The effects of deep excavations in soils and rock on adjacent structures
The 2011 R.M. Hardy Address

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ABSTRACT
Urban developments require construction close to adjacent structures or directly above underground infrastructure. The excavation for the new structure induces stresses and displacements onto the existing structures and the effects have to be evaluated. The methodology to address this problem is illustrated by two well-documented case histories. The first case describes the deflections and stresses imposed onto the lining of subway tunnels in soil in Toronto. The second case deals with excavation in weak rock for the Bow Tower in Calgary and the effects on surrounding structures. Lessons learned regarding the evaluation of controlling soil and rock parameters, interpretation of monitoring results for ongoing construction, and impact on design approach are presented.

RÉSUMÉ
Les développements urbains requièrent qu'une construction se trouve à proximité des structures adjacentes ou juste a au-dessus des infrastructures souterraine. L'excavation de cette nouvelle structure provoque des déplacements et des contraintes sure les structures avoisinantes, et les effets doivent être évalués. La méthodologie permettant d'aborder ce problématique est illustrée par deux études de cas bien documentés. Le premier cas décrit les déformations et les contraintes imposées sur la doublure de tunnels de métro dans le sol, à Toronto. La deuxième cas traite d'une excavation localisée dans un roc friable pour la construction de la Tour de Bow, à Calgary ainsi que de ses effets sur les structures avoisinantes. Seront présentes leçons apprises au sujet de l'évaluation du contrôle des paramètres de sol et de roche, de l'interprétation des résultats de la surveillance de constructions en cours, ainsi que de l'impact portant sur la démarche du design.

1 INTRODUCTION

It is a great privilege and honour for me to present the Hardy Keynote Address to the 14th Pan-American Conference on Soil Mechanics and Geotechnical Engineering (PCSMGE), the 64th Canadian Geotechnical Conference (CGC) and the 5th Pan American Conference on Teaching and Learning of Geotechnical Engineering (PCTLGE).

The subject that I have chosen to discuss in my presentation is the effect of deep excavation in soils and rock on adjacent structures. The objective is to describe the application of a methodology, which has been utilized in deep excavations in such a way so that the impact of existing structures are minimized to satisfy pre-determined criteria, but at the same time feasible projects may be constructed without undue penalization. To apply this methodology, it is necessary to identify the predominant soil or rock parameters and properties that govern the performance of the excavation, and the uncertainties defined with a plan of accounting for the uncertainties developed. The engineering consideration may be described in the following sections.

2 DEFINITION OF THE PROBLEM

Rapid growth in urban development necessitates the construction of buildings close to or directly above existing structures. The excavation for the new structure induces stresses and displacements on the existing infrastructure. It is therefore required to assess the impact of construction on the existing structures.

In general, the following items should be examined:
(i) the extent of vertical, horizontal and rotational displacements that may affect the structural or operational integrity of adjacent structures;
(ii) the change in factors of safety in the existing structure and structural components; and
(iii) in deteriorating infrastructures (e.g. transit tunnels, sewage and water mains) the decrease in factor of safety from the original design value and the increase in sensitivity to deformation should be taken into account in the evaluation (i.e. baseline condition).

3 METHODOLOGY

The methodology in dealing with this design problem involves the following steps:
(i) determination of soil stratigraphy and soil parameters relevant to the stress paths in the unloading case;
(ii) analyses of stresses and displacements in both the transverse and longitudinal directions of the tunnels;
(iii) establishment of allowable limits of induced stresses and deflections;
(iv) development of monitoring programs for construction control;
(v) evaluation of results of field measurements to compare with design assumptions and predictions during the process of excavation;
(vi) modify sequence, lifts, and locations of excavations if necessary; and

(vii) investigation of the existence of disturbed zones around the lining, created during the construction of the tunnels, so that localized problems of overstress in a few rings may be assessed and prevented.

In the Toronto area, deep excavations had been carried out both in soils and rock. Some case histories of these excavations are shown below.

I. Downtown Area – Excavations in weak swelling rock (Georgian Bay Shale) subjected to high horizontal stress (generally 2-6 MPa but can be higher)

- Scotia Plaza
  1984 24 m depth
- Shangri-La Tower
  2008-2010 26 m depth
- Rogers Centre (Skydome) and water supply tunnels underneath in the general area
  1985-1987
- Enwave tunnels for deep lake water cooling
  2001-2009

II. Excavation in Till

- York Mills Centre (Yonge St.-York Mills St.)
  1987-1990 9 m depth
- Condominium Bloor St. - Sherbourne St
  1989 8 m depth
- Sheppard Tail Track Phase III
  1995 20 m depth
- Empress Plaza (Yonge St. -Empress St.)
  1995-1997 19 m depth
- Dundas Square (Yonge St. -Dundas St.)
  1999 12 m depth

III. Windsor-Detroit Tunnel – Soft clay (Significant as an early detailed case history of excavation over an aged tunnel constructed in 1929. The earth pressure acting on the lining and the stresses in the concrete lining were measured in situ).

In this presentation, we shall discuss a case history in soil, the York Mills Centre in Toronto and a case history in weak rock, the Bow Tower in Calgary, Alberta.

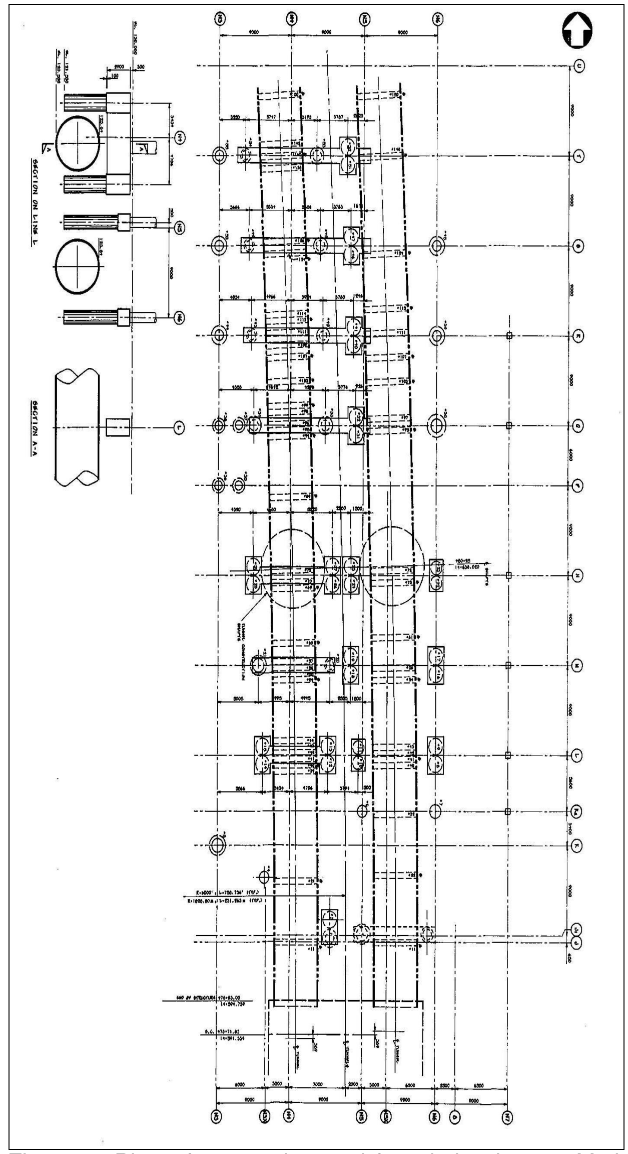

Figure 1. Plan of excavation and foundation layout- York MillsCentre

Figure 2. TTC Subway tunnel

4 YORK MILLS CENTRE, TORONTO

4.1 Site Conditions and Construction History

The proposed construction was Phase III of the York Mills Centre, located at the northeast corner of the intersection of Yonge Street and York Mills Road in Toronto, Ontario. Conventional soils investigation was started in March 1987 and excavation and installation of foundation elements completed by September 1990.

Figure 1 shows the foundation layout over the TTC subway tunnels. To eliminate the net structural loading onto the tunnels the loads were transferred to the subsoil below the invert by caissons of 1.1 to 2.4 m diameters, straddling the tunnels. The design and construction issues were therefore to minimize the effect of excavation for the substructure so as to satisfy the structural and operational requirements of the subway tunnels.

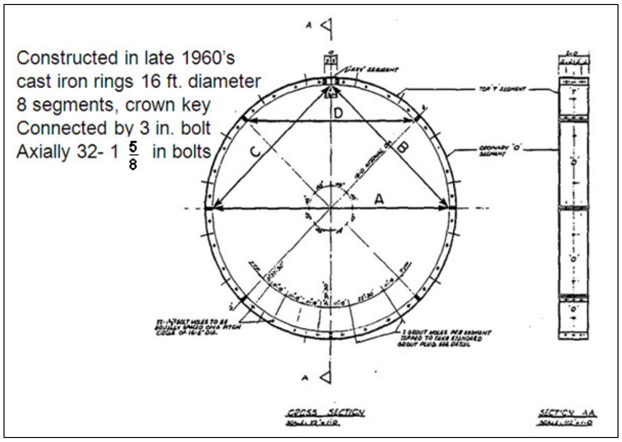

Figure 3. Detail of cast-iron segmental lining of TTC subway

The excavation for the substructure was approximately 90 m by 40 m, with depth of excavation generally 7 m to 9 m. The proximity of the general excavation to the tunnel crown was about 2 m. However, local excavation for the load transfer girders was within 0.5 m from the extrados of the lining. A photo of the tunnel is shown in Figure 2. Details of the cast-iron segmental lining are shown in Figure 3. This section of the Yonge Line was constructed in the late 1960s, with cast-iron segmental lining.

Preliminary analysis using soil parameters from conventional sampling and testing showed that the deformations and stresses imposed onto the tunnels would not satisfy structural and operational requirements of the subway structures. The objective of the detailed investigation and design analysis was to obtain more representative soil parameters and detailed analyses to ascertain the feasibility of the project. The approach was to apply the methodology as described in the previous section.

4.2 Soil Conditions and Evaluation of Relevant Soil Parameters

The subsoil condition is a uniform deposit of grey dense silt, approximately 47 m thick, overlying bedrock (at E1.91.5 m). The water content is 11.4% and the unit weight is 22.6 kN/m^3.

Block samples were recovered from a test trench at an excavation east of the subway station at E1.126 m. Specimens were trimmed from the block samples and anisotropically consolidated drained tests were performed in the compression and extension modes. The relevance of triaxial tests to tunneling problems has been discussed (Ng and Lo 1985). In this class of problem, the extension modulus of deformation at small strains plays an important role in the design analysis.

Typical results of a triaxial extension test are shown in Figure 4. It may be seen that the initial portion of the stress-strain curve is linear up to at least 0.4% axial strain. The strain at failure is approximately 1%. The sample dilates during shear but the volumetric strain is small. The Poisson's ratio is 0.42 in this test.

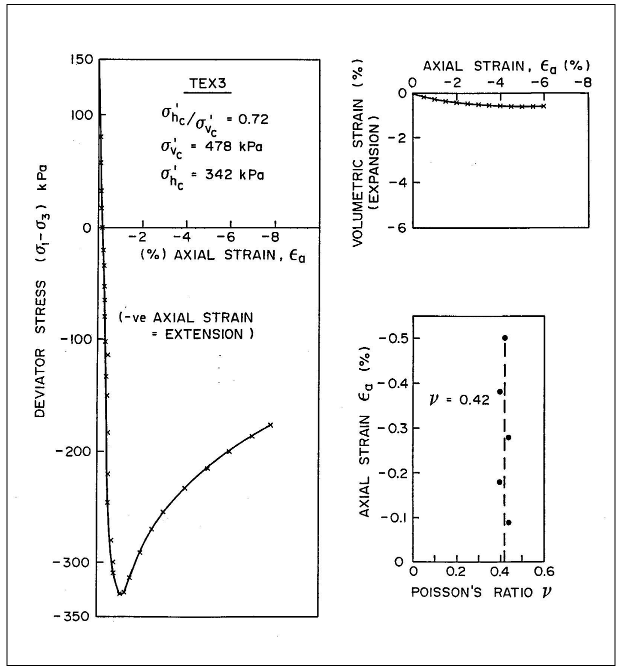

Figure 4. Typical results of anisotropically consolidated drained triaxial extension test

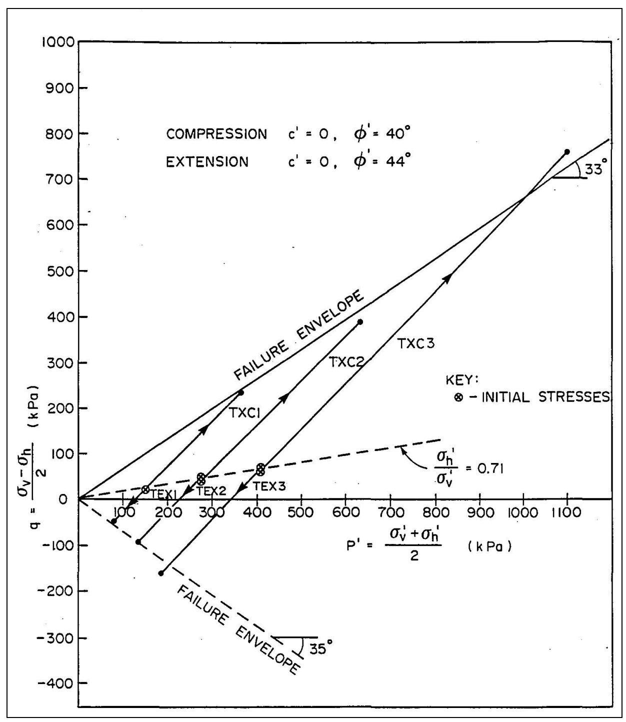

Figure 5. Failure envelope of triaxial compression and extension tests

The strength envelope for the compression and extension tests are shown in Figure 5. The samples were anisotropically-consolidated to an effective stress ratio of

$\frac{\sigma'_h}{\sigma'_v}$ = 0.7. For design purpose, the effective stress parameters were taken to be c' = 0, ϕ' = 40°.

For the evaluation of soil deformation imposed onto the tunnel, the soil deformation modulus is a most important parameter to be reliably assessed. Initially, analysis using the modulus profile from conventional sampling and testing showed that deformations would be excessive and more precise evaluation of soil parameters would be required.

In the second stage, block samples were recovered at the location of the south-east portion of the excavation already completed. Typical results of drained extension tests, shown in Figure 4 were employed to obtain the distribution of modulus with depth. Finally, advantage was taken of the on-going excavation west of the Don Mills subway station (see Figure 6) to install instruments for heave monitoring. At locations P1 and P2, magnetic extensometers were installed and the heaves at six different depths were measured as local excavations were carried out in this area. No de-watering is allowed within the zone of influence of the heave monitoring area, to ensure that the heave measurements were not affected. The results of these measurements are shown in Figure 7.

4.3 Analysis of Proposed Excavation

The proposed excavation in plan, as of December 1987, is shown in Figure 6.

During the course of design, however, the geometry and depths of excavation were changed several times. The final longitudinal section adopted, together with two other proposals analyzed, is shown in Figure 8.

Complications in analyses and interpretation also arise because excavation was continuing at the south portion (south of column line M in Figure 1) with permission of the engineers, so as not to delay construction. In principle, therefore, for any given profile of final excavation, the predicted displacements (and other relevant quantities) should be considered in the following manner:

(i) displacements that had already occurred prior to December 1, 1987. These can be computed; however, because monitoring had not yet started, they cannot be verified;

(ii) displacements induced by the remaining main portion of the excavation. These can be predicted and directly compared with field measurements; and

(iii) total displacements resulting from the entire excavation.

However, it may be seen from Figure 6 that the critical areas for both transverse and longitudinal analyses are north of column line M, so that items (2) and (3) are almost identical in these areas.

4.4 Design Soil and Structural Parameters

The soil parameters, based on laboratory test results are shown in Table 1. The two soil deformation modulus profiles in Figure 7 were employed, but design decisions were based on the profile evaluated from heave measurement.

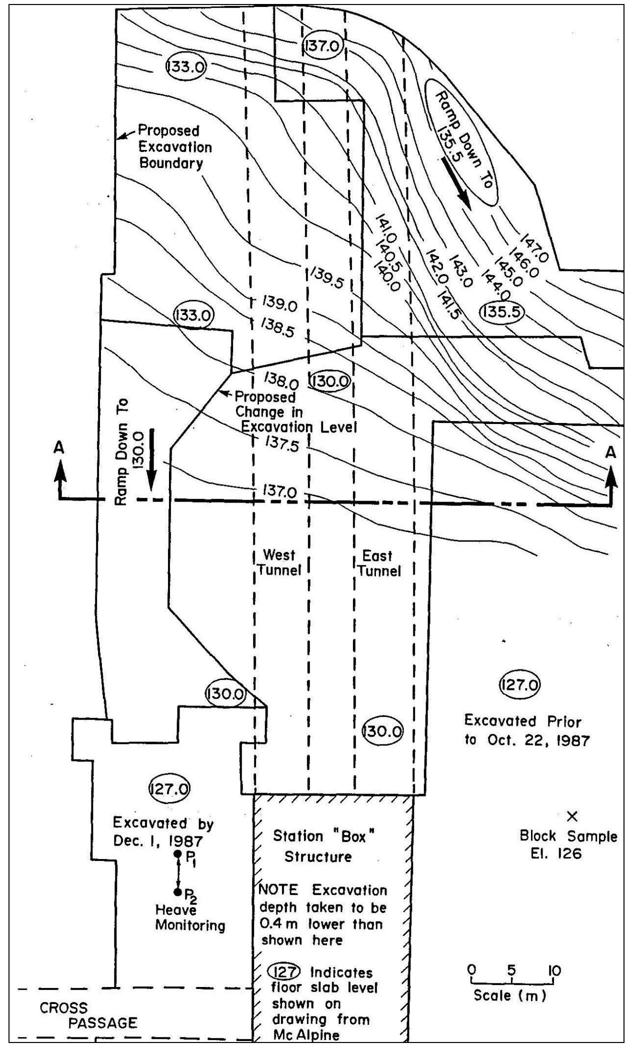

Figure 6. Plan showing final excavation levels (proposed as of December 1987)

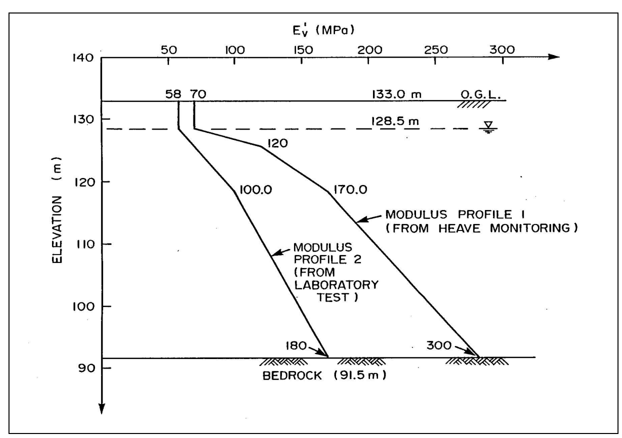

Figure 7. Distribution of unloading modulus with depth

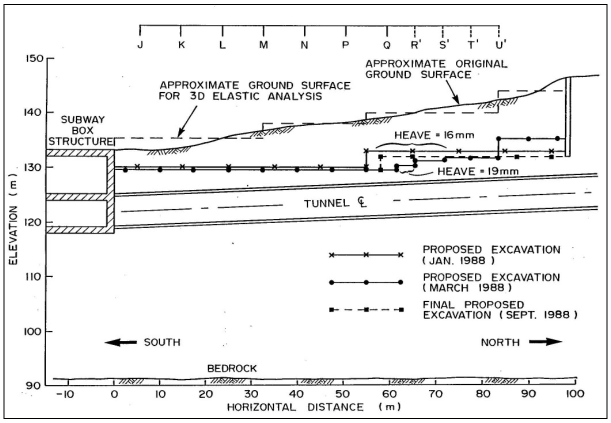

Figure 8. Longitudinal section of proposed excavations along tunnel

The cast iron segmental tunnel lining structural properties were given by the structural engineer and are shown in Table 1.

Table 1. Properties of till, tunnel lining and bolt.

Till Properties	
Saturated Unit Weight γ_{sat} (kN/m^3)	22.5
Poisson's ratio, ν	0.4
Coefficient of Lateral Pressure at Rest, K_o	0.67
Effective Friction Angle, ϕ' (degree)	40°
Cohesion Intercept, c' (kN/m^2)	0.2
Dilation Angle, ψ	0.0
Soil Deformation Modulus, E'_v (kN/m^2)	Fig 7
Tunnel lining Properties: Transverse Section	
Area, A (m^2/m)	0.3658×10^{-1}
Moment of Inertia, I (m^4/m)	0.7143×10^{-4}
Young`s Modulus, E (kN/m^2)	0.1×10^{9}
Shear Modulus, G (kN/m^2)	0.4×10^{8}
Axial Rigidity, EA (kN/m)	0.3658×10^{7}
Flexural Rigidity, EI (kN-m^2/m)	0.7143×10^{4}
Shear Rigidity, GA (kN/m)	0.146×10^{7}
Poisson`s Ratio, *v*	0.25

Bolt:

- Allowable shear stress = 69 MPa
- 1″ϕ bolt, shear capacity = 54 MPa
- Tensile yield stress = 230 MPa

Longitudinal Direction:

- EI = 2.3×10^7 kN-m^2
- Yield moment = 6.2×10^3 kN-m

4.5 Design Criteria

Design criteria were established, in consultation with the structural engineer to meet the structural and operational requirement of the subway tunnels during the excavation process. The geotechnical and structural criteria are listed in Table 2. Essentially, the displacements and change in shape, transversely and longitudinally, of the lining are computed, and the moments and forces imparted onto the structural elements evaluated and compared with the criteria to be satisfied.

Two types of analysis were performed to examine the effects of the excavation on the subway tunnels;

(i) in the transverse direction, a plane strain elasto-plastic finite element analyses were performed on the chosen critical section A-A on Figure 6 and is illustrated in Figure 9;

(ii) in the longitudinal direction, three-dimensional finite-layer analyses were performed. This analysis takes into account the three-dimensional nature of the unloading but does not explicitly model the tunnel itself. While this assumption is appropriate for the flexible lining in this project, it will not be adequate for a rigid lining.

The predicted longitudinal deflection of the east and west tunnels at different stages are shown in Figure 10 to Figure 12. Further details of analysis and interpretation of analysis results may be found in Lo and Ramsay (1991).

Table 2. Design criteria adopted for impact of excavation on tunnels.

(A) Geotechnical Criteria	
(1) Heave at crown	≤ 15mm
(2) Springline closure	≤ 10mm
(3) Crown-invert extension	≤ 10mm
(B) Structural Criteria: Based on computed deflections, check	
(1) Maximum bending moment, M= EI d^2y/dx^2	
(2) Maximum total shear, S= EI d^3y/dx^3	
(3) Maximum tension on bolts	
(4) Maximum shear force on bolts	

The results of analyses indicated that the proposed excavation with the final profile shown in Figure 8 was technically feasible without causing undue distress to the lining and it was decided that the construction would go ahead with a monitoring program to verify the design assumptions and construction control.

4.6 Field Monitoring and Construction Control

A monitoring program was developed to guard against any unpleasant surprises that might arise from variability of modulus profile, undetected disturbed zones around the tunnel, or close proximity of local excavation for caissons and transfer girders. At selected rings along both the northbound and southbound tunnels, the following measurements were carried out: (i) elevations of crown and invert by precise levelling; (ii) change in chord length at springline, shoulder and between crown and springline by tape extensometer.

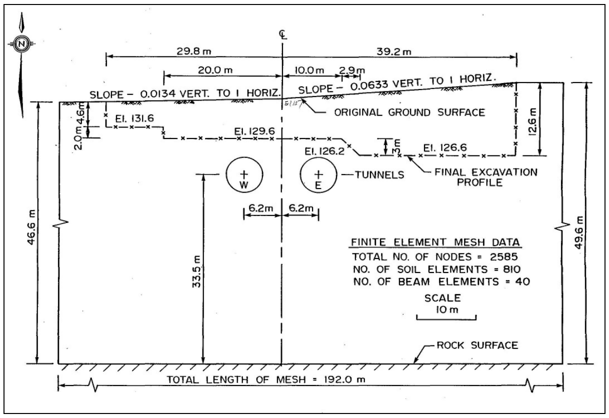

Figure 9. Condition modelled in transverse finite element analysis

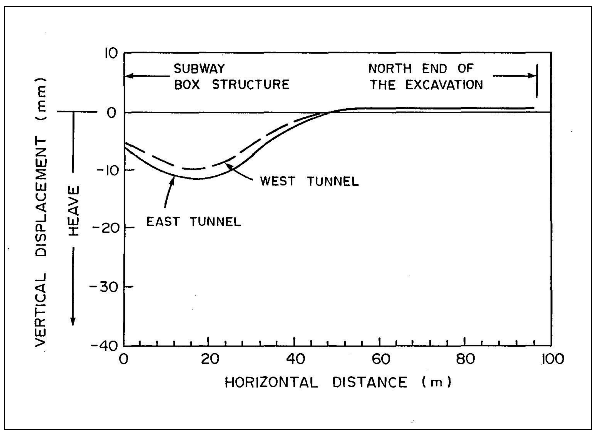

Figure 10. Predicted displacement profiles of the east and west tunnel for the area excavated prior to December 1, 1987 – prior to monitoring

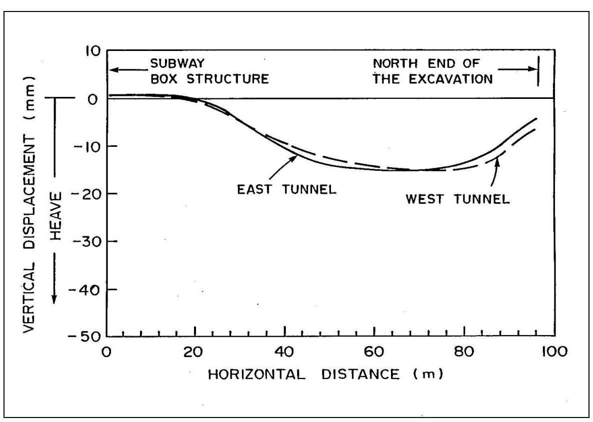

Figure 11. Predicted additional displacements along the east and west tunnels due to proposed additional excavation

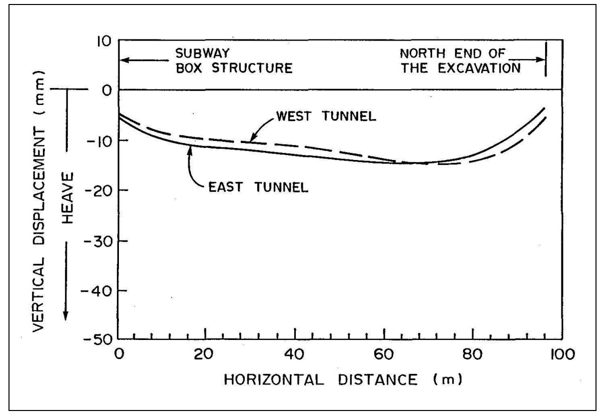

Figure 12. Predicted displacement profile of east and west tunnels after general excavation

Table 3. Comparison of predicted and measured distortion- two-dimensional analysis.

Area of Tunnel		Crown-Invert-Extension (mm)	Springline Closure (mm)
Profile of January 1988	southbound	6	4
	northbound	7	4
Profile of May 1988	southbound	9	5
	northbound	12	8
Measured (ring 94-107)	southbound	4	2.9-3.5
	northbound	4	3.8-4.4

Note: Design criteria crown-invert extension ≤ 10mm
Springline closure ≤ 10mm

It was established that for the general excavation to be completed without causing any distress to the tunnels, the following criteria should be met:

(i) heave at crown < 15 mm;
(ii) crown-invert extension ≤ 10 mm; and
(iii) springline closure ≤ 10 mm.

The excavation was divided into stages. The results of monitoring were transmitted to the engineers, who interpreted the results and made appropriate decisions on further excavation. Measurements usually were performed between 2 a.m. and 5 a.m., when the subway trains were not operating. The results were transmitted to the engineers in the morning to decide on the location of further excavation.

4.7 Comparison of Results of Field Measurements with Predicted Deformations

The results measured in the monitored rings between Rings 94 to 107 at locations of maximum deflections of the tunnels are summarized in Table 3 for comparison with predictions.

Table 4. Comparison of predicted and measured quantities and factors of safety.

	Predicted Values	Computed From Field Measurements of Deflections	Factors of Safety
Maximum bending moment (MN-m)	1.1	0.9	6
Maximum total shear (kN)	404	440	2.5
Maximum tensile stress on bolts (MPa/bolt)	50	41	4.5
Maximum shear force on bolts (kN/bolt)	12.6	13.8	2.5
Maximum heave (mm)	17.8	15	N/A

Note: Maximum allowable bending moment= 6.2 MN -m; Maximum allowable shear = 1115 kN.

It may be seen from Table 3 that the measured diameter changes are close to or below those predicted, and are also below the design criteria established.
The measured results are rather close to those predicted for the January 1988 profile. The reason for this agreement is that the final excavation profile adopted was not substantially different from the profile used in the analysis.

In the longitudinal direction, the deflections predicted for the northbound and southbound tunnels are compared with results of monitoring in Figure 13 and Figure 14 respectively. It can be seen that the maximum heaves measured were slightly below those predicted for both tunnels. It may be observed that the design criterion for heave was satisfied.

4.8 Factors of Safety in Structural Elements

From the deflection curves measured, the maximum bending moment and shear force may be calculated and the tensile stress and shear force acing on the bolts may be deduced.

The results of calculation of these quantities are shown in Table 4, together with the predicted values. The factors of safety of the bolts against tension and shear as a result of longitudinal deflection of the tunnel are also shown. It may be seen that the agreement between predicted values and those deduced from field deflection curves is satisfactory. There are also adequate factors of safety in different structural elements composing the lining.

It may therefore be concluded that general excavation to the final grade is in accord with design expectations. The chronology of events in this case history is shown in Table 5.

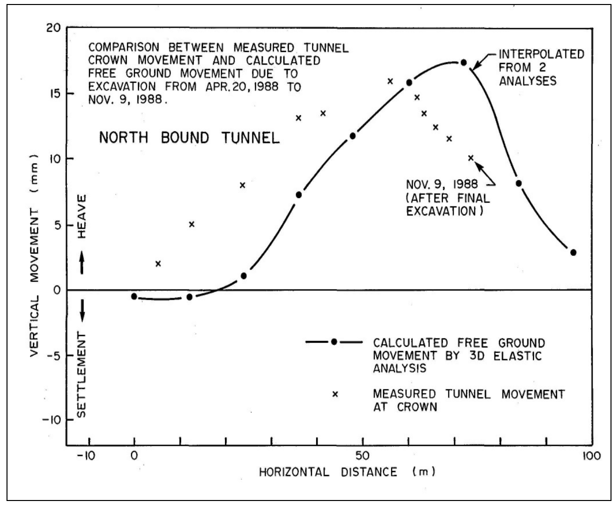

Figure 13. Comparison between predicted and measured heaves – northbound tunnel

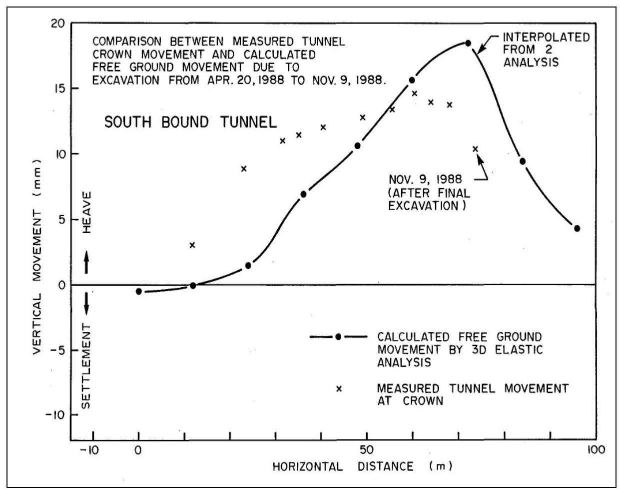

Figure 14. Comparison between predicted and measured heaves – southbound tunnel

4.9 Conclusions

While a number of observations may be made in this case history, the main conclusions resulting from this study are:

1. The methodology, as described, has been used for the successful completion of several deep excavations close to existing tunnels.
2. The single most important soil parameter that governs the design is the magnitude and distribution of the deformation modulus in unloading. Values obtained from conventional sampling and testing may lead to the wrong conclusion that the project is not feasible.

Table 5. Chronological event.

1	March 30, 1987	Conventional soils investigation and report
2	Sept. 2, 1987	Excavation started adjacent to TTC Station (Hatch retained K.Y. Lo Inc.)
3	Sept. 1987	Preliminary analyses (2D FE and 3D Finite Layer) indicated 75 mm heave and tension and shear in bolts exceeded capacity using E = 100 MPa. Required detail testing and analyses.
4	Nov. 1987	Completed extension and compression tests on block sample taken at E1 126.
5	Oct. 22 – Dec. 1, 1987	Heave monitoring to evaluate modulus distribution with depth. Excavate from E1 133 to 128.5 (sloping 127 to 130 S to N) No dewatering within 50 m of area so that monitoring was not affected.
6	Jan. 1988	Report 1 – excavation technically feasible. Rec. monitoring of tunnel lining and piezometers
7	March 1988	Project management proposed deepening of excavation north of column P to a maximum of 3.3 m. Re-analyses was performed.
8	May 3, 1988	Report 2 – detailed recommendation of monitoring and staging of excavation, delineation of loose zones behind tunnel lining
9	April 20 to Nov 9, 1988	(Completion) Monitoring between 2 a.m. and 5 a.m. Results reported at 8 a.m. Comparison of field measurements with predictions. Decide on next stage of excavation.
10	September 1990	As-built report submitted.

5 THE BOW TOWER EXCAVATION, CALGARY, ALBERTA

5.1 Introduction

The excavation for the construction of the Bow office tower involved two city blocks. It covers a footprint of 17,000 m^2, 190 m long, 100 m wide and 21 m deep. It covers a footprint of 17,000 m^2, 190 m long, 100 m wide and 21 m deep. As shown in Figure 15, a number of structures exist at close proximity to the excavation. These structures include the Telus building in the north, the Calgary Light Rail Transit in the south, the Petro-Canada Centre in the west, the Andrew Davidson building and the historic Royal Canadian Legion No. 1 building in the south-east side.

The geological conditions at the site include 7 m of fluvial deposit of sand gravel and cobbles, followed by a deep deposit of the Porcupine Formation consisting of interbedded mudstone, siltstone with occasional sandstone. The water table fluctuates between 3.5 m to 6 m depth.

It is known from local experience that excavation into the Porcupine Formation may lead to excessive deformation of the rock impacting on adjacent structures. It is also known that shear bands may exist which is the main cause of large displacements but the presence, location and identification are difficult to establish. To account for these and other additional uncertainties a comprehensive monitoring program was carried out by the design/build team.

The instrumentation program consists of twelve (12) inclinometers, anchored to a depth of generally 13 m below the excavation bottom, six multiple point extensometer anchored 25 m behind the wall. The inclinometers are labeled I01 to I012 in Figure 15. In addition, survey points were established in all surrounding structures.

The support system of the excavation consisted of anchored caisson walls on all sides 2 m into rock, followed by anchored shotcrete walls of 75 mm thickness.

Figure 15. The Bow site: (1) The Bow; (2) Petro-Canada Centre; (3) Telus; (4) Calgary's Light Rail Transit; (5) The Royal Canadian Legion No. 1; (6) Chamber of Commerce and (7) Andrew Davidson Building (modified from Lo et al. 2009)

Table 6. Anchor properties for the analyses.

Anchored caisson wall – 2 m into rock followed by anchored shotcrete wall (75m)
Caisson wall: 800 m interlocking with soldier piles set between tow filler piles
Anchor details used in analysis

Anchor No.	A[1]	B[1]	C[2]	D[2]	E[2]	F[2]
Pretension force (MN)	0.63	0.63	0.36	0.36	0.36	0.36
Tensile capacity (MN)	1.05	1.05	0.69	0.69	0.69	0.69
Total length (m)	15	13	12	12	13	9
Bonded ratio (%)	46	54	50	50	60	67
Depth (m)	3.50	7.25	10.25	12.25	14.25	16.25

[1]Lateral spacing is 2.1 m
[2]Lateral spacing is 3.0 m

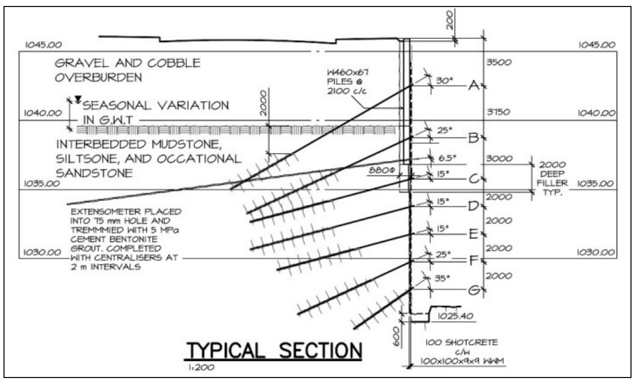

Figure 16. The Bow excavation –typical section

The caisson wall was 800 mm in diameter interlocking with soldier piles set between two filler piles. The general arrangements of the anchors are shown in Figure 16. Details of the anchor capacities are shown in Table 6. It should be noted anchors were added whenever required during construction when impending problems became evident.

5.2 Uncertainties in the Bow Excavation and the Approach to Address the Uncertainties

As excavation proceeded and results of monitoring became available, it became evident that potential threat of excessive deformation exists. Figure 17 shows that even though the excavation was only 12 m deep and was 6 m into rock and 14 m above the shear band, the displacement at the shear band had already been triggered resulting in a displacement of 10 mm. It was therefore necessary to define the uncertainties involved in the design problem and address these issues so that construction may proceed with a better understanding of the rock behaviour for construction control.

The principal uncertainties in the Bow excavation include:
(i) rock properties were not precisely known;
(ii) the presence and locations of shear bands and other weak zones were not known; and
(iii) the magnitudes and directions of the in situ horizontal stresses in the rock formation were not known.

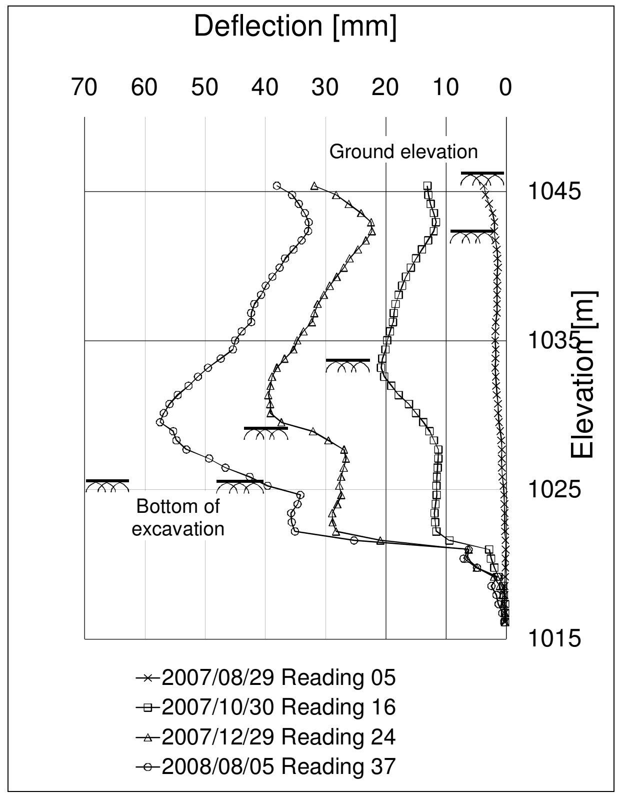

Figure 17. Inclinometer 4 Readings

The approach to address issues involved the following processes:
(i) rock properties were re-evaluated from results of site investigation reports. Additional laboratory tests were performed on specimens from block samples recovered during construction;
(ii) presence and locations of weak zone/shear bands were identified from results of monitoring and from exposed rock surface as excavation proceeds; and
(iii) the magnitudes and distribution of the rock stress were chosen based on experience elsewhere, and the assumptions were continuously examined by analysing the monitoring results as excavation proceeded to final grade.

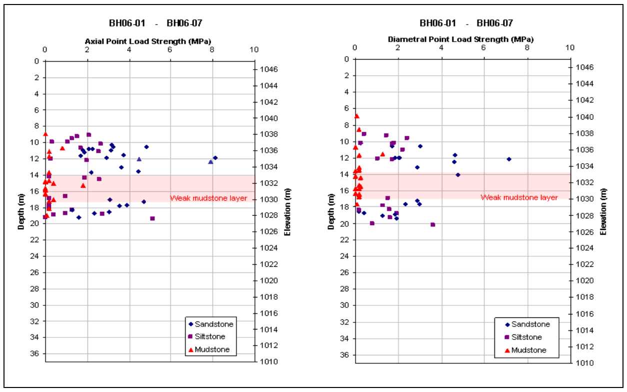

Figure 18. Point load strength profile

A detailed review of geotechnical site investigation report was performed. A summary plot of all the point load test results from BH06-01 to BH06-07 in both the axial and diametral directions are shown in Figure 18. It can been seen from the results of diametral point load test that there is a clear demarcation of a weak zone of mudstone exist within the depths of 13.5 m to 17 m across the area of the excavation. The results of axial tests did not give a distinct trend. The difference in behaviour is due to the layered nature of the rock. Failure in the axial direction tends to involve more than one layer while failure in the diametral direction tends to be tensile fractures in the weak mudstone. It will be shown that this weak zone correlates well with results of inclinometer monitoring. It also played a major role in contributing to some significant movements to the surrounding structures.

To obtain the required strength and deformation parameters, block samples were recovered from the bottom of the excavation at the location shown in Figure 15. The highly fissile nature of the mudstone rendered both sampling and preparation of specimen for triaxial and shear box tests very difficult. Nonetheless, adequate test results were obtained for the on-going analysis and excavation process.

The results of shear box and triaxial tests are shown in Figure 19. To verify the reliability of test results, tests were performed at two laboratories. It can be seen that the results were quite consistent, yielding peak strength parameters of c' = 0.35 MPa and ϕ = 24° and residual parameters c_r' = 0 MPa and ϕ_r' = 15°. Further details of test results and deformation parameters obtained therefrom may be found in Lo et al (2009). The engineering parameters employed in numerical analysis for each rock unit are summarized in Figure 20.

It appears that no in situ rock stress measurements have been carried out in the near surface rock formations in the Calgary area. Therefore, the magnitudes and directions of the in situ horizontal stresses which provide the driving force in rock deformation are not known. Based on experience of stress measurements in weak rock formation elsewhere, it was assumed that the magnitude of the horizontal stress may lie between 1 to 2 MPa. The distribution of assumed horizontal stress is shown in Figure 21, together with the vertical stress distribution based on overburden weight.

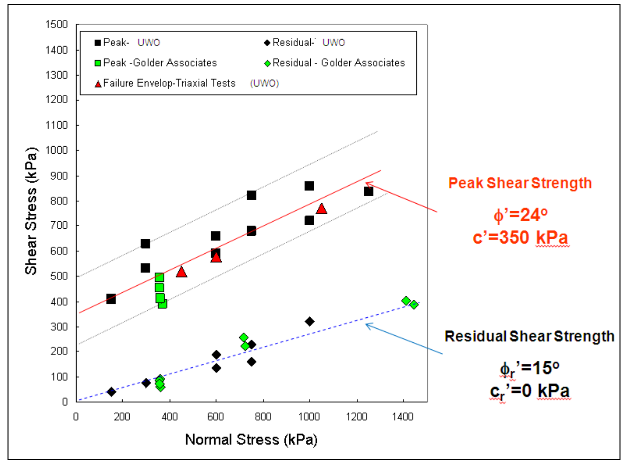

Figure 19. Multistage direct shear test results

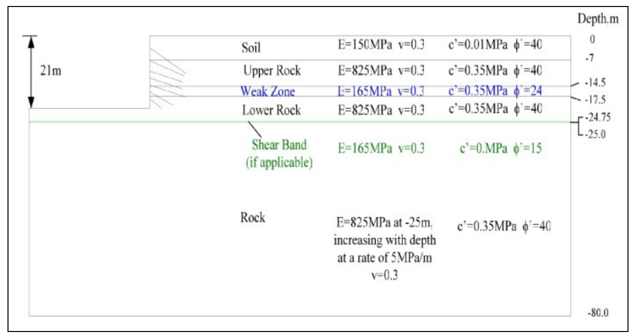

Figure 20. Engineering Parameters

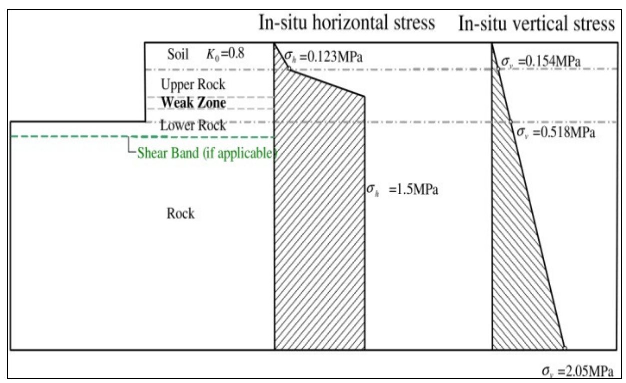

Figure 21. Assumed vertical and lateral *in-situ* stresses

5.3 Numerical Analysis

Numerical analyses have been performed at various locations around the excavation. For the purpose of this presentation, the results on the movements of the north wall and east wall are described. At these two locations, the excavation progress was close to plane strain condition. Several programs including Phase II, Plaxis and Abaqus were used. Comparisons of results obtained showed very little difference between these programs.

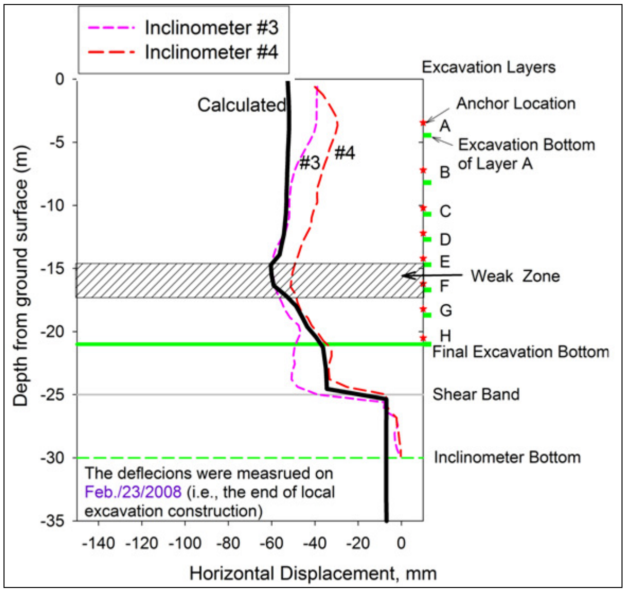

Figure 22. Calculated and measured deflections- North wall

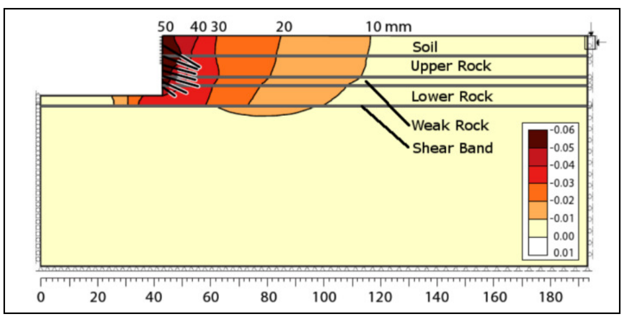

Figure 23. Computed horizontal displacements- North wall

5.3.1 North Wall (I03 and I04)

The results of analysis at the north wall are compared with the results of monitoring from inclinometers No. 3 and No. 4 as shown in Figure 22.

The measured displacements from the two inclinometers are quite comparable and exhibit the same characteristics in the distribution of movements with depth. The calculated displacements show the same features. In particular, the following characteristics of movements may be observed.

(i) The shear band induced an abrupt displacement of about 30 mm at a depth of 4 m below the bottom of the excavation.

(ii) The weak zone between 14.5 m to 17.5 m depth caused a bulging effect at its corresponding elevations consistent with the results of diametral point load tests shown in Figure 18.

(iii) The magnitude and depth of maximum lateral movement (60 mm) from monitoring and computations are consistent. The pattern of deformation behind the excavation is shown in Figure 23.

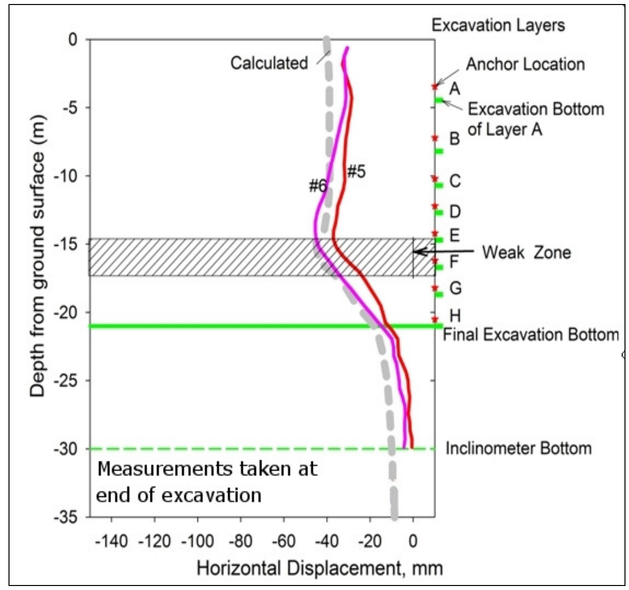

Figure 24. Calculated and measured deflections – East wall

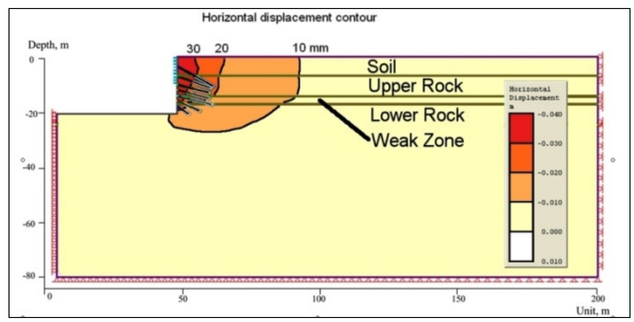

Figure 25. Computed horizontal displacement – East wall

It is significant to note that the displacement contours show a clear discontinuity at the shear band, indicating interface slip. This interface slip resulted in substantial lateral displacements far away from the excavation face as shown in the figure. Thus, at a distance approximately 4 times the depth of the excavation, the horizontal displacement is still as much as 10 mm.

The maximum calculated settlement is 13.3 mm at a distance of about 10 m from the north wall. The maximum base heave is 28 mm at a distance of about 12 m from the excavation face. While no precise measurements were made with regard to these quantities, general observations at the site are quantitatively consistent with the estimates.

5.3.2 East Wall (I05 and I06)

The calculated and measured deflections at the East Wall at locations of inclinometer 5 and 6 are shown in Figure 24. At these locations the shear band did not exist but the effect of the weak zone was apparent. No abrupt discontinuity of the displacement distribution with depth occurred. As can be seen in Figure 24, the results of numerical analysis represent the measured deflections of two inclinometers quite satisfactorily both in magnitude and distribution.

The general pattern of horizontal displacement is shown in Figure 25. The calculated near surface lateral displacement is about 30 mm at 7 m, 20 mm at about 17 m and 10 mm at about 40 m behind the wall. A comparison of Figure 23 and Figure 25 highlights the effects of the shear band as follows:

(i) introduces interface slip and consequently a discontinuity displacement contour;

(ii) contributes to larger displacement and,

(iii) propagates far-field displacement behind the wall face.

5.4 Occurrence of Shear Band and Weak Zone

From results of inclinometer monitoring, it is possible to identify the occurrence and location of the weak zone and shear band across the site. The results are shown in Figure 26. It is noted that the weak zone was present at all locations monitored by the inclinometers.

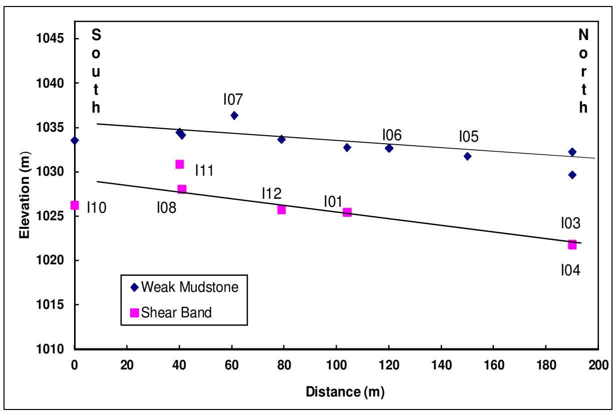

Figure 26. Inclination of the weak zone and shear band across the site

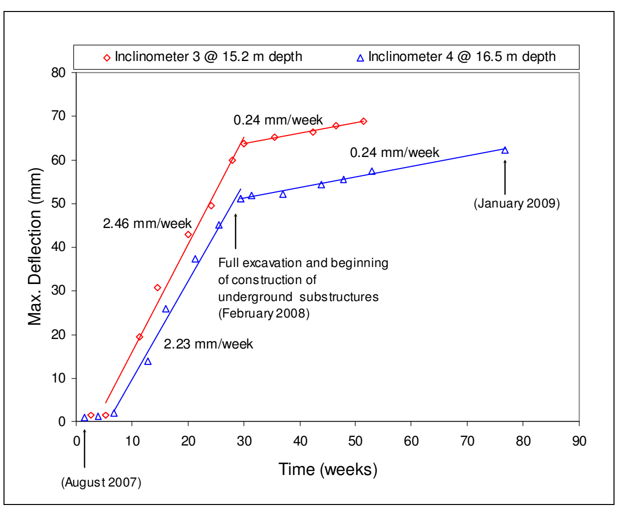

Figure 27. Maximum deflection vs. time inclinometers at the north wall of the Bow excavation

The shear band, however, was not exhibited at locations of I05, I06 and I09. It is also interesting to note that both the shear band and weak zone have apparent movement towards the Bow River in the north.

The effects of these two geological features, combined with relatively high horizontal stresses resulted in significant lateral displacements. The impacts on the surrounding structures are illustrated by the photos shown.

5.5 Time-Dependent Deformation

Monitoring of movements was continued after completion of excavation at the North Wall area. The results of observations on movement with time from I03 and I04 are shown in Figure 27. It can be seen that there was a clear trend of time-dependent deformation of the rock at a rate of 0.24 mm/week after completion of excavation. The results are consistent with the results of a limited number of swelling tests performed on specimens of block samples. Although time-dependent deformation is not an issue in this project, future development involving underground structures may have to take this behaviour of the Porcupine Formation into consideration.

5.6 Conclusions

Based on the results of laboratory tests, field monitoring and numerical analyses of the movements of the caisson wall and the surrounding structures, the following conclusions may be drawn.

1. The rock at the site contains geological weakness in the form of shear bands and weak zones in the mudstone.
2. These features contribute predominantly to the large deflection of the caisson wall.
3. The shear band, generally located about 4 m below excavation bottom, can be easily triggered even though excavation is still well above its location. The shear band not only causes substantial displacements but also induces far field movement behind the caisson wall.
4. The weak zone also contributes significantly to the total deformation.
5. The rock of the Porcupine Formation may possess high horizontal stresses, similar to rock formations such as those in Southern Ontario although the magnitude is smaller. It has been deduced that the magnitude may be in the order of 1.3 to 1.5 MPa but the directions and magnitudes of the major and minor principal stresses in the horizontal plane are not yet known. There is an urgent need for in situ rock stress measurements to verify these results for the design of structures in rock in these areas.

6 ACKNOWLEDGEMENTS

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