

Performance of a deep excavation in downtown Toronto

Performance d'une excavation profonde au centre-ville de Toronto

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ABSTRACT: This paper presents field measurements of soldier pile walls installed in the clayey soils and shaly rock in downtown Toronto. The method of deducing wall bending moments from the inclinometer measurements was evaluated and discussed. Back-analysis using a finite element program has been carried out to evaluate the shoring wall performance as well as the creep behaviour of the shaly rock. Recommendation for the design of soldier pile walls in the similar soils and bedrock conditions were provided.

RÉSUMÉ : Cet article présente des mesures de terrain pour des murs de pieux soldats installés dans les sols argileux et le roc shaleux du centre-ville de Toronto. La méthode donnant les moments de flexion du mur à partir des mesures inclinométriques a été évaluée et discutée. Une analyse a été effectuée avec un programme d'éléments finis pour évaluer la performance du mur étayé ainsi que le comportement en fluage du shale. Des recommandations, pour la conception des murs de pieux soldats dans des conditions semblables de sols et de roc, ont été fournies.

KEYWORDS: deep excavation, field measurement, inclinometer, bending moment, finite element, time-dependent deformation

1 INTRODUCTION

Underground structures such as basements and subway have to go deeper today than in the past due to limited space in densely populated urban environments. As deep excavations induce large stress and strain, underground structures and the adjacent structures/utilities will confront risks of being damaged. As the soil/rock stress-strain behaviour is non-linear and affected by many factors, it is difficult to predict the ground movement induced by excavation. In practices, field measurements are widely used to monitor soil/rock behaviour and to control ground movement.

This paper presents a case study of a deep excavation in downtown Toronto. Soldier piles with tiebacks were used to support the excavation. Two inclinometers and one hundred and twenty seven reflective targets were installed to monitor the movements of the shoring walls during and after excavation. The inclinometer measurements have been used to deduce the wall bending moments. A finite element program has been carried out to evaluate the performance of the shoring walls. It is found that the total stress analysis leads a good prediction of wall deflections during the excavation, whereas the effective stress analysis is required to model the behaviour of shoring walls after excavation. The back-analysis also shows the evidence of the creep movement of the shaly rock.

2 GROUND CONDITION AND TEMPORARY SUPPORT SYSTEM

The site is located at 352 Front Street West in Toronto, Ontario. Field investigation with drilled boreholes revealed that the site stratigraphy was made up of about 1 m thick, compact sand to gravel fill with asphalt surface overlying 3 to 4 m thick, firm to hard clayey silt fill over 2 to 5 m thick, stiff to very stiff clayey silt till. Both clayey fill and till are low plasticity soils. Georgian Bay formation of shale and limestone/siltstone was encountered at 9 to 9.5 m below existing ground surface. The groundwater table was about 5 m below grade.

The Georgian Bay formation is generally massive shaly rock with widely spaced jointing and sub-horizontal bedding planes. The influence of sedimentary shaly bedrock formations on the engineering performance of underground structures in Southern Ontario was summarized by Lo (1989). The shaly bedrock formations are subjected to high in-situ horizontal stresses with typical coefficient of lateral earth pressure K_0 of 4 or greater. Upon relief of the high residual horizontal stresses, time-dependent, creep-like deformations take place. These time-dependent deformations that are highly stress dependent, persist well beyond the initial elastic deformations and generally exceed the magnitude of the elastic movements.

Soldier piles of steel H-beam W610x82 at 3.05 m spacing with wood lagging were employed to support an approximately 14 m deep excavation in which 9.3 m excavation was inside overburden soils and 4.7 m excavation inside the bedrock. The soldier piles were installed typically 16 m below the existing ground surface in 910 mm diameter drilling holes. The drilling holes were backfilled by 0.4 MPa concrete with the exception at the pile toe, where 20 MPa concrete was used to support the pile toe. Two layers of tiebacks were installed at approximately 3.3 and 8.3 m below the existing ground surface, respectively to support the soldier pile walls during excavation. The tiebacks were installed within 150 mm dia. cased boreholes and bonded in bedrock. Each tieback was made up of 6 to 7 numbers of 15 mm strand tendons. The upper and lower tiebacks were installed at 45° and 25° to the horizontal direction, respectively. The bond length of the upper tiebacks was typically 5 m and the free length 9.4 m. The bond length of the lower tiebacks was typically 3 m and the free length 3.9 m. The tiebacks were generally post-grouted the day after they were installed. The typical design loads for the upper and lower tiebacks were 1000 and 800 kN respectively. Figure 1 shows outlook of soldier piles with wood lagging supported by tiebacks.

Two performance tests for the tiebacks were conducted up to 138% and 200% of the design load, respectively. The test loads were maintained for 0.5 to 1 hour and the tests met the PTI criteria (PTI, 1996). Proof tests were carried out for all

tiebacks. The test load was 133% of the design load and maintained for 10 minutes. All tiebacks except three tiebacks met the PTI criteria. The three tiebacks could not reach the test load due to the broken wires. A lower design load was used for the three tiebacks. Detailed discussions on the tiebacks are presented by Cao and Peaker (2011).



Figure 1. Outlook of soldier piles with wood lagging supported by tiebacks

Two inclinometers were installed inside the soldier pile walls during the pile installation. The inclinometers were monitored during and after the excavation. Figure 2 shows the monitoring results of one inclinometer including the reading taken after upper and lower tieback installations, 1 day after the excavation to bottom, and 11 months after the excavation. The lateral deflections measured by reflective targets installed at the top of soldier piles are also shown in Figure 2. The measurements of reflective targets are consistent with the inclinometer measurements.

3 BENDING MOMENT FROM WALL INCLINOMETER MEASUREMENTS

The inclinometer measurements have been used to estimate wall bending moments by some researchers (Poh et al. 1999). The in-wall inclinometers provide a direct measurement of the rotation. These measurements can be subsequently converted into wall deflections along the wall. The wall curvatures κ can be derived from the wall deflection data. The second differential equations of the wall deflection will give the κ along the wall. The bending moment M can be computed from κ using the following equation (West, 1993)

$$M = EI\kappa = KI(d^2y/dx^2) \quad (1)$$

where E is the elastic modulus of the wall, I is the inertia moment of the wall, y is the lateral deflection of the wall and x is the distance along the wall.

Using Microsoft Excel spreadsheet, the inclinometer measurements were fitted with a sixth-degree polynomial and double differentiation of this polynomial gave κ . The coefficient of determination value obtained during the curve fitting ranged from 0.98 to 0.99, indicating minimal error during the process of curve fitting. The Young's moduli of 0.4 MPa concrete and H-beam W610x82 were taken as 2.8 GPa and 200 GPa, respectively. The sum of concrete EI and H-beam EI was used in the calculation of the bending moment. Figure 3 shows the bending moments deduced from the wall inclinometer measurements. Higher bending moments were observed at the locations of tiebacks. However, significant high values of bending moments were obtained near the ground surface, which is against the typical distribution of bending moment along a cantilever beam. This could be an error induced in the double differentiation of the wall deflection. Further study using a

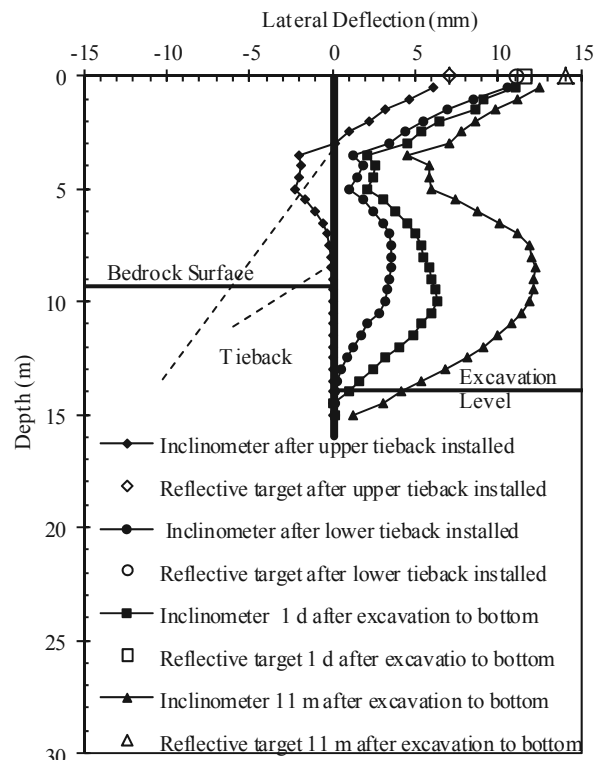


Figure 2. Lateral deflection of soldier pile wall

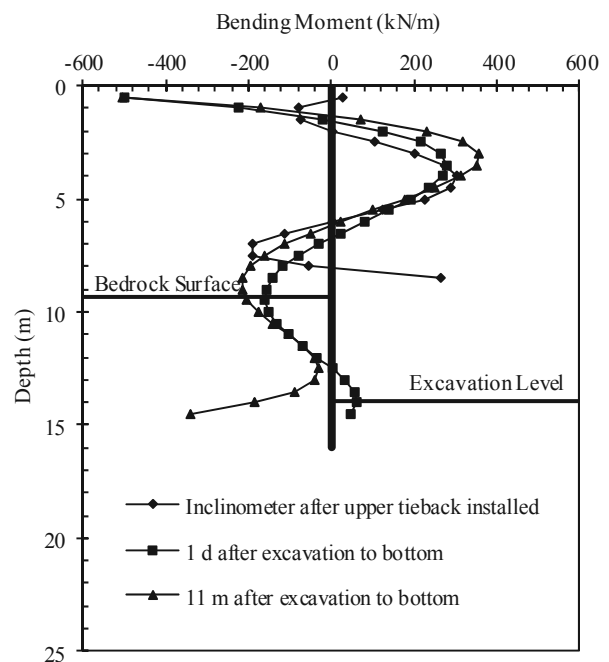


Figure 3. Wall bending moments deduced from inclinometer measurements

higher degree polynomial and a defined boundary condition is required.

4 FINITE ELEMENT BACK-ANALYSIS

The finite element program Phase 2 (version 8.0) was used in the back-analysis. The program can be used to simulate excavation in soil and rock under plane strain condition. Six-node triangle elements were used to model the soil and bedrock media. The soldier pile wall and tiebacks were modelled by

structural beam elements. The analysis modelled a half width of the excavation where the right-hand boundary of the mesh represented the line of symmetry at the centre line of excavation. The finite mesh was 140 m long and 84 m deep. The half width of the excavation was 20m. The bottom boundary was strained from both vertical and horizontal movements. The left-hand and right-hand boundaries were free to move in the vertical direction.

The soil and bedrock profiles used in the analysis were based on borehole logs. The groundwater level was taken at 5 m below existing ground surface at the initial stage. During the excavation, the groundwater level was assumed to be drawn down to the excavation level at the excavated side.

For the surface sandy fill, the Young's modulus E of 25 MPa and the friction angle ϕ of 30° were assumed. For the clayey soils, the undrained shear strength s_u was estimated from $6N$, where N is the blow counts of the standard penetration testing. The residual s_u was taken as 50% of the initial s_u . The undrained E of clayey soil was estimated from $1500s_u$ for the native low plasticity clayey silt till and $500s_u$ for the clayey silt fill, respectively. The soil Poisson's ratio was taken as 0.3. The unit weight γ was obtained from available laboratory testing data. Mohr Coulomb failure criterion was used for soils. The soil properties used in the analysis are shown in Table 1.

Table 1. Soil parameters used in the finite element analysis

Depth (m)	Type	γ (kN/m ³)	E (MPa)	s_u (kPa)	ϕ	K_o
0 – 1	Sandy fill	20	25	0	30	0.5
1 – 4.5	Clayey fill	20	30	60	-	0.75
4.5 – 9.3	Clayey till	21	225	150	-	0.75

Note: K_o is the coefficient of lateral earth pressure (total stress)

For the jointed shaly bedrock, the generalized Hoek-Brown constitutive model was used. The following parameters were used to generate the generalized Hoek-Brown rock-mass strength criterion: (1) The geological strength index was taken as 60 for sound bedrock, respecting blocky to very blocky, good to fair joint surface, and 30 for weathered bedrock, respecting blocky/disturbed/seamy joint surface; (2) The intact rock constant was taken as 8 for sound bedrock (highest value for shale) and 4 for weather bedrock (lowest value for shale); (3) The disturbance factor was taken as 0 for excellent quality controlled excavation; and (4) The modulus ratio was taken as 250 for bedrock (highest value for shale) and 150 for weather bedrock (lowest value for shale). The intact compressive strength was obtained from available results of rock point load testing and unconfined compressive testing. The rock Poisson's ratio was taken as 0.15. Based on the above assumptions, the obtained strength parameters for the generalized Hoek-Brown's model are summarized in Table 2.

Table 2. Rock parameters used in the finite element analysis

Depth (m)	Type	γ (kN/m ³)	E (MPa)	m_b	s	a	K_o
9.3 – 11.3	Weathered bedrock	25	244	0.3	0.004	0.52	2
>11.3	Sound bedrock	26	3072	1.3	0.004	0.51	4

Note: m_b , s and a are parameters used in generalized Hoek-Brown's model

The soldier pile wall was modelled as reinforced concrete with W610x82 at spacing of 3.05 m. The equivalent thickness of 0.4 MPa concrete was taken as 0.2 m and the Young's modulus was 2.8 GPa. The concrete compressive and tensile strengths were taken as 400 kPa and 40 kPa, respectively. The compressive and tensile strengths of W610x82 were taken as 345 MPa. The Poisson's ratio for steel and concrete was taken as 0.2. The equivalent bolt diameters for the upper and lower tiebacks were taken as 32 mm and 24.5 mm, respectively. The Young's modulus of tiebacks was taken as 200 GPa. The bond shear stiffness was taken as 6000 kN/m/m based on the tieback proof test results. The bond lengths of the upper and lower tiebacks were taken as 5 m and 3 m, respectively. The spacing of tiebacks was taken as 3.05m.

The measured and computed wall deflections after the installation of upper tiebacks and the excavation just to the bottom are shown in Figure 4. The computed wall deflections are in a good agreement with the inclinometer measurements, indicating that the in-put parameters used in the analysis are reasonable.

The computed bending moments for the excavation just to bottom are compared with those deduced from the inclinometer measurements as shown in Figure 5. The bending moments deduced from the inclinometer measurements are comparable with the computed except near the ground surface where significant high values deduced from the inclinometer measurements. Ignoring the high bending moments near the ground surface, the bending moments deduced from the inclinometer measurements can be used for the checking of the capacity of the soldier piles.

The inclinometer measurements show that up to 7 mm lateral movement was developed after the excavation to bottom as shown in Figure 2. This could be due to three possible reasons: (1) the consolidation of clayey soil; (2) de-stressing of tiebacks; and (3) time-dependent deformation of the shaly rock upon relief of the initial high horizontal stresses. The first two possible reasons have been studied in the finite element analysis using the effective parameters and reduced modulus for the

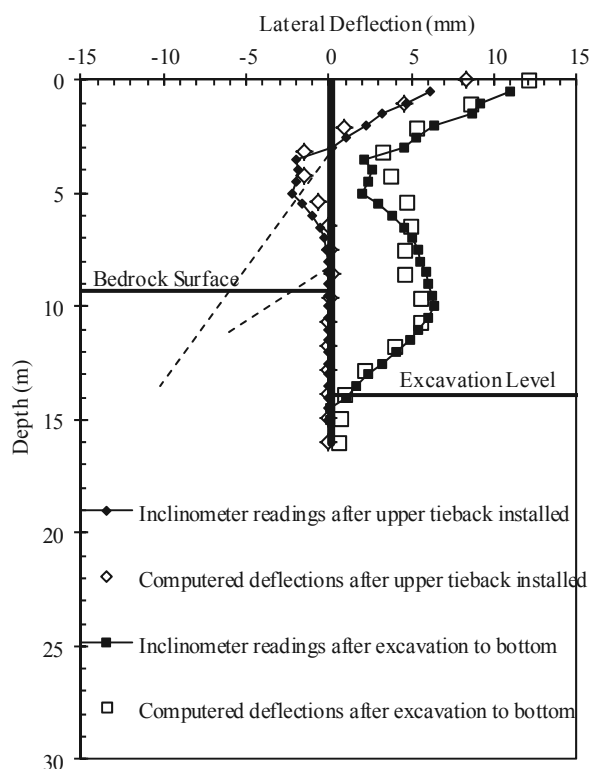


Figure 4. Measured and computed wall deflections

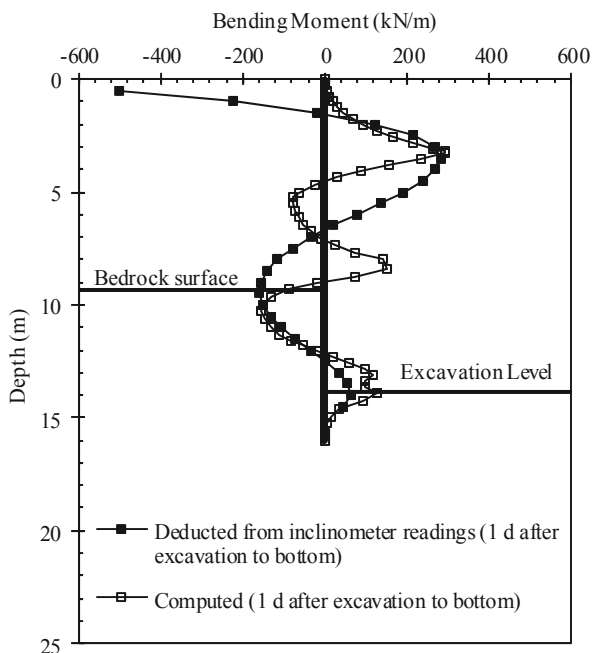


Figure 5. Deducted and computed wall bending moments

clayey soils, and reduced prestressing for tiebacks. Figure 6 show the comparison of the computed and measured lateral deflections. In this analysis, the E was taken as 80% of the initial E ; the effective ϕ and cohesion for clayey fill were taken as 30° and 3 kPa, respectively; the effective ϕ and cohesion for clayey till were taken as 32° and 7.5 kPa, respectively; and the lower tiebacks were assumed to be de-stressed to 50% of the initial prestressing. The computed lateral deflections within the upper portion of the overburden are in a good agreement with the inclinometer reading. However, there is a difference of up to 5 mm between the computed and measured deflections in the lower portion of the overburden and the bedrock, which should belong to the time-dependent deformation of the shaly rock as the de-stressing of tiebacks or the consolidation of clayey soils could not lead such deformation.

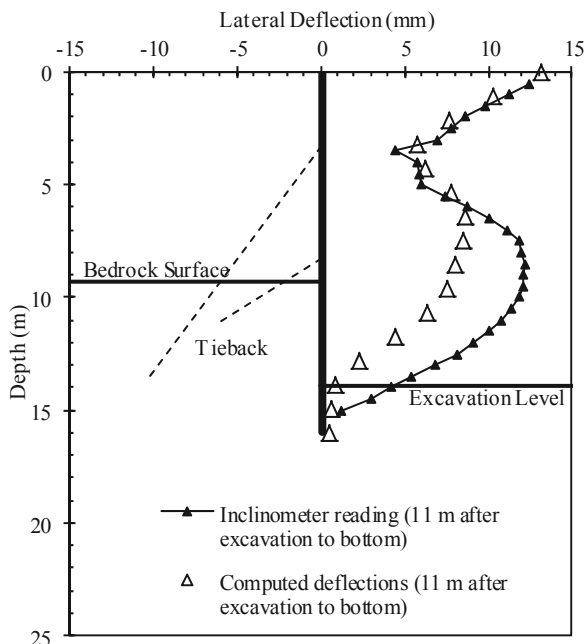


Figure 6. Measured and computed wall deflections (11 months after excavation)

5 CONCLUSIONS

In-wall inclinometer has an important role in the monitoring of shoring walls during and after excavation. Wall bending moments can be estimated from the inclinometer measurements except near the ground surface where the bending moments are overestimated probably due to the error in double differentiation of the wall deflection.

Both total and effective stress analyses should be used for the design of shoring walls installed in the clayey soils. As supported by the finite element back-analysis, the clayey soils behave as undrained during excavation and as drained after excavation. The finite element analysis confirmed that the time-dependent deformations of shale rock during 11 months after excavation could be up to 5 mm for 4.5 m excavation in the shaly rock. The time-dependent deformation should be considered in the shoring wall design.

6 REFERENCES

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