Hand-mined tunnel crossing of a railway in clay till

Melanie Woytiuk & Heinrich Heinz
Thurber Engineering Ltd., Calgary, Alberta, Canada
Ryan Kidd
Water Resources, City of Calgary, Calgary, Alberta, Canada

ABSTRACT
The Shepard tunnel is located near the eastern boundary of the City of Calgary, close to the hamlet of Shepard, Alberta. The tunnel is 63 m long and crosses under five sets of active Canadian Pacific Railway tracks. It has a horseshoe shaped section and was hand-excavated, with primary support consisting of steel ribs and steel or wood lagging. The host ground is predominantly a moderately overconsolidated clay till. This paper reviews the geotechnical field program, laboratory testing and site conditions. Allowable track settlements are compared to predicted and actual measurements taken during construction.

RÉSUMÉ
Le tunnel Shepard est situé à l'extrémité est de la ville de Calgary à proximité du village de Shepard en Alberta. Le tunnel est 63 m de long avec une section transversale en forme de fer à cheval. Il croise cinq voies ferrées actives du Chemin de fer Canadien Pacifique. Le tunnel a été creusé à la main, et est principalement supporté par des blindages en acier et en bois. Le sol d'assise essentiellement est un till argileux légèrement surconsolidé. Cet article examine la reconnaissance géotechnique et les essais de laboratoire effectués, ainsi que les conditions sur le terrain. Les tassements admissibles des voies ferrées sont aussi comparés aux tassements prévus et observés pendant la phase de construction.

1 INTRODUCTION

The $72M Shepard Stormwater Diversion Project project is currently under construction with completion scheduled for late 2008. The project itself was divided into five phases for construction and tendering purposes; Phase 3 being the Canadian Pacific Railway (CPR) crossing. When it becomes operational, the project will direct stormwater to a new Wetland Treatment Facility near the east boundary of Calgary, and from there to the Bow River. The constructed wetland will be the largest in Canada with a 227 hectare footprint at full capacity.

One of the key components of this project is the crossing of the CPR tracks, located immediately west of the hamlet of Shepard. CPR operates three siding tracks and two mainline tracks at this crossing location (Figure 1). The crossing allows the necessary 44 m^3/s of stormwater to pass under the tracks on the way to the Shepard Wetland. Geometric and hydraulics constraints dictated the requirement for a tunnel crossing, with a hand-excavated method being chosen because of the short length (63 m), and available expertise from the City of Edmonton Asset Management and Public Works Group.

The City of Edmonton was retained as a sole source contractor on a cost plus contract basis. A hand excavated tunnel crossing was designed utilizing a horseshoe shaped, approximately 4 m high by 4 m wide section. The initial liner consisted of steel ribs and lagging. The permanent tunnel liner consisted of a cast in place concrete conduit with a finished internal diameter of 3 m set on a slope of about one percent.

Figure 1. Oblique aerial photograph of tunnel location

Due to the size of the tunnel and proximity of the two main high speed CPR tracks, there were numerous requirements that needed to be met before CPR would allow construction to take place. CPR indicated that the speed of the trains could not be reduced and a track monitoring program needed to be in place before the start of construction. CPR also requested an estimate of potential track settlement and a third party review of the tunnel design.

As part of the application process, the City of Calgary, in association with Stantec Consulting and Thurber Engineering, made several submissions detailing the unique aspects of the tunnel construction and addressing specific questions by CPR and their reviewers before approval for construction was issued.

2 GEOLOGICAL SETTING

The surficial soil deposits in the area of the tunnel are primarily of glacial origin, and were deposited during sequences of advance and retreat of the last continental ice-sheet from the northeast. Based on the surficial geological mapping of the area (Moran, 1986) the youngest of the glacial tills is attributed to the Crossfield Formation, and overlays tills of the Balzac and possibly Lochend ages.

The glacigenic topography of the area has been described as hummocky moraine, which has been thought to have resulted from the reworking of superglacial till from stagnant ice blocks. However, Eyles et al (1999) formulated a subglacial model that described the formation of hummocky moraines, where the overburden pressure of the stagnated ice results in the squeezing up of till from under the ice. Adams et al (2008) concluded that the tills in the Shepard area are subglacial in origin, as the area is in close proximity to subglacial landforms and the tills present characteristics common to subglacial tills (high clay content, striated clasts) and are moderately overconsolidated.

In late glacial and post glacial times, fine-grained sands, silts and clay lake deposits formed in the lows on the hummocky moraine surface. Organic sediments are often incorporated in these deposits. Many of the low areas have been infilled due to farming activities, and possibly the construction of the railway. The bedrock in the study area consists of interbedded sandstones, siltstones and claystones of Upper Cretaceous age.

3 GEOTECHNICAL CHARACTERISTICS

3.1 Geotechnical Investigations

Geotechnical investigations were carried out in the area in 1998 and 2001. Figure 2 shows the stratigraphy along the tunnel centre line. Both investigations included drilling (solid stem auger) and coring of the bedrock, with representative samples from the auger flights and Standard penetration tests collected and retained for laboratory testing. Cone penetration testing and insitu hydraulic conductivity testing in the soils and in the underlying bedrock were also completed. The laboratory program included extensive visual classification, natural moisture content determination and soil index testing, as well as a limited number of strength and consolidation tests on bedrock core and clay till samples collected from Shelby tubes.

3.2 Subsurface Conditions

In general, the soil stratigraphy consisted of about 1 m to 2.5 m of variable fill and topsoil materials, in some locations underlain by stiff silty clay and / or fine grained silty and clayey sand. These layers were underlain by clay till, which is the dominant host soil along the tunnel alignment. The clay till is predominantly medium plastic, stiff to very stiff, with natural moisture contents near its plastic limit. The clay till is underlain by a dense silt till which, in places, grades to a sand till. The silt/sand till is underlain by bedrock below the base of the tunnel.

Based on the tests performed and previous experience in the Calgary area, the relevant properties of the clay till unit can be summarized as shown in Table 1. Using Terzaghi's classification, the clay till was classified as "firm ground" prior to construction, however some areas near the crown were re-classified as "slow ravelling" based on observations made during construction. Comparison of properties from this "Shepard Till" and other tills from west Calgary and downtown Edmonton performed prior to construction indicated that index properties for all these tills were quite comparable, whereas there was a wide variation in consistency and strength (i.e., the Shepard till was typically weaker than the others).

Groundwater levels in the fall of 1998 and in the winter of 2001 were up to 3.5 m above the tunnel crown. Water level measurements taken in the summer of 2006 indicated that the groundwater level had risen by as much as 2.2 m, and was up to 5.0 m above the crown of the tunnel. This high groundwater was seen as a potential concern during the design stages, as the clay till contained sand layers or lenses; however most of these were observed to drain quickly after excavation.

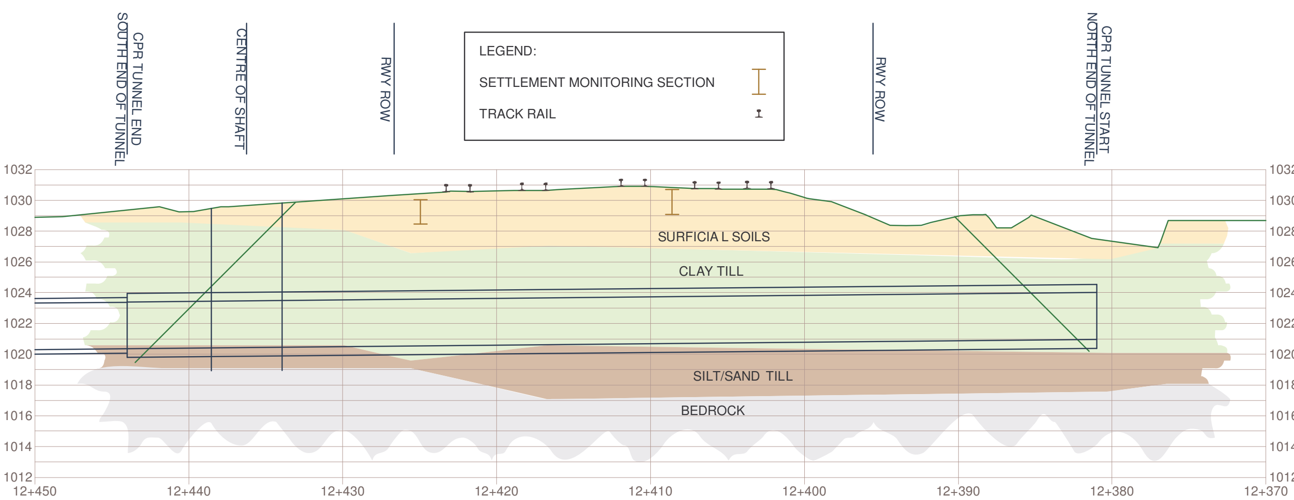

Figure 2. Longitudinal section

Table 1. Properties of Clay Till Unit

Properties	Range (“suggested”)
Bulk Density (kN/m^3)	21 – 22
Undrained Shear Strength (kPa)	30 – 200 (170)
Drained Friction Angle (degrees)	30 – 36 (32)
% Clay	12.4 – 28.0 (24)
% Silt	36.0 – 61.3 (41)
Modulus of Elasticity (MPa)	15 – 80 (20)
Insitu Hydraulic Conductivity (cm/s)	3E-07 - 5E-07
Pre-consolidation Pressure (kPa)*	300 – 520 (375)
Overconsolidation Ratio*	2.5 – 4.2 (3.0)
Coefficient of Earth Pressure at Rest	0.7 – 0.8 (0.7)

* Based on one test / different interpretation methods

The soil stratigraphy and laboratory data relevant to this paper are summarized in Figure 3 and Table 1. A cone factor (N_{kt}) equal to 17.5 was used for calculating the undrained shear strengths shown in Figure 3.

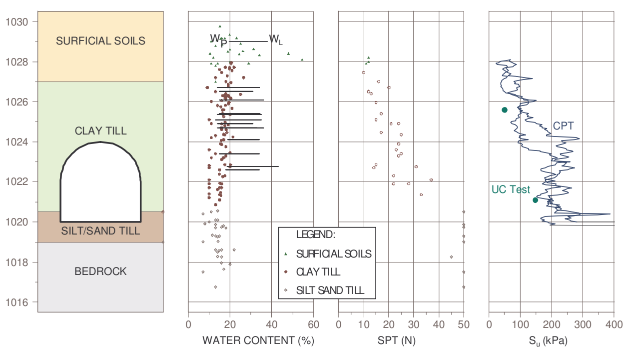

Figure 3. Soil stratigraphy and laboratory data

4 OVERVIEW OF DESIGN AND CONSTRUCTION

As noted earlier, the tunnel excavation incorporated an approximately 4 m wide by 4 m tall horseshoe shaped section. The depth to the crown ranged from 6.4 m to 6.7 m below the railway tracks, which corresponds to about 1.5 times the tunnel diameter. Accordingly, for design purposes, the tunnel was treated as a shallow tunnel.

The stability number was estimated at 1.8 prior to construction, using Equation 1. In this equation σ_v is the total vertical stress at tunnel axis level, p_i is the internal tunnel support pressure (assumed to be zero), and S_u is undrained shear strength of the soil, conservatively assumed at 100 kPa.

$$N = (\sigma_v - p_i) / S_u \qquad [1]$$

The critical stability ratio N_c was estimated to range between 5.0 and 7.0 using solutions presented by the Pipe Jacking Association (1995), indicating a factor of safety against face collapse in the order of 3 to 4. The heading was therefore considered stable in the short term, with the relatively high safety factors against collapse interpreted as indicative of potentially small ground strains.

Design of the initial lining was the responsibility of the contractor, and it is understood to have been based primarily on previously used successful designs. Design of the final concrete lining was performed assuming a condition of full-overburden pressure plus surcharges associated with four stationary and one running train, as calculated using a two-dimensional linear elastic finite element model. Stationary loads were based on a line load of 136 kN/m uniformly distributed over 2.6 m long ties (i.e., 53 kPa). A magnification factor of 3.0 was utilized on the running track, to account for the dynamic loading (i.e., 169 kPa).

Selection of the excavation methodology and initial support was primarily based on the past experience of the City of Edmonton crews with tunnels excavated in similar ground conditions. Excavation was performed using a heading and bench technique, using jack-hammers and shovels, with the spoil removed using a cart pulled along a track to the entry shaft. An average of 6 workers were employed per shift during excavation and lining placement operations.

The initial support was steel ribs and wooden lagging, with the ribs specified at 0.9 m and 0.6 m centres outside and within the CPR right-of-way, respectively. It is noted that, at CPR's request, within their right-of-way, the wooden lagging in the upper portion of the tunnel was replaced by relatively thin steel profiles. This measure was aimed at limiting void formation due to long term deterioration of the lagging.

W100 x 19 steel ribs were used in the crown and sides of the tunnel, and for the temporary steel bracing at the heading's temporary invert. A W150 x 37 steel horizontal spreader was used at the tunnel's permanent invert, as illustrated in Figure 4.

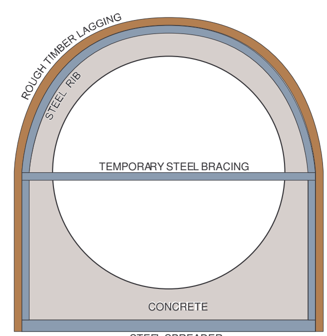

Figure 4. Tunnel cross-section

Excavation of the tunnel progressed from the south entry shaft towards the north. Two twelve hour shifts were employed while the excavation face was within the railway right-of-way, generally with the heading advancing during the night shift, and the bench advancing during the day shift. This procedure resulted in an average advance rate of slightly over 1 m in 24 hours.

The construction procedure described above was modified at about 21 m advancement from the entry shaft after a noticeable reduction in the stand-up-time of the ground in the upper portion of the tunnel, evidenced by small cave-ins at the face and at the crown, was observed. The modification entailed excavation of a “long” heading all the way to the end of the tunnel, followed by excavation of the bench. By altering the construction procedure, and continuously excavating the heading with immediate placement of the lining, better ground control was achieved.

5 SETTLEMENT MONITORING PROGRAM AND PERFORMANCE

An extensive instrumentation program was installed prior to construction at the request of CPR, to monitor the effects of tunnel construction on the overlying railway tracks. A total of 17 surface settlement points were monitored daily during construction using precise levelling, with the results being processed immediately and compared to allowable criteria set out by CPR. In addition, an automated settlement monitoring program using electrolevels installed directly on the railway tracks was implemented, with data processed and made available to all affected parties through a web based system. Alarm notifications were sent out via email when the programmed threshold values of tilt or settlement had been reached.

At this specific location, there were three siding tracks designated as Class 2, with a maximum speed of 25 mph, and two mainline tracks designated as Class 4, with a maximum speed of 55 mph. CPR provided a table with allowable track surface defects, which indicated that for the track speeds and geometry at this location, maximum allowable settlements were in the range of 32 mm to 38 mm.

Actual settlement data for similar railway crossings using hand-mined tunnel techniques was found to be scarce. Predictions of surface settlements were carried out prior to construction assuming ‘greenfield’ conditions, and the shape of the transverse settlement trough above the tunnel represented by an error function, as described by O’Reilly and New (1982). The transverse distance from the tunnel centerline to the point of inflection (i) of the trough was estimated using Equation 1, proposed by O’Reilly and New (1982) for cohesive soils (where Z is the depth to tunnel axis, and i and Z in metres).

$$i = 0.43\ Z + 1.1 \qquad [2]$$

The maximum short term settlements were calculated assuming that the volume of the surface settlement trough (V_s) is a percentage of the excavated tunnel volume (V_{exc}), also called the surface settlement / tunnel volume ratio (V_s/V_{exc}). Case history data for hand excavated tunnels in comparable soils indicated values of V_s/V_{exc} typically in the range of 1% to 3% (Peck, 1969; O’Reilly and New, 1982). Corresponding short term settlements were estimated to be in the range of 12.5 mm to 38 mm, which were considered acceptable when compared to the CPR criteria. The 12.5 mm settlement, corresponding to about V_s/V_{exc} = 1%, was selected as the initial alarm level, which would trigger a review of construction procedures by the project team, in conjunction with the CPR. It is also noteworthy that the predicted settlements were estimated to increase by up to 20% due to consolidation using Schmidt’s (1989) empirical procedure, with the final values attained within approximately one year. The ‘long-term’ settlement trough was flatter, i.e. differential settlements transverse to the tunnel centerline were predicted to decrease with time.

Predicted settlement troughs for V_s/V_{exc} = 1% and 3% are given in Figure 5, together with the actual settlement readings for the two settlement monitoring sections shown in Figure 2. The settlement readings are comparable at the two sections. Settlement data are presented for the “short term” and “final” conditions. For the purposes of this project, the situation where the excavation face had progressed about 2.5 tunnel diameters beyond the instrumented section is denoted as “short term” and the “final” condition was defined as a rate of increase of surface settlement of less than 1 mm/week. The “final” condition was attained approximately 5 months after completion of excavation and placement of the initial support at the instrumented section. The 2.5 diameters settlement is usually accepted as the “short term” value associated with excavation of the tunnel, with any additional settlements attributed to time-dependent processes such as soil consolidation and/or creep.

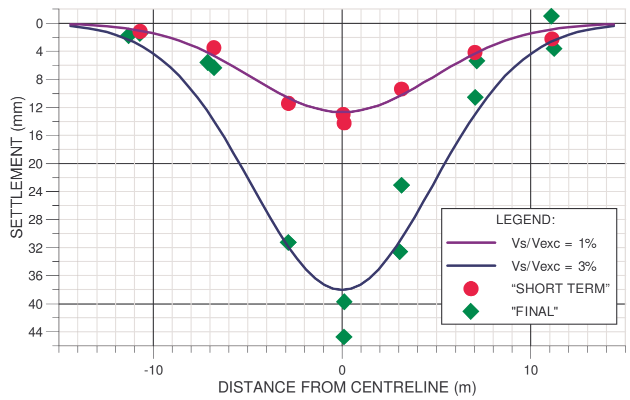

Figure 5. Predicted and actual surface settlement troughs

A reasonably good agreement was obtained between the V_s/V_{exc} = 1% curve and the short term settlements, suggesting that this settlement/tunnel volume ratio was reasonable for estimation of short term settlements. The final settlements were, however, much larger than predicted, and closer to the 3% settlement/tunnel volume ratio. It is also apparent that the actual settlement trough is markedly narrower than that predicted, suggesting that some time-dependent failure mechanism may have developed above the tunnel crown. It should be noted that both convergence measurements taken inside the tunnel, and the subsurface settlements measured near the tunnel crown using multipoint extensometers did not indicate a significant time-dependent increase. This behaviour is still being investigated and it is the intention of the authors to present a more detailed discussion in an upcoming paper.

6 CONCLUDING REMARKS

During the planning and design stages of this project, published performance data for railway crossings using similar tunnelling techniques was found to be scarce. An attempt was therefore made to document the geotechnical properties, construction procedures, and settlement performance, to provide a meaningful reference for future projects. Estimates of surface settlements prior to construction using simple and well established empirical techniques were found to agree reasonably well with actual short term performance. However, a significant component of the surface settlement developed after the primary lining was fully installed, and is believed to be associated with a failure mechanism which developed above the tunnel, together with time dependent creep and/or consolidation phenomena. Final surface settlements were, however, within limits acceptable to the CPR.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the contribution of a number of organizations and individuals to this paper. The City of Calgary, for allowing for the data from this project to be used in this format. Personnel at Stantec Consulting and the City of Edmonton, for their work before and during the construction of the tunnel. Thurber staff, especially D. Gorling and C. Holmes for their help during installation of the monitoring instrumentation, and S. Coates and B. Ramdharry for their assistance in the review and preparation of the paper. Acknowledgements are extended to an anonymous reviewer for the constructive comments provided.

REFERENCES

Adams, K., Osborn, G., Heinz, H. and Woytiuk, M. 2008. Origin of Hummocky Moraine in the Calgary Area, Alberta, Canada (Poster). *Cordilleran Section (104th Annual) and Rocky Mountain Section (60th Annual) Joint Meeting, Geological Society of America*, Las Vegas, Nevada, USA.

Eyles, N., Boyce, J.I, and Barendregt, R.W., 1999. Humocky moraine: sedimentary record of stagnant Laurentide Ice Sheet lobes resting on soft beds. In: Sedimentary Geology 123, pp. 163-174.

Moran, S.R., 1986. *Surficial geology of the Calgary urban area.* Alberta Geological Survey, Edmonton, Alberta, Canada.

O'Reilly, M.P. and New, B.M., 1982. Settlements above tunnels in the United Kingdom; their magnitude and prediction. *Tunnelling '82; Third International Symposium*, Brighton, East Sussex, United Kingdom, 173-181.

Peck, R.B., 1969. Deep Excavations and Tunneling in Soft Ground. Proc. 7th Int. Conf. Soil Mechs. And Fdn. Engineering, Mexico City, State-of-the-art Volume, pp. 225-289.

Pipe Jacking Association. 1995. *Guide to best practice for the installation of pipe jacks and microtunnels*. Pipe Jacking Association, London, United Kingdom.

Schmidt, B. 1989. Consolidation settlement due to soft ground tunneling. *12th Int. Conf. on Soil Mechanics and Foundations Engineering*, Rio de Janeiro, Brazil, 2: 797-800.