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DEFORMATIONS AND PORE PRESSURES IN THE VICINITY OF A PRECAST, SEGMENTED, CONCRETE-LINED TUNNEL IN CLAY

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

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Deformations and pore pressures in the vicinity of a precast, segmented, concrete-lined tunnel in clay¹

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During construction of a 2.16 m diameter tunnel in a soft to firm clay deposit in Thunder Bay, Ontario, an initial array of instrumentation to provide data indicating overall performance and a second array to provide detailed spatial deformation and pore-water pressure distribution were installed. The tunnel lining consisted of unbolted rings composed of four trapezoidal, unreinforced concrete sections assembled within the tailpiece of the tunnel boring machine.

This paper presents and discusses detailed observations of soil deformation, pore-water pressure changes, and total pressure on the lining obtained from the second array of instrumentation. The soil stratigraphy was similar to that at the first array and the data obtained are compared with the earlier data. Monitoring of the instruments is continuing and data from the first year of observations are presented.

Lors de la construction d'un tunnel de 2.16 m de diamètre dans un dépôt d'argile molle à ferme à Thunder Bay, Ontario, une première série d'instruments a été installée pour obtenir des indications sur le comportement général de l'ouvrage et une deuxième série a été installée pour mesurer en détail la distribution des pressions interstitielles et des déformations. Le revêtement du tunnel était constitué d'anneaux non boulonnés, formés par quatre sections trapézoïdales de béton armé, assemblées à l'arrière d'une machine-tunnel.

Cet article présente les observations détaillées de déformations du sol, de variations de pressions interstitielles et de contraintes totales sur le revêtement, obtenues à l'aide de la deuxième série d'instruments. La stratigraphie du sol étant semblable à celle prévalant à proximité de la première série; les données obtenues sont comparées. On continue à faire le relevé des instruments, et les données relatives à la première année d'observation sont présentées.

[Traduit par la revue]

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Introduction

In March 1976 construction was commenced on a 3.3 km section of a 2.16 m diameter sanitary trunk sewer in the city of Thunder Bay, Ontario. The tunnel was constructed through soft clay soil using a tunnel boring machine together with a segmented precast concrete tunnel lining. This was the first use of such a construction technique in North America, and because the subsoil conditions were known to be difficult for tunnelling an extensive field instrumentation program was undertaken.

Owing to the unique nature of the project, an array of instrumentation was installed close to the start of construction to provide both an initial overall indication of the tunnel performance and on-site experience with the instruments and soil conditions in prepara-

tion for a second similar installation planned for a later section of the tunnel. In fact, the initial array of instrumentation was so satisfactory that the second array was redesigned to provide more detailed information on the spatial distribution of deformations and pore pressure, with less emphasis on overall behaviour.

The field data obtained from the initial set of instrumentation have been presented by Belshaw and Palmer (1978). In the present paper the results obtained from the second array of instrumentation are presented and discussed.

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

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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 mined diameter of the tunnel is 2.47 m and the outside diameter of the completed lining 2.38 m. The lining is assembled within the protection of the tunnelling machine tailpiece and is designed to serve as both the primary and secondary lining.

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

Subsoil Conditions

General

The tunnel is constructed within a deltaic plain formed at the confluence of the Neebing, McIntyre, and Kaministiquia Rivers in the City of Thunder Bay, Ontario (Fig. 1). Although arrays 1 and 2 are about 1.25 km apart, the stratigraphy is very similar. Postglacial fluvial deposits of sand and silty sand are underlain by late glacial lacustrine deposits of layered and varved clays that extend to black shale bedrock of the Gunflint Iron Formation of the late Precambrian age.

Site Conditions

Array 2 is located in an area of mixed bush with moderately difficult access. The stratigraphy (Fig. 2) consists of about 0.9 m of peat underlain by 7 m of sand and silty sand, then 16.6 m of silty clay to clay extending to bedrock. The depth to the centre line of the tunnel is about 10.5 m.

The sand deposit is generally very loose to loose in relative density, with standard penetration test N values ranging from 1 to 10 blows. The N values of 20–45 blows (shown in Fig. 2) are not representative of the overall deposit. The grey sand grades to silty sand with depth and occasional clay seams are present. The groundwater table was at a depth of about 1.5 m when the instruments were installed.

The upper 5 m of the cohesive deposit consists of grey silty clay. The remainder is made up of alternating grey and reddish brown silty clay to clay layers with occasional layers of silt or fine sand. The shear strength, as indicated by unconfined compression tests, varies from 15 to 32 kPa and, as indicated by Geonor vane tests, varies from 30 to 65 kPa with sensitivities from 5 to 11. The data shown in Fig. 2 were obtained in the course of installing the instru-

ments. Penetration resistance, unconfined compression test samples, and the Geonor test data were obtained from the boreholes for slope indicators A, B, and C, respectively.

Instrumentation

General

Data previously obtained at array 1 indicated that the surface settlement trough was symmetrical with respect to a vertical plane through the axis of the tunnel and very similar from section to section along the tunnel centre line (Belshaw and Palmer 1978). Almost all vertical deformation instruments and piezometers of array 2 were installed as much as possible within one cross section, the piezometers only on one side of the tunnel centre line (Fig. 3). To verify that the instrumented area represented average tunnel behaviour in the vicinity of array 2 surface settlement over the tunnel centre line was monitored at about 6 m intervals for a distance of 24 m on either side of the principal cross section. In addition, total pressure cells, stressmeters, and piezometers were mounted in one ring of the tunnel lining approximately beneath the location of settlement auger No. 4 (Fig. 3). The tunnel lining instrumentation will be discussed separately.

Soil Deformation

Thirty settlement augers and five lines of magnetic extensometers were installed to monitor vertical deformation. The surface settlement augers were installed at a depth of 1.5 m, which was below the anticipated depth of frost penetration. The deep settlement augers and magnetic extensometer points were located at various depths, as shown in a composite section (Fig. 4). The reference for all settlement measurements was a deep benchmark seated on bedrock (Fig. 3). Standard surveying techniques with an accuracy of ± 2 mm were employed.

To monitor lateral deformation, five slope indicator casings were installed, as indicated in Fig. 4. The four deep casings were seated in bedrock. The position of the top of each casing was periodically surveyed. In addition, any settlement of the casings was observed as part of the settlement auger observations.

Pore-water Pressure Changes

Thirty piezometers were installed, located in plan as shown in Fig. 3 and in composite section as indicated in Fig. 5. In the immediate vicinity of the tunnel, seven pneumatic piezometers (T1–T7) were used to monitor rapid changes in pore pressure. Three Geonor piezometers (G1, G3, G5) were in-

FIG. 1. Tunnel location.

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

stalled to provide a duplication of three of the pneumatic piezometers and two others were located about 5 m from the tunnel for the purpose of *in situ* hydraulic fracturing tests. The remaining piezometers were of the standpipe type and were installed to monitor only the long-term pore-water pressures. A standpipe piezometer was installed as a reference about 50 m north of the array. Vibrating wire type piezometers (I21-I24, Fig. 5) were installed in the centre of each segment in the instrumented ring. These provided a rapid response during tunnel construction and could be monitored remotely without difficulty.

Tunnel Instrumentation

The ring of tunnel lining (i.e., four segments) was also instrumented to monitor total pressure acting on the lining, longitudinal stress caused by thrust of the tunnelling machine as it was advanced, and hoop compression in the segments. Although this instrumentation is not all of primary importance to the topic of this paper it is included to complete the record of the field observations. Similar pressure cells and stressmeters were used in array 1 and are described in more detail by Belshaw and Palmer (1978).

Except for the four piezometers, the other 20 instruments were installed in the segments at the

FIG. 3. Plan of instrumentation.

casting plant (about 1440 km from the site). Subsequently, the segments were shipped to the site by truck as part of a regular shipment. Ideally, this ring of lining should have been installed directly beneath the surface instrumentation, but it was considered important to have a completely normal, continuous tunnelling operation through the instrumented area. The ring was installed, therefore, as the last unit on the last night shift before the instrumented area was reached. As a consequence of cooperation on the part of the contractor and his men, plus some luck, the ring was installed within 7 m of the fully instrumented area. The instrumentation was designed so

that all wiring, checking, and protection could be completed in the available 7 h before the commencement of the day shift. The readout terminal was installed at the top of the nearest shaft, which was located about 135 m east of the instrumented ring.

Field Observations

All instruments were installed more than 3 weeks prior to construction through the array. Monitoring of the instrumentation intensified as the tunnelling machine approached the site. The face of the machine was about 15 m east of the centre of the array at 19:30 h on 19 August 1976, at which time an ob-

FIG. 4. Composite section of soil deformation instrumentation.

server was stationed in the tunnelling machine to record progress and mining procedure.

All subsequent surface observations were coordinated with the tunnelling operation. At 01:00 h on Friday, 20 August, when the night shift ended, the face of the machine was 4.1 m east of the cross section of settlement augers (settlement auger 5, Fig. 3). Tunnelling recommenced at 07:55 h the same day and by 15:40 h the face of the machine was 15 m west of settlement auger 5. There was some delay in the mining at this time and no night observations were taken. No mining was carried out during the weekend; but by 10:30 h on Monday morning the face of the tunnel had been advanced to 30 m west of auger 5. The rate of change of all observations was so small that the next full set of readings was delayed until 20 days after tunnelling (i.e., after 20 August). A further full set of readings was taken 23 July 1977, or almost 1 year after construction. All instruments have been protected and monitoring will be continued for a 5 year period if possible.

Results and Discussion

Soil Deformation

The vertical soil deformation as indicated by measurements on the surface augers is shown in

Fig. 6. A settlement of 54 mm over the centre line of the tunnel was the maximum movement recorded throughout the instrumented section after 20 days. The remaining centre line augers 1-9 (Fig. 3) indicated deformations of 37-47 mm, with an average of 44 mm. Centre line settlements were minimum at augers 1 and 9 (i.e., on entering and leaving the array) and increased to a maximum at auger 5. This may be an indication of the disturbance caused by the instrument installation or may be coincidental (similar behaviour was not evident at array 1). Nevertheless, the detailed section shows the maximum surface settlement recorded. In comparison, centre line surface settlements of 47-76 mm were recorded at array 1 (Belshaw and Palmer 1978).

The volume of settlement represented by the area of the trough shown in Fig. 6, times a unit length, is the maximum measured within the instrumented zone. It is about 7.5% of the corresponding total excavated volume of the tunnel and approximately equal to the volume of the tailpiece void. The minimum size trough may be estimated on the basis of a centre line deformation of 37 mm and would be about 5% of the total excavated volume. The observed trough is similar in size to troughs for tunnels in stiff clays reported by Peck (1969). Peck *et al.* (1969) show an average relation for trough width and subsidence, expressed as

$$[1] \quad i/a = (z_0/2a)^{0.8}$$

where i = trough width (Schmidt 1969); a = radius of tunnel; and z_0 = depth of axis of tunnel. For the settlement trough shown (Fig. 6), $i/a = 2.5$ measured versus 3.5 calculated, indicating a trough width ratio appreciably less than that experienced on the average.

The sequence and distribution of vertical deformation is illustrated in Fig. 7, where the observed movement of the magnetic extensometers over the centre line of the tunnel is combined with the observations at auger 5 and plotted relative to the position of the face of the tunnelling machine. As the tunnelling machine approached any point a slight heave was observed. At the ground surface, settlement commenced before the machine face reached the plane of the point. At greater depths, such as 6 m, settlement did not commence until after the face of the machine had reached the point. This indicates that a bowl or trough of settlement precedes the tunnelling operation. After the tailpiece of the machine had passed any point, settlement progressed rapidly and was practically complete within 6 h (or within about 15 m of advance). Approximately 60-70% of the settlement was observed after passage of the machine tailpiece.

The vertical deformation after almost 1 year had

FIG. 5. Composite section showing piezometers and piezometric heads after 1 year.

FIG. 6. Surface settlement along line through settlement auger 5.

increased less than 15%, whereas at array 1 the settlement after 1 year increased almost 100% (Belshaw and Palmer 1978). There were two major differences between arrays 1 and 2. The first was the thickness of the clay cover, which was 1.2 m at array 2 versus about 3.5 m at array 1. There was, therefore, appreciably more clay to be remoulded and reconsolidated at array 1. The second difference was that

the mining of the tunnel commenced 90 m from array 1. Consequently the contractor had very little tunnelling experience with the new machine and lining technique when the machine entered the instrumented area. Alignment difficulty and grouting problems were noted by the engineer observing the tunnelling operation as the machine passed through array 1. These factors probably combined to remould the clay significantly in the vicinity of the tunnel at array 1, leading to increased long-term settlement.

The magnitude and distribution of lateral deformations are shown in Fig. 8, where observations from the slope indicator installations are combined with vertical deformation data and plotted as apparent displacement vectors. Very little deformation occurred beneath the depth of the tunnel invert and all major deformation took place very close to the tunnel alignment.

Pore-water Pressure Changes

The change in pore-water pressure as tunnelling proceeded is illustrated in Fig. 9. As the machine approached the plane of the piezometer, the pore-

FIG. 7. Vertical soil displacement over centre line of tunnel.

FIG. 8. Displacement vectors in vertical plane perpendicular to tunnel axis (after 20 days).

water pressure within 1 m of the tunnel increased by about 40 kPa or about 10%. As the machine passed the piezometer, the pressure rapidly decreased by as much as 40% and reached a minimum when the face

of the machine was about 9–10 m beyond the location. Subsequently, the pore-water pressures gradually increased, but indicated a small net decrease after 10 months.

It was anticipated that the tunnel might act as a drain and that a long-term flow net could be determined from the measurements of the piezometer array. Net changes were very small. The measured hydrostatic head at each piezometer after approximately 1 year is shown in Fig. 5. The head shown for piezometer T1 is based on the change in pressure rather than on the measured head because the zero reading of the piezometer shifted during installation and recalibration was not possible. Piezometer S9 was destroyed by vandals (physically pulled out), and several of the other piezometers were damaged, but readings still appeared to be reliable. A crack that developed in the tunnel lining at piezometer I23 was never completely sealed, so that the water pressure measured there should be regarded as low. Piezometers I21 and I22 are more reliable. Piezometer I24 was loosened and the wires were cut off during tunnel cleanup; it was removed and returned to the laboratory for a calibration check and observation of long-term zero drift. The data in Fig. 5 indicate a minor drawdown in the tunnel vicinity, although substantially less than would be predicted for a clay having an isotropic permeability.

It is considered that permeability in the tunnel vicinity may have been affected by the construction procedure. As tunnelling proceeded, a clay grout was injected into the tailpiece void. At the same time, the soil immediately adjacent to the tunnel must have

FIG. 9. Change in pore pressure during tunnelling operation.

FIG. 10. Hypothetical flow net in clay.

been disturbed and partially remoulded. If it is arbitrarily assumed that the net effect of the two operations is to create a zone 0.3 m thick with a permeability reduced by a factor of 16, then the flow net in the clay would be distorted as shown in Fig. 10. The pore-water pressure drops determined for each piezometer on the basis of Fig. 10 are close to but still

greater than those actually measured in the field. This suggests that either a thicker or a more impermeable layer must exist adjacent to the tunnel.

Tunnel Lining Instrumentation

The location of all transducers in the instrumented ring of lining is shown in Fig. 11. In general, the pressure cells performed very well, except for zero stability. A prototype pressure cell and two of the piezometers used to construct the pressure cells are being monitored in the laboratory. Observations have shown that the zero reading of the piezometers gradually changes with time. The shift is of the order of 0.02 kPa apparent pressure increase per day. Piezometer I24 when recovered after 1 year had a zero shift of about 0.01 kPa per day. The pressure readings shown in Fig. 11 have not been corrected for zero shift and should be regarded as the maximum probable pressure at each cell. Although the lead wires to all instruments in the tunnel invert were cut during tunnel cleanup, cells 9, 10, and 12 were rewired successfully.

After a year's observation the total pressure on the lining was approaching uniformity, except on the diagonal through cells 6 and 12. These will be observed in the long term to see whether pressure redistribution does occur. Although the pore-water pressure tended to increase with time, the total pressure on the lining was still very low after 1 year. In general, the total pressure distribution on the lining at array 2 was more uniform than that recorded at array 1 (Belshaw and Palmer 1978), but the average pressure measured was about the same at each array.

The stressmeter installations were unsuccessful,

FIG. 11. Instrumentation in lining and pressure data.

because it was impossible to achieve positive seating of the instruments in the concrete lining. Although the magnitude of stress could not be determined the stressmeters did indicate that during thrusting on the instrumented ring the stress in the lining was approximately uniform, of the order of 1.4 MPa. This is equivalent to a total thrust of about 1.2 MN applied to the lining to advance the tunnelling machine.

Conclusions

Detailed observations of soil deformation, pore-

water pressure changes, and total pressure on the tunnel lining indicate the following:

(1) Use of a tunnel boring machine together with a precast segmented concrete lining, which acts as both primary and secondary support, has been confirmed as a successful tunnelling procedure in soft to firm clay.

(2) The magnitude and extent of the surface settlement trough in the instrumented area were less than those normally observed for tunnels in similar soils. The volume of the surface trough was about 5-7½% of the excavated volume of the tunnel.

(3) Approximately 85% of the measured settlement occurred within 6 h after the face of the tunnelling machine passed the plane of the measuring point. During this time the tunnel was advanced about 15 m; 60–70% of the deformation occurred after the tailpiece of the tunnelling machine passed the point.

(4) Vertical deformation over the centre line of the tunnel increased by less than 15% during the first year following construction. It is suggested that this small amount of additional settlement, in contrast with the nearly 100% increase observed for an earlier array of instrumentation, is a result of a combination of reduced thickness of clay above the tunnel and of more experience with the construction technique.

(5) Pore-water pressures near the tunnel increased as the tunnelling machine approached, decreased as it passed, and then gradually increased again. Readings taken after 10 months indicated a small net decrease in pressure compared with the original values.

(6) The observed piezometric heads in the vicinity of the tunnel after construction indicated that drainage was toward the tunnel. The extent of the draw-down indicated that there was a zone of soil around the tunnel in which permeability must have been much less than that in the undisturbed soil.

(7) Total pressures (as measured) acting on the tunnel lining were not uniform, but tended toward uniformity in the first year of observations. The average pressure was about the same as that reported for a previous array of instrumentation, i.e., about one half of the overburden pressure.

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