the casting of the segments and the cells were installed in the casting plant (about 1440 km from the site) prior to shipment of the segments by truck to the job site. The pressure in each cell was monitored using an IRAD vibrating wire pressure transducer with a sensitivity of 0.7 kPa (IRAD, Lebanon, New Hampshire). A thermistor was included in each transducer to monitor temperature. After installation of the ring in the tunnel, lead wires were brought to the ground surface through an alignment hole and were connected to a rotary switch so that at each switch position pressure and temperature could be read simultaneously.

Stressmeters

IRAD vibrating wire stressmeters were installed in holes 38 mm in diameter, drilled through the segments after casting in the locations indicated in Fig. 6*a*. These instruments were of the solid inclusion type and monitored the deformation across one diameter of the hole in which they were installed (Fig. 6*b*). Stress changes could be calculated using an assumed deformation modulus of the tunnel lining material. Four instruments were installed to monitor the longitudinal compression in the ring caused by the thrust of the tunnelling machine as it was advanced. The other four stressmeters were installed to monitor deformations caused by hoop compression in the ring. The transducers were installed after the ring was assembled in the tunnel, then lead wires were brought to the ground surface and connected to the same switch as the pressure cells. The same readout was used for both systems.

Tunnel Deformation

Although traditional closure monitoring of the lining could not be obtained initially because of obstruction by the conveyor of the tunnelling machine, measurements were obtained on the horizontal diameter and on two subvertical chords from the invert to the circumferential joints on the roof segment. Pins were installed in the lining with a Hilti gun and a rod extensometer was used to measure changes in the distance between pairs of pins. After the trailing gear passed the instrumented ring a full set of closure readings was initiated.

Results

Soil Deformation

The vertical soil deformations in the vicinity of the centre of the instrumented section are illustrated in Fig. 7. Surface settlement was very similar from cross section to cross section. Figure 7 represents the average behavior along the 60 m length of tunnel. The centre line settlement varied from 51–64 mm except at one section (the seventh line of settlement points) where a settlement of 76 mm was recorded. This anomaly was caused by a combination of steering problems with the machine and a blockage in the grouting system. Figure 8 presents the data for the extensometer over the centre line of the tunnel and illustrates the relationship of surface settlement to position of the face of the tunnelling machine. Settlement of the points began before the face of the tunnel reached the extensometer, then continued at a relatively slow rate until the tail of the tunnelling machine passed (the tunnel boring machine had an overall length of 5.5 m). After the tailpiece cleared the instrumentation, settlement progressed rapidly as a consequence of construction activities and essentially

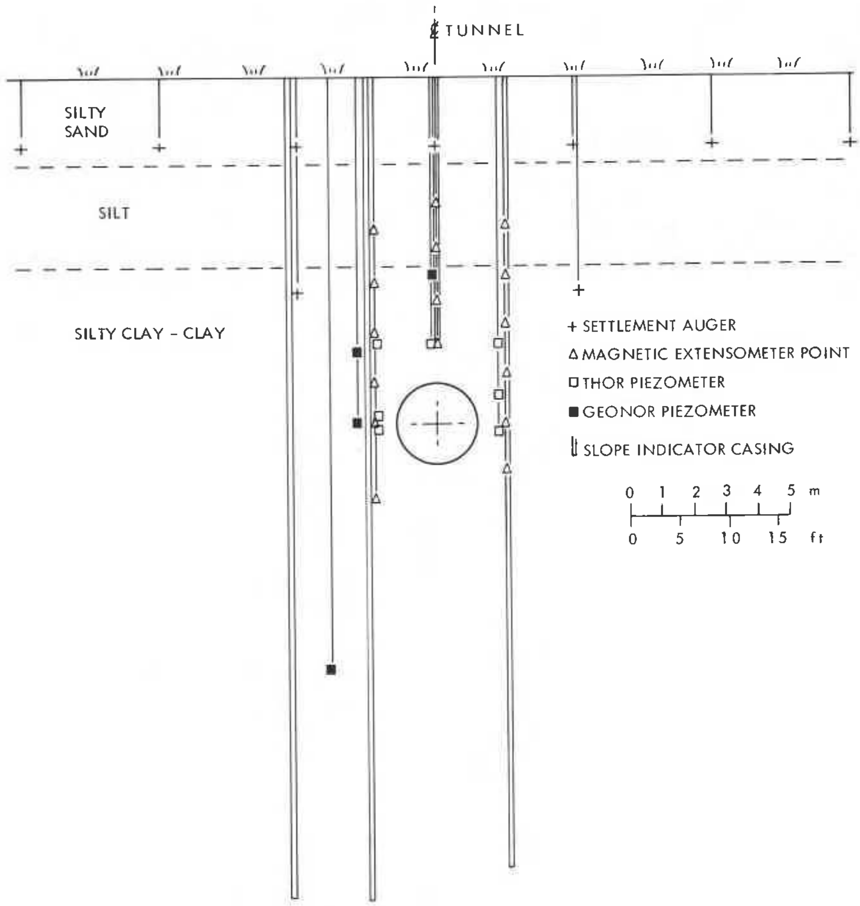

FIG. 5. Composite section of instrumentation near centre line of array.

terminated within 8 h which in this case corresponded to a tunnel advance of 12 m. A similar sequence of deformation was noted in the lateral direction (Fig. 9) with a maximum movement of about 32 mm.

Although the data shown in Figs. 7 and 8 would seem to indicate that ground surface settlement had essentially terminated, readings were continued on the instruments and by the end of the first year the surface settlement over the centre line of the tunnel had in fact almost doubled (Fig. 10). The additional settlement may be the result of reconsolidation of the remoulded soil around the tunnel and of consolidation of the clay stratum due to an increase in effective stress caused by the drawdown of the water table. As will be discussed later, the piezometric data obtained were inconclusive so that a detailed analysis will not be attempted until data from the second array are available.

A set of readings was not taken on the slope indicators at the end of the first year. However, readings obtained on August 21, 1976 (i.e., about 138 days after the tunnel face had passed the slope indicators), were sufficiently similar to the data obtained when the tunnel face was at a distance of 20 m that they were not included in Fig. 9.

The observations indicate that the pattern of soil deformation was essentially similar along the 60 m section of tunnel and that all deformation occurred within a lateral distance from the centre line equivalent to the depth of the tunnel. The initial settlement seemed to be related to the closure of the tailpiece void and, in spite of the injection of grout, the average volume of the surface settlement troughs was about equal to the volume of the void. The magnitude and extent of this settlement are quite acceptable for the tunnelling conditions. Using the terminology employed by Peck (1969), the ratio of the depth to the width of the tunnel (i.e., $z_0/2a$) equals 5 and the ratio of the trough width to tunnel width (i.e., $i/2a$) approximately

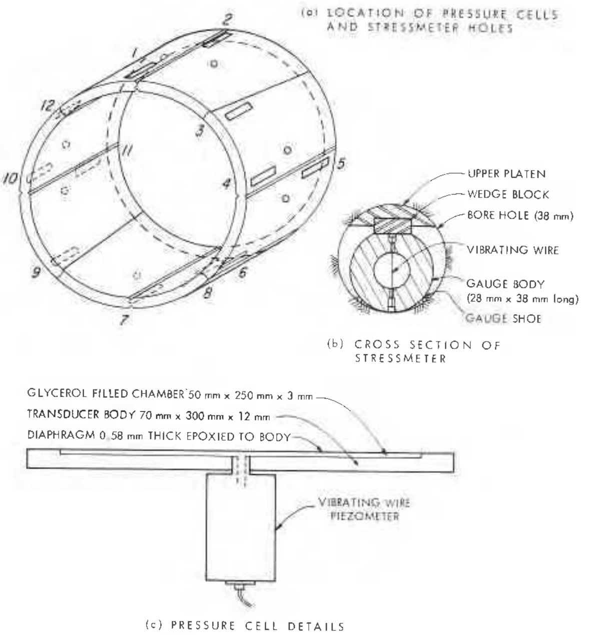

FIG. 6. (*a*) Location of pressure cells and stressmeter holes. (*b*) Cross section of stressmeter. (*c*) Pressure cell details.

equals 3.4. These ratios agree very closely with the relations presented by Peck (1969) and by Peck *et al.* (1969).

Piezometer Data

The data from the piezometers were somewhat difficult to interpret because of difficulty experienced with the readout equipment. In general, the water pressure decreased rapidly as the tunnelling machine passed the point and then increased gradually. The main conclusion derived from the data was that significant water pressure changes did occur and that piezometric pressure should be monitored in more detail in the second array of instrumentation.

Total Pressure on Lining

Data obtained from the total pressure cells (located as shown in Fig. 6*a*) are presented in Fig. 11. One cell (No. 11) was not readable after April 8, 1976, because of a short circuit in the lead wires and it was not practical at that time to repair the fault. Wires to the cells in the invert of the tunnel (Nos. 7, 8) were torn out by the conveyor trailing gear, but were reconnected later. The

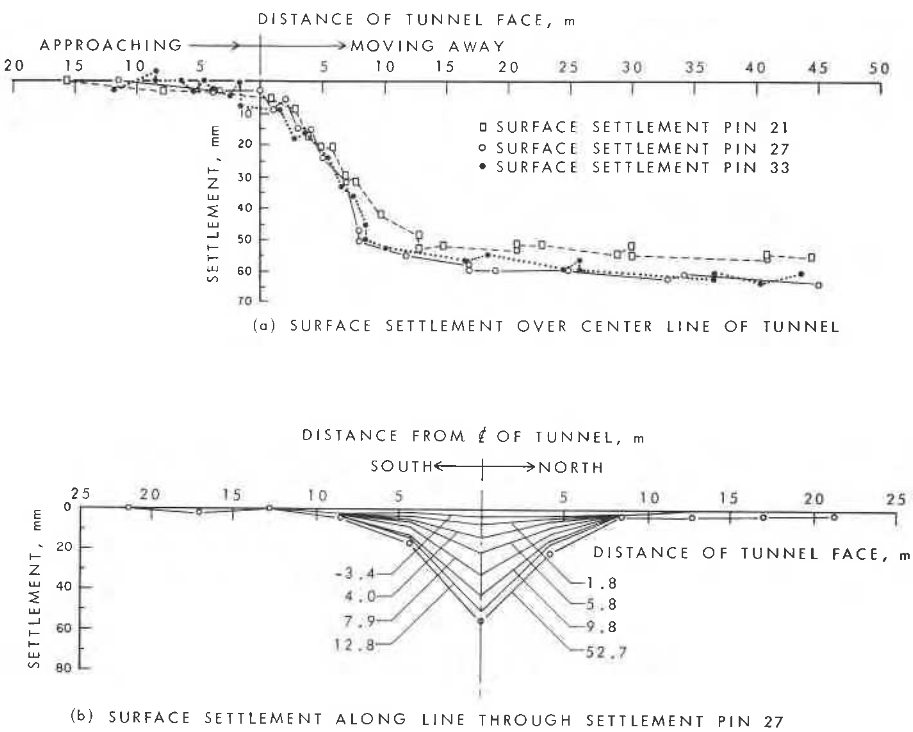

FIG. 7. (*a*) Surface settlement over centre line of tunnel. (*b*) Surface settlement along line through settlement pin 27.

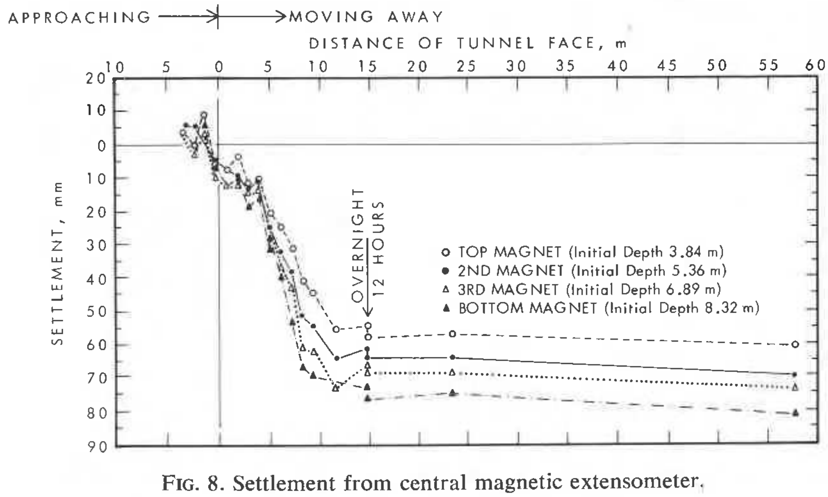

FIG. 8. Settlement from central magnetic extensometer.

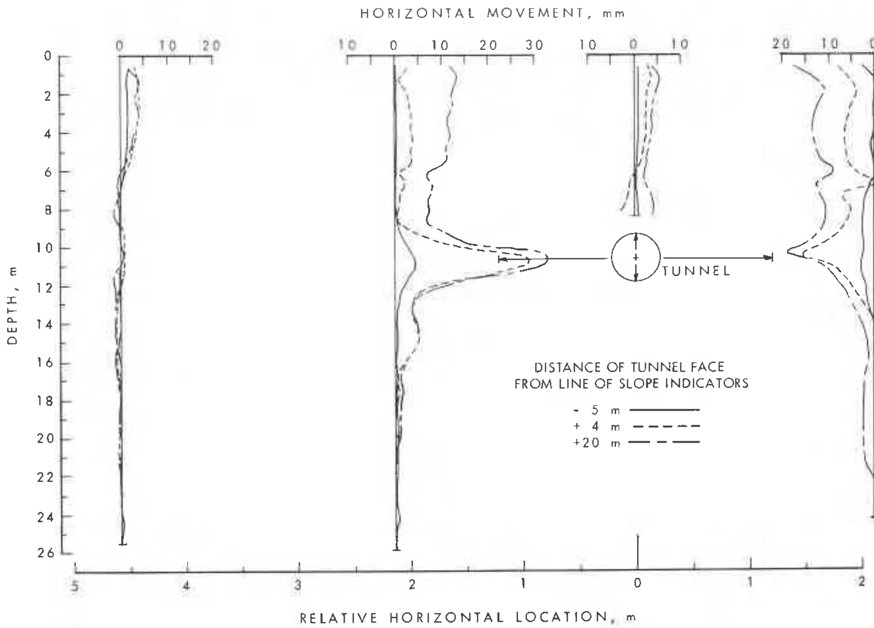

FIG. 9. Slope indicator data perpendicular to tunnel axis.

lead wire to cell No. 6 was sheared off at the end of the piezometer by the trailing gear and it was not possible to recover this cell. The performance of the cells was excellent in spite of the problem with lead wires.

It is evident from Fig. 11 that the total pressure acting on the ring was not uniform but varied by about ±25% from the average. The initial indicated pressure was approximately one half of the total vertical stress, in other words, close to the initial pore water pressure. Because this could be interpreted as an indication that all cells had ruptured and were recording water pressure only, a piezometer was installed through the wall of the tunnel at the springline between cells Nos. 10 and 11. The recorded pressure 5 weeks after installation of the ring was about 51 kPa. The seal at the circumferential joints was not entirely effective through this section of tunnel and drainage occurred toward the joints. Consequently all cells must have been intact and were sensing some pressure in excess of the pore water pressure.

The readings obtained after 1 year of observation indicate a slight overall increase in total pressure.

Stressmeters

The stressmeters proved to be easy to install

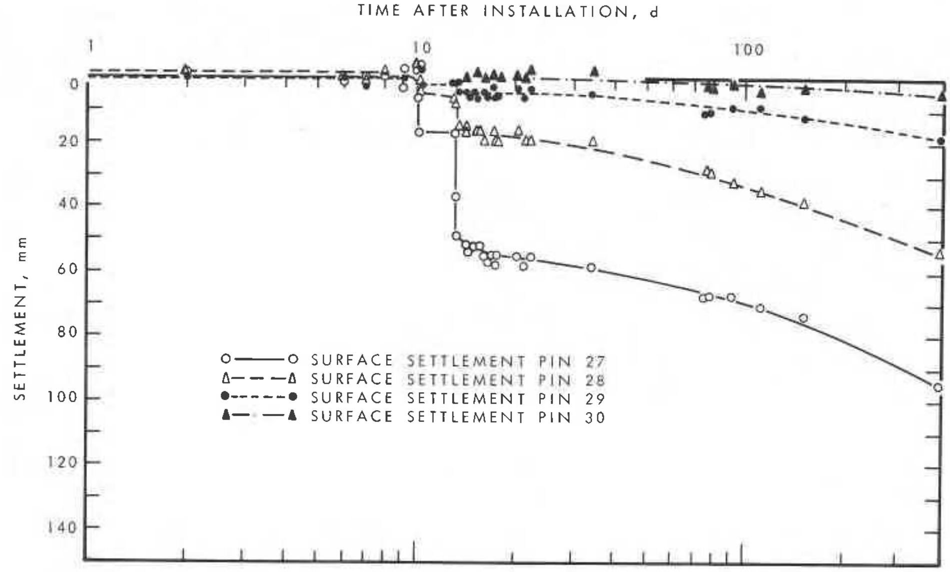

FIG. 10. Surface settlement vs. time.

(approximately 20–30 min per installation) and were convenient to monitor. These instruments were designed for use in rock. During installation it is necessary to prestress the instrument in the borehole in order to achieve a 'rigid inclusion.' Unfortunately the prestress applied for the eight stressmeters, although quite appropriate for use in rock under *in situ* stresses, was excessive for the unstressed concrete and the data obtained were essentially creep curves for point loaded concrete.

Tunnel Deformation

When the deformation pins were installed, the ring of lining was fully assembled but essentially unstressed within the protection of the tailpiece of the tunnelling machine. Soil and grout pressure became active on the ring as it passed out of the machine. Consequently, the joints between the segments tended to close. Because of the mining operation the next set of measurements was not obtained until 3 h after mining recommenced. By this time, the tunnel had been advanced 5 m and the instrumented ring was about 3.5 m clear of the tailpiece of the machine. The measurements indicated that the ring had closed laterally about 17 mm and had also closed vertically about 5 mm on the subvertical chords. Measurements taken during the next month indicated changes of less than 1 mm which were not regarded as significant. By the end of the first year, closure measurements indicated that the tunnel had tended to 'squat' somewhat with the horizontal diameter increasing and the vertical diameter decreasing by about 3 mm.

Discussion

One of the objectives of the measurement program was to document the performance of the tunnelling system. All data confirmed that the system in the test section was performing well. Early production figures were impressive in that 1 m of tunnel could be completed within 0.5 h. (Later in the project, production rates of better than 3 m/h of completed tunnel were reported.)

The performance after construction, although requiring further observations for verification, clearly indicates that post-construction settlement, probably as a consequence of consolidation of the clay stratum, will be greater than the settlement caused by the tunnelling procedure itself. It is significant that almost all settlement is localized and that the surface settlement occurs within a lateral distance of about 10 m from the centre line of the tunnel.

A second and equally important objective of the measurement program was to evaluate the instrumentation prior to installing the second array. In general, the instruments performed better than anticipated, but some improvements and changes were necessary.

The instrumentation to measure soil deformations performed well. The magnetic extensometers were easily and quickly read and checked well with single deep settlement points. The data obtained indicated symmetrical and uniform settlement but were not considered adequately detailed in the vicinity of the tunnel. Consequently, for the second array, it was decided to reduce the extent and

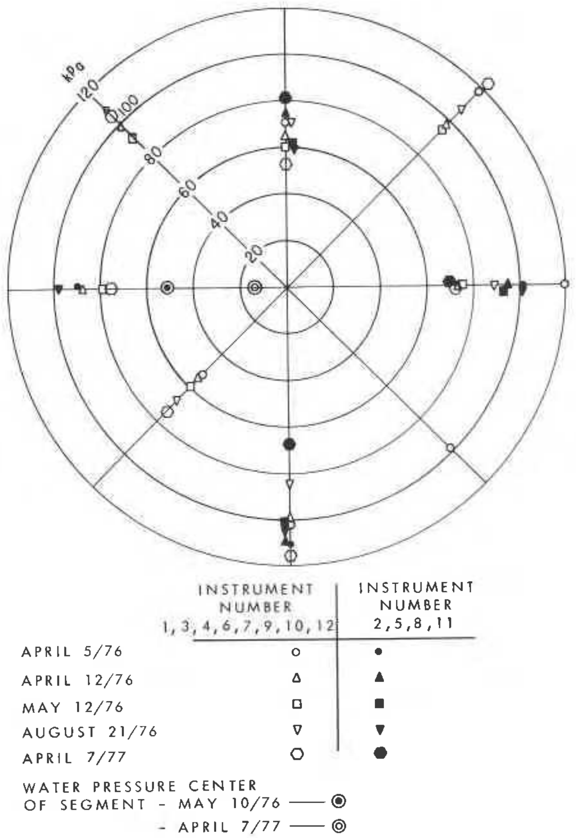

FIG. 11. Data from pressure cells located as shown in Fig. 6*a*.

number of surface settlement points and to concentrate instrumentation in the vicinity of the tunnel at one section. In addition, more information was desired for below the elevation of the invert of the tunnel. Although more lateral deformation observations were desirable, a practical limitation was the length of time required to obtain the readings (approximately 20 min to obtain a complete set of readings for each casing). One additional slope indicator casing was added and because of confidence gained in the tunnelling method and the instrument installation, the contractor permitted the casings adjacent to the tunnel to be installed closer to the tunnel alignment.

The piezometer data were considered inadequate so a full section of piezometers was planned for the second array to attempt to delineate any groundwater flow toward the tunnel. It was decided to place Thor piezometers in the vicinity of the tunnel to monitor rapid changes during tunnelling, with standpipe-type piezometers to verify these measurements and determine the long-term flow net. Four IRAD vibrating wire piezometers were to be placed in the instrumented ring of the lining.

No changes in the number or location of the pressure cells in the lining were necessary, but better protection for the lead wires was required.

It was decided to greatly reduce the installation stress of the stressmeters and to install them in the segments at the casting plant prior to shipment to the job site. This procedure reduced the installation period required in the tunnel and provided additional time for any creep due to the installation stresses to occur.

As the tunnel boring machine passed through the instrumented area the weight of spoil removed was monitored. The data correlated well with the volume of soil excavated indicating little overexcavation but were not considered sufficiently informative to justify monitoring for the second array.

Conclusions

On the basis of the data obtained from the first array of instrumentation for a 2.16 m diameter tunnel constructed in soft to firm clay, the following conclusions may be derived.

(*a*) The use of a tunnel boring machine together with a precast segmented concrete lining that acts as primary and secondary support has been shown to be a successful tunnelling procedure in soft to firm clay.

(*b*) The vertical soil deformation was very similar from cross section to cross section along a 60 m length of tunnel and was approximately symmetrical about a vertical plane through the centre line of the tunnel.

(*c*) The surface settlement during construction along the centre line of the tunnel varied from 51–64 mm except for one location where a settlement of 76 mm was recorded. This additional settlement could be attributed to the tunnelling procedure.

(*d*) An additional 50 mm of settlement along the centre line of the tunnel has occurred within the first year after construction. It is suggested that this settlement is the consequence of reconsolidation of the clay stratum.

(*e*) About 70% of the initial deformation occurred after the tailpiece of the tunnel boring machine had passed. Deformation at the end of construction may be attributed to closure of the tailpiece void.

(*f*) All surface deformation was confined within a lateral distance from the centre line of the tunnel equal to the depth of the tunnel (i.e., about 10 m).

(*g*) The total pressure measured on the tunnel lining 1 month after installation was approximately one half of the overburden pressure. The measured pressure increased slightly over the period of the first year of observation.

(h) The distribution of total pressure acting on the tunnel lining was not uniform.

Acknowledgments

The instrumentation program was funded in part by the corporation of the City of Thunder Bay through their consulting engineers for the project, R. V. Anderson Associates Ltd., in part through a contract between the Department of Supply and Services of Canada (DSS), and Morton, Dodds & Partners Limited, as a result of an unsolicited proposal, and in part by the Division of Building Research of the National Research Council of Canada.

Morton, Dodds & Partners Limited were contract managers for the project and the Division of Building Research was the scientific authority for the DSS contract. R. V. Anderson Associates Ltd. supplied personnel for some of the field work and provided the necessary liaison for the project.

This project could not have been undertaken without the permission and cooperation of the Corporation of the City of Thunder Bay, the general consultant, R. V. Anderson Associates Ltd., and the general contractor, Mole Construction Co. of Cleveland, Ohio and Thunder Bay. In addition, cooperation of the tunnel boring machine manufacturer, Lovat Tunnel Equipment Inc. of Toronto, Ontario, and the segment fabricators, Pre-Con Ltd. of Brampton, Ontario, is gratefully acknowledged.

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