

Influence of exsolved gases on slope performance at the Sarnia approach cut to the St. Clair Tunnel

J. Paul Dittrich, R. Kerry Rowe, Dennis E. Becker, and K.Y. Lo

Abstract: In 1993, over 100 years after the completion of the original St. Clair Tunnel and its approach cuts, work commenced on the new St. Clair Tunnel. The new tunnel used the existing approaches, but required additional excavation to widen and deepen the original cuts. In Sarnia, the new work initiated unusual deep-seated deformations on the south slope of the approach. Effective stress finite element analysis (FEA), using an elliptical cap soil model coupled with Biot consolidation theory, was used to model the 1993 construction, but initial predictions were unable to capture the trend of deformations noted in the field. Naturally occurring gases are frequently encountered near the base of the overburden in the Sarnia area and this phenomenon was observed during drilling investigations in the Sarnia approach cut. Including the effects of the presence of exsolved natural gases in fine-grained soils subjected to unloading in the FEA results in substantially better predictions in the trend of deformations on the slopes of the approach cut.

Key words: natural gases, excavation, slope deformation, St. Clair clay plain, finite element analysis.

Résumé : En 1993, plus de 100 ans après la fin du tunnel original de St. Clair et ses coupes d'approche, le travail a débuté sur le nouveau tunnel de St. Clair. Le nouveau tunnel a employé les approches existantes mais a exigé l'excavation additionnelle pour élargir et approfondir les coupes d'original. À Sarnia, le nouveau travail a lancé des déformations peu communes situées en profondeur sur la pente du sud de l'approche. L'analyse par éléments finis (« FEA ») en contrainte effective utilisant un modèle elliptique de sol couplé à la théorie de consolidation de Biot est employée pour modéliser la construction de 1993 mais les prévisions initiales ne peuvent pas prendre en compte la tendance des déformations observées sur le chantier. Des gaz naturels sont fréquemment produits près de la base des terrains de recouvrement dans la région de Sarnia et ce phénomène a été observé pendant les investigations par forage dans la coupe d'approche de Sarnia. L'inclusion des effets de la présence des gaz naturels ex-résolus dans les sols granuleux fins soumis au déchargement dans le FEA a donné des prévisions sensiblement meilleures de la tendance des déformations sur les pentes de la coupe d'approche.

Mots-clés : gaz naturels, excavation, déformations de la pente, plaine d'argile de St. Clair, analyse par éléments finis.

Introduction

The St. Clair Tunnel project consists of two railway tunnels, approximately 1800 m in length running parallel to each other beneath the St. Clair River from Sarnia, Ontario, Canada, to Port Huron, Michigan, USA (see Fig. 1). The first tunnel was constructed between 1889 and 1891. During this time, the approach cuts to the tunnel portals were constructed by trial and error and numerous slope failures occurred. The second tunnel was constructed — for CN North America — over 100 years later, from 1993 to 1995. The new tunnel is located immediately to the north of the origi-

nal and the new portal structures and track alignment make use of the existing approach cuts at each end of the old project. Owing to the size of the new tunnel and location of the new alignment, additional excavation in the form of widening and deepening of the north side of the approach cuts was required. During the new construction, deep, unexpected deformations occurred on the south slope of the Sarnia approach cut in response to the new excavation. The factors contributing to the slope deformations observed in the Sarnia approach cut in 1993 are the focus of this paper.

Background

The approach cut to the St. Clair Tunnel investigated here is located in Sarnia, Ontario, on the east side of the St. Clair River. In January 1889, a 60 m wide excavation into previously flat farmland (ground surface at elevation 182.5 m) was started with a side slope profile of 1.5(horizontal(h)):1(vertical(v)). After 7 months of construction, the excavation was nearing the required depth of 15 m when failures occurred consecutively on two slopes (west and south) of the approach (end of first stage). Over the next two months, the slopes of the approach were flattened to a profile of 2(h):1(v), allowing successful completion of the first stage to a depth of 15 m. Between September 1889 and January 1891 the cut remained stable as the original tunnel was constructed and the approach was lengthened, but not deepened. In the

Received 26 June 2008. Accepted 21 January 2010. Published on the NRC Research Press Web site at cgj.nrc.ca on 27 August 2010.

J.P. Dittrich.¹ Golder Associates Ltd., 2390 Argentia Road, Mississauga, ON L5N 5Z7, Canada.

R.K. Rowe. Queen's University, Kingston, ON K7L 3N6, Canada.

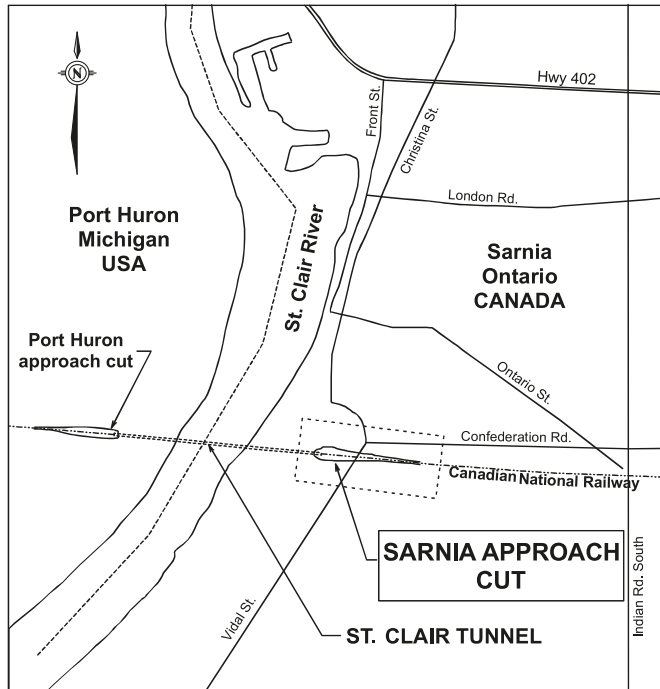
D.E. Becker.² Golder Associates Ltd., 1000, 940 - 6th Avenue SW, Calgary, AB T2P 3T1, Canada.

K.Y. Lo. The University of Western Ontario, London, ON N6A 5B9, Canada.

¹Corresponding author (e-mail: pdittrich@golder.com).

²Present address: Golder Associates Ltd., 102, 2535 - 3rd Avenue E, Calgary, AB T2A 7W5, Canada.

Fig. 1. Site location map – Sarnia approach cut.



following month, the cut was deepened by removing the toe of the north slope to a depth of 17 m to accommodate construction of a masonry retaining wall at the base. This triggered the failure of the north slope of the approach (end of second stage). Figure 2 shows the early stages of excavation in the cut. Additional details of the history of the construction at the site between 1889 and 1993 have been discussed previously (Becker et al. 1996; Dittrich et al. 1997, 2000) and full details are provided in Dittrich (2000). The events surrounding the 1993 construction at the site have been discussed in Dittrich et al. (2002) and additional details of the soil parameter assessment, analyses, and results are described in this paper.

At the completion of construction of the original tunnel in 1891, the approach cut in Sarnia was approximately 17 m deep with a masonry retaining wall at its base and overall side slope profiles of about 3.5(h):1(v) in benches. The slopes in the approach maintained this geometry until 1993, when construction for the new tunnel project started. The new tunnel was designed with a diameter 1.5 times that of the original and the new alignment was located about 15–25 m north of the existing. Geologic restrictions at the project site imposed by the elevation of the riverbed and bedrock surface required that the invert of the new tunnel lie almost 2 m below that of the existing tunnel. These requirements necessitated a widening and deepening of the north side of the existing approach cuts, as shown in Fig. 2.

The effects of this would be most severe in the Sarnia cut due to its greater depth in the portal area. Physical constraints at the site imposed by the allowable distances to the north property line – site boundary would not permit flattening the north slope to an amount required to attain the target factor of safety of 1.3. To accommodate the required geometry and achieve the target factor of safety, the north slope was cut back as much as possible and a modular concrete

diaphragm wall system was constructed at the toe of the north slope over the deepest section of the Sarnia cut (see Figs. 2 and 3). The diaphragm wall extended 150 m eastward from the portal of the new tunnel and was constructed in a series of panels, each approximately 5 m long, 1 m wide, and 9 m deep.

Response of slopes to 1993 construction

The first stage of construction involved excavating back the north slope of the approach to make room for the track alignment and new portal. During the excavation, the north slope performed satisfactorily with horizontal movements of less than 5 mm measured in the slope inclinometers. Interestingly, the inclinometers located on the upper parts of the north slope showed the expected down-slope movements; however, the inclinometers located near the bottom of the north slope showed up-slope movement in response to the unloading. On the opposite, untouched south slope, total down-slope deformations of about 10 to 15 mm in the direction of the unloading were measured in the inclinometers during this time.

Following completion of the excavation on the north slope, excavation and construction of the panels for the diaphragm walls at the base of the cut were carried out. In response to the panel construction, the inclinometers on the south slope showed deep, concentrated zones of movement developing at a depth of about 20 to 25 m below the ground surface (about 15 m below the base of the cut). Above this zone, no additional horizontal movements were measured; the behaviour appeared to be essentially that of a rigid block moving along a slip surface. In addition, a tension crack appeared about 10 m behind the crest of the south slope. By the end of construction of the panels, approximately 60 mm of total horizontal movements had been measured in the inclinometers on the south slope. In contrast, the north slope continued to perform adequately during this same period of construction. Total down-slope movements on the north slope by the end of construction of the panels were on average less than 10 mm and did not exceed a maximum of 20 mm.

Figure 3 shows a plan view of the Sarnia site highlighting the old and new tunnel alignments, the extent of the 1993 excavation and construction, the overall direction of slope deformation, and the location of a cross section employed to analyse the slopes. Figure 4 shows a section view of the south slope and includes profiles of horizontal, down-slope deformation measured in two slope indicator installations during construction in 1993.

Stratigraphy and problem geometry

The stratigraphy of the overburden at the site of the Sarnia approach consists of approximately 36.5 m of firm to stiff, silty clay till (part of the St. Clair Clay Plain) underlain by a relatively thin, very dense, silt and sand basal till aquifer. A stratigraphic profile through the approach cut is shown in Fig. 5. The silty clay till stratum has a heavily overconsolidated crust underlain by a lightly overconsolidated stratum becoming normally consolidated with depth. Details of the constitutive parameter assessment of the soils

Fig. 2. Cross section of stages of excavation at Sarnia approach cut.

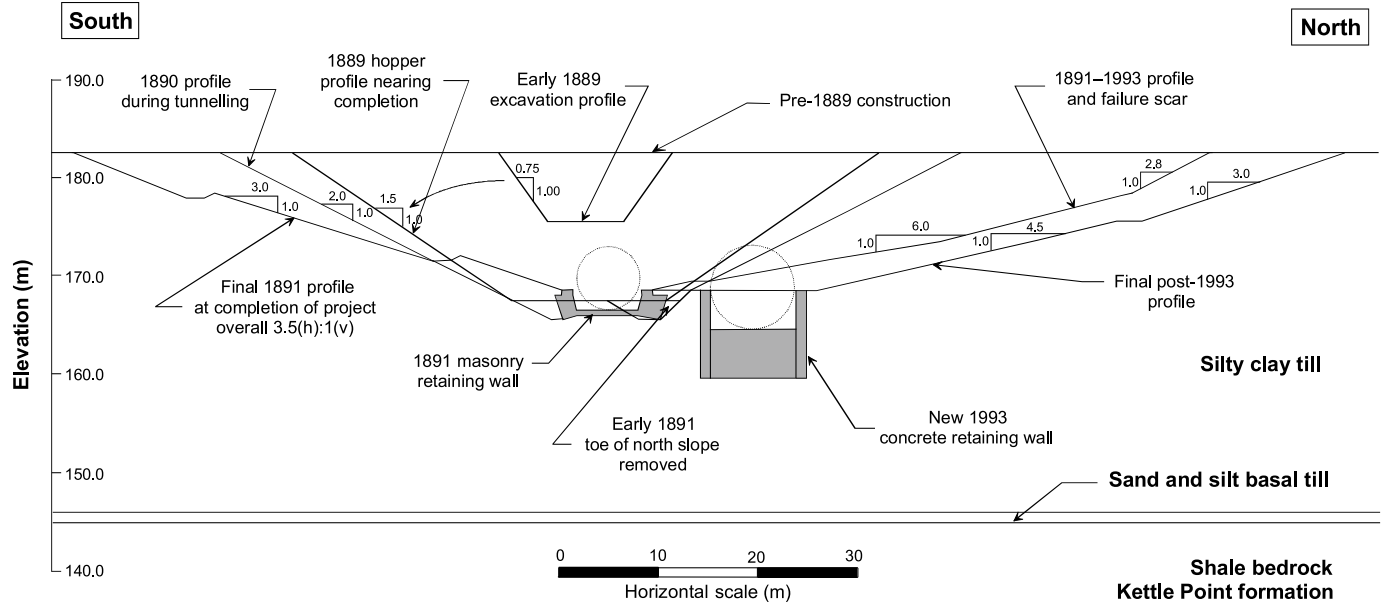


Fig. 3. Plan view of the Sarnia cut showing direction of slope movement and location of section for analysis.

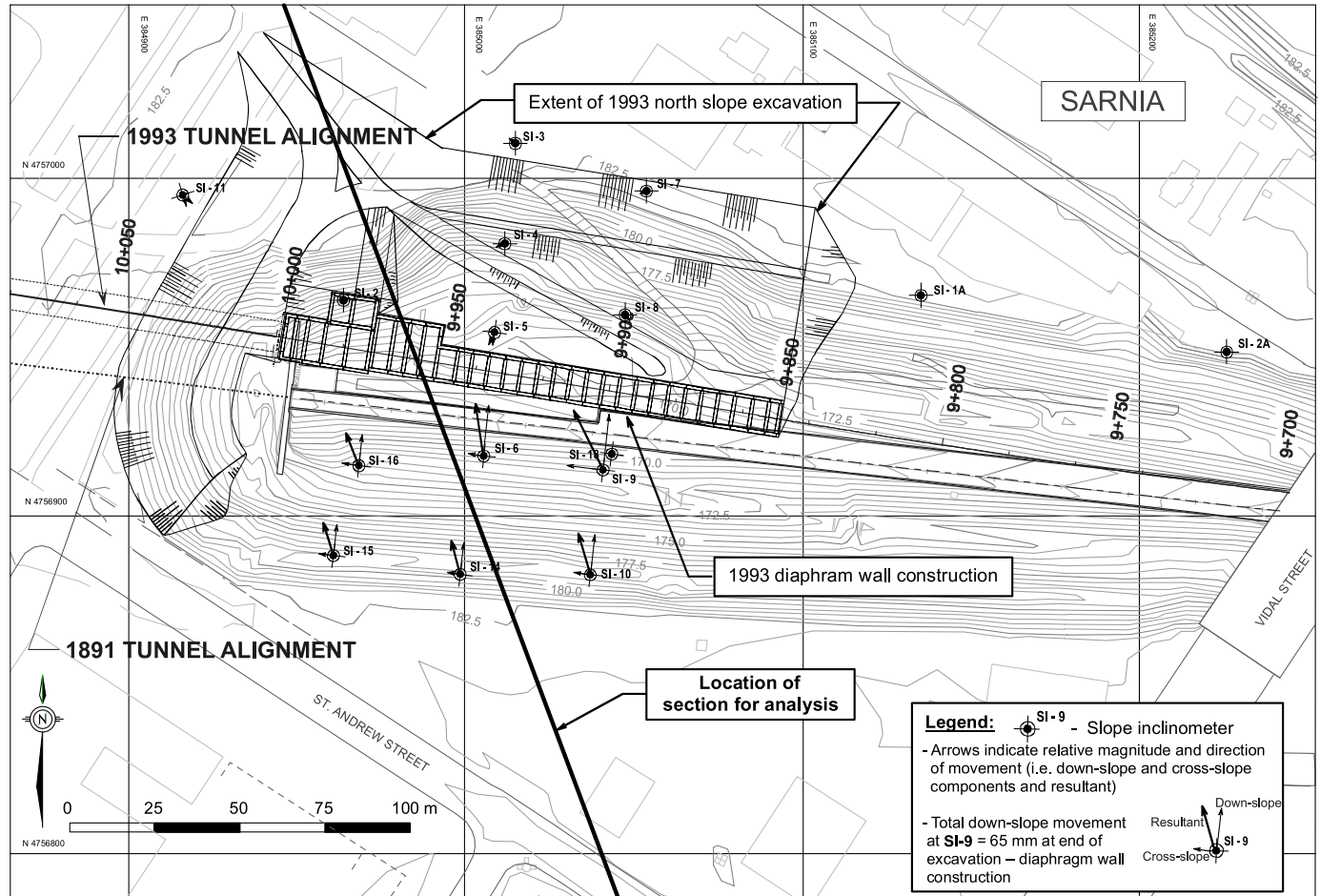


Fig. 4. Location of slip surface on south slope of Sarnia approach (based on slope inclinometer data). SI, slope inclinometer.

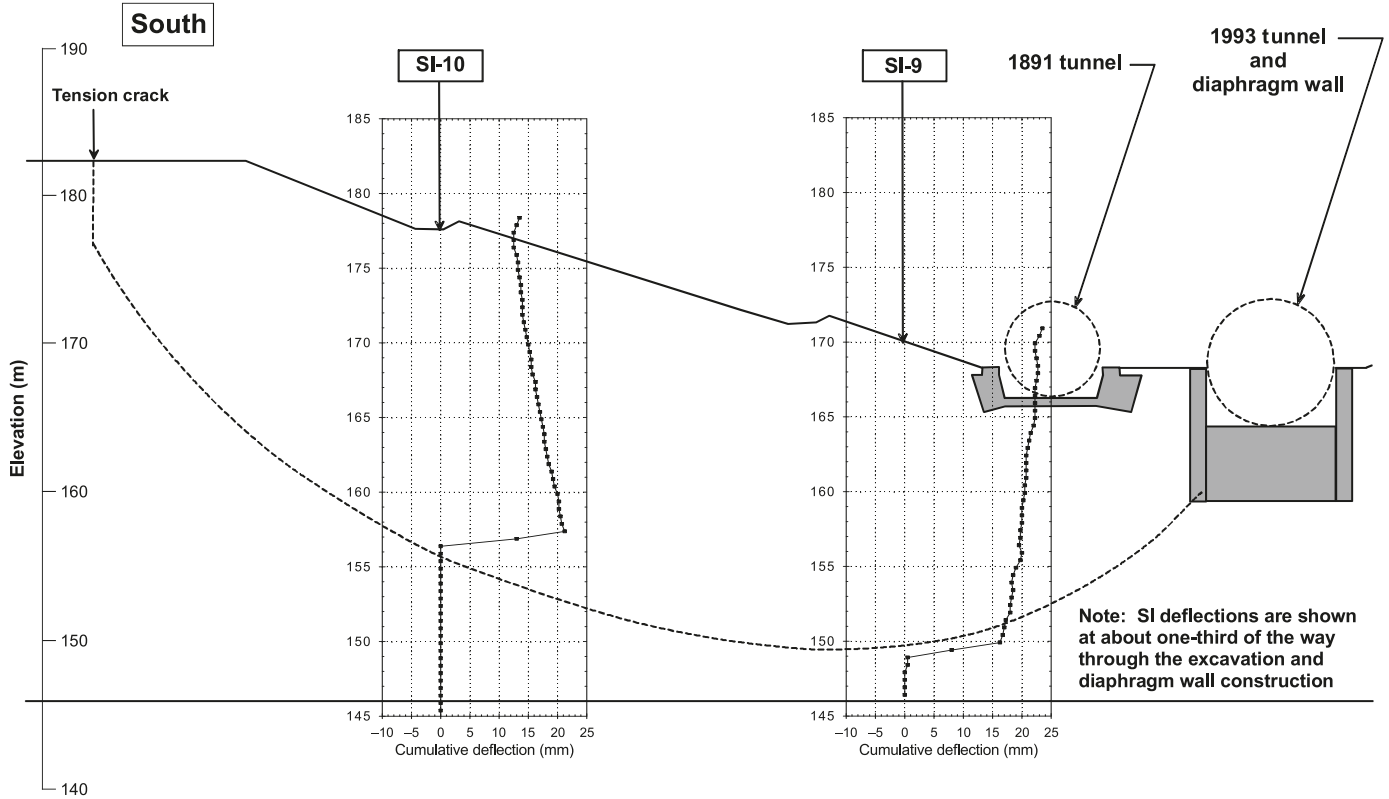


Fig. 5. Stratigraphic profile and approximate location of section used in analysis.

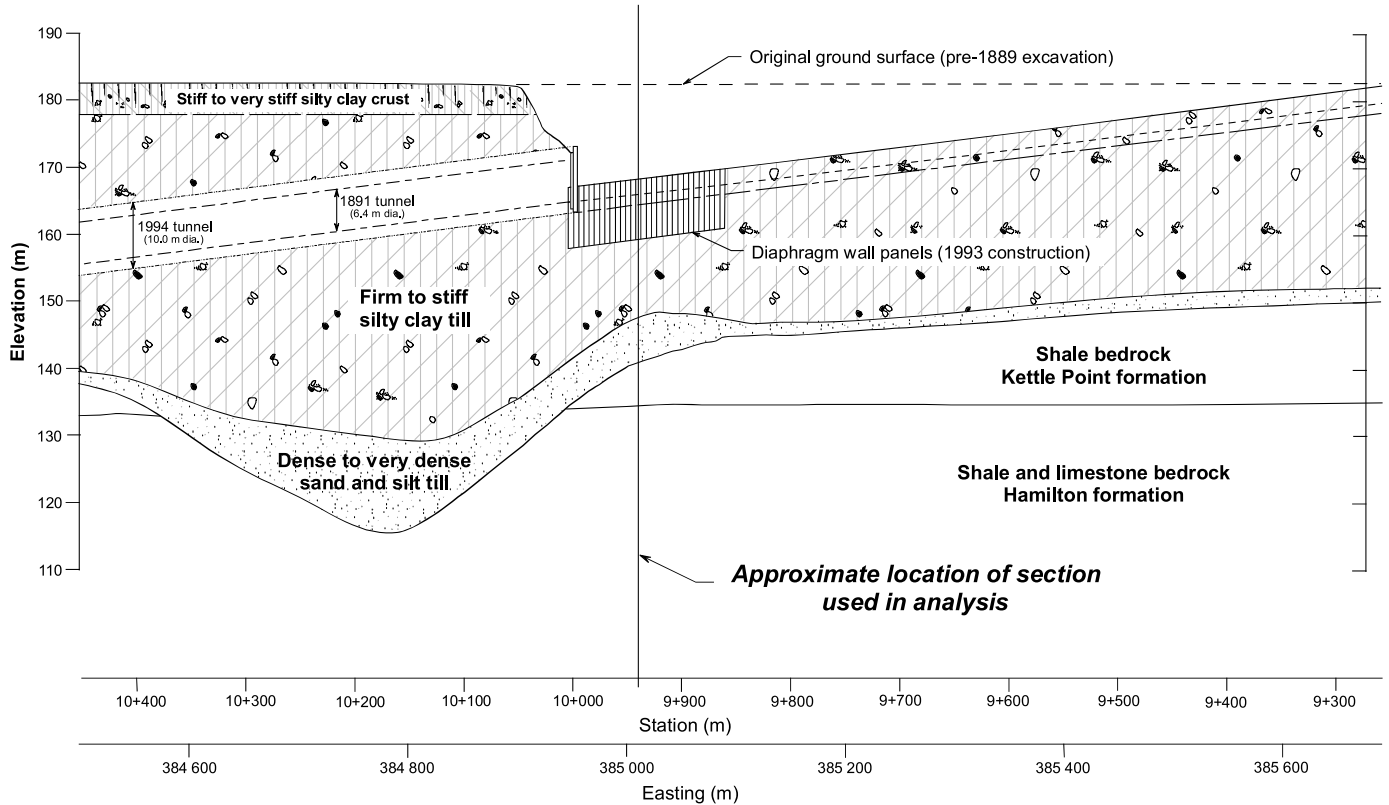


Fig. 6. Schematic of cross section selected for analysis. GWT, groundwater table.

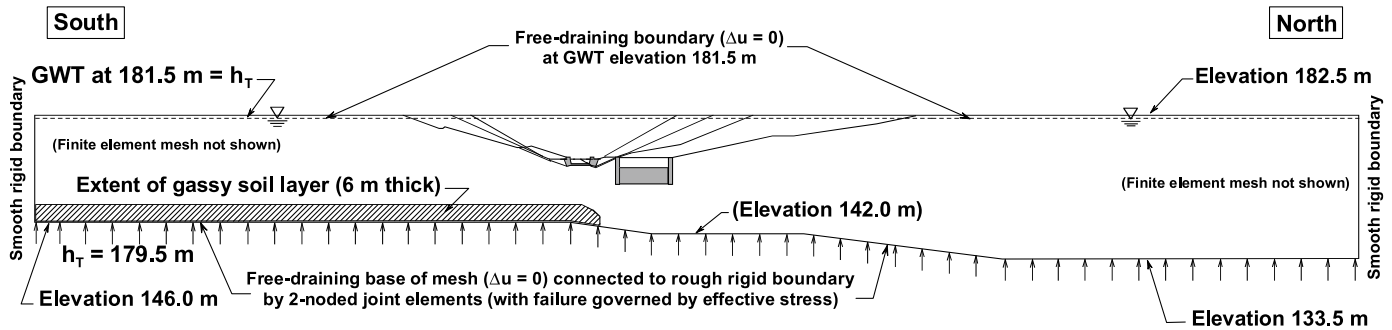
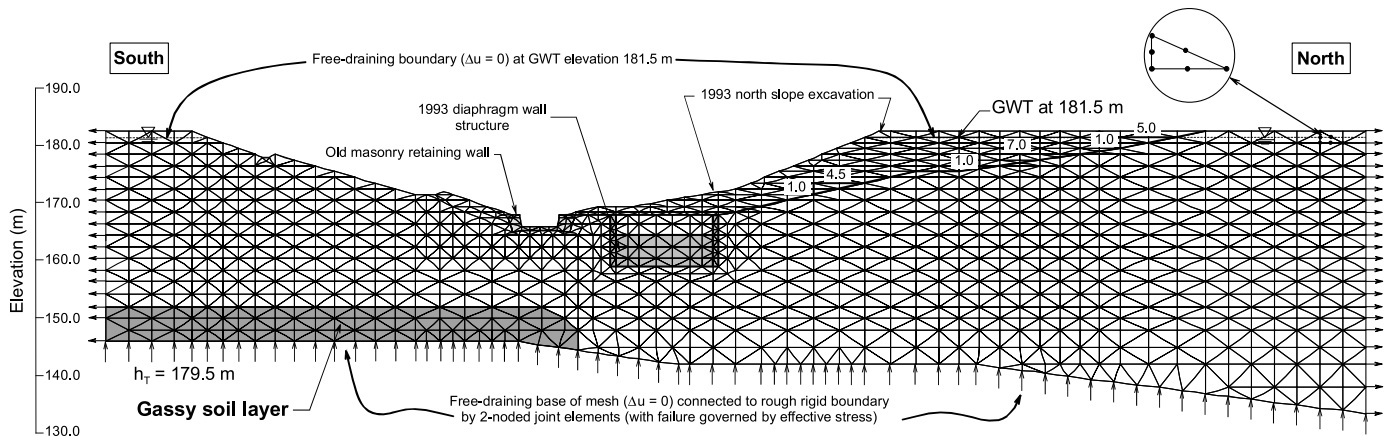


Fig. 7. Finite element mesh and geometry of excavation and 1993 construction at Sarnia.



at the site can be found in Dittrich et al. (2000) and Dittrich (2000).

Figure 6 shows a schematic of the critical cross section selected for analysis. The alignment of the section is consistent with the overall direction of resultant slope movement as measured in the slope inclinometer installations shown in Fig. 3. The significance of the gassy soil zone shown in Fig. 6 as well as the rationale for its location and extent is discussed in the section titled “Exsolved natural gases in Sarnia area”. The details of the finite element mesh used to analyse the movements of the silty clay till stratum are shown in Fig. 7 together with the geometry of the construction conducted in 1993. The lateral extent of the full mesh has not been shown. It extended about 185 m to the south and 265 m to the north of the centreline of the original track alignment. The extent of the mesh was selected to minimize the effect of the far field boundary conditions and was based on recommendations by Kulhawy (1977). The full mesh consisted of 4508 six-noded, isoparametric, linear strain triangular elements with displacements and pore pressures calculated at all nodes. The far field lateral boundaries of the mesh were specified to be smooth and rigid while the bottom of the finite element mesh was connected by 205 two-noded joint elements to a rough rigid boundary. Free-draining pore pressure boundaries were assumed along the top of the basal till aquifer as well as along the nodes located at elevation 181.5 m, representing the surface of the groundwater table in the stratum. This is based on the presence of fissures that increase the hydraulic conductivity in the crustal soils and on measurements of an overall downward hy-

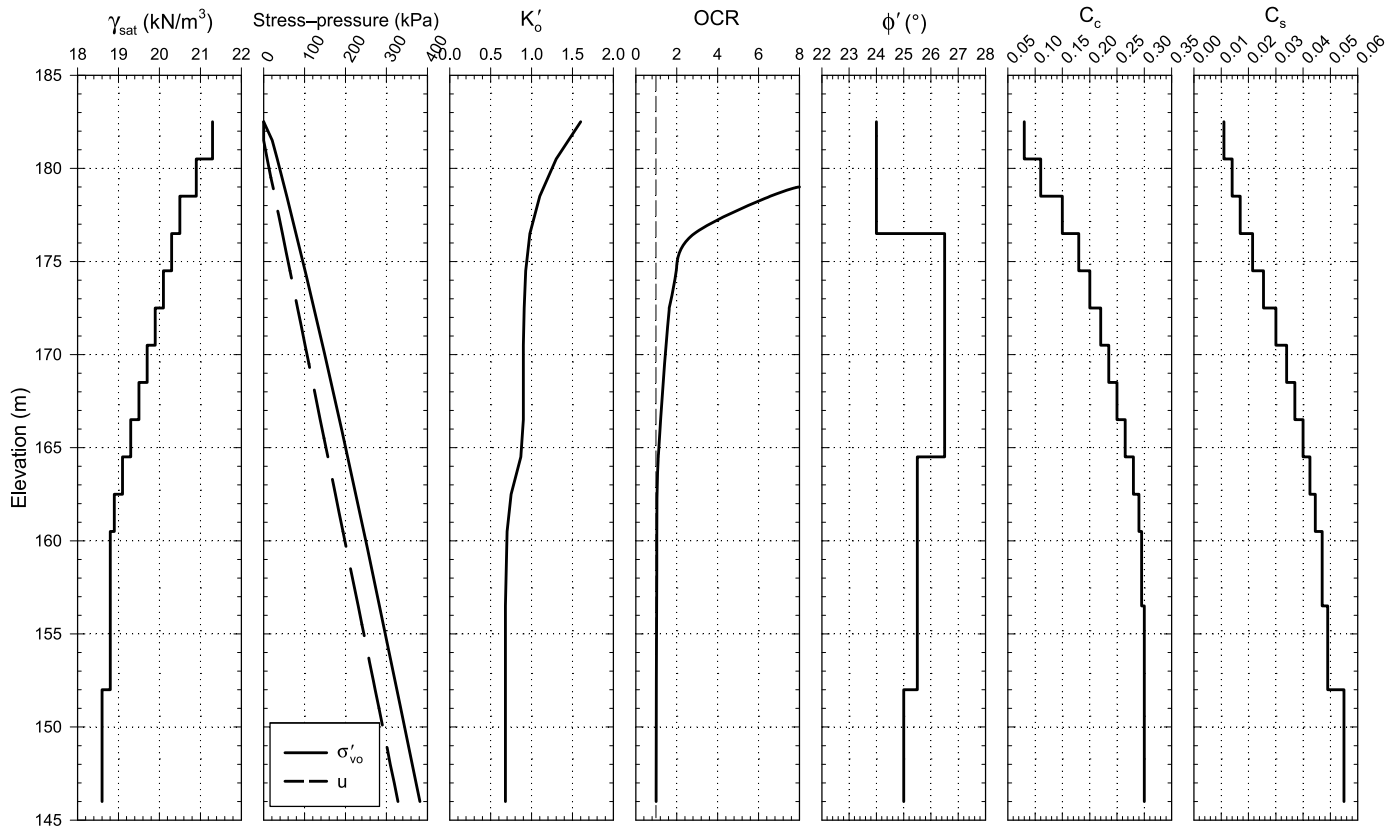
draulic gradient in the silty clay till in the Sarnia area. The exposed surfaces of the slopes including those following excavation were treated as no-flow boundaries. Excavation of the north slope and construction of the diaphragm walls was simulated in 1.0 m thick lift increments.

Numerical model and parameters

The program AFENA (Carter and Balaam 1990), modified to include an effective stress, elliptical cap soil model (Chen and Mizuno 1990) coupled with Biot consolidation theory (Biot 1941), was employed for the finite element analysis (FEA) of the excavation and construction.

An extensive review and re-assessment of existing data on the properties of the St. Clair till at the site combined with additional laboratory testing was conducted to provide the best estimate and likely range of key parameters required for the analysis. This included using both field and laboratory data acquired by Golder Associates Ltd. in 1992 and 1993 (Golder Associates Ltd. 1992, 1994) for the design of the new St. Clair Tunnel, testing on specimens collected at the site in 1995 for this study, as well as information found in the literature. The justification for the choice of constitutive model and verification of the selection of constitutive parameters was based on extensive modelling of the early stages of construction, including sensitivity analyses to examine the influence of the range in some parameters. The results presented by Dittrich (2000) and Dittrich et al. (2000) demonstrate that the chosen finite element model and input parameters have the ability to accurately predict

Fig. 8. Parameter summary for finite element analysis.



the early stages of slope failure in the Sarnia cut in both 1889 and 1891. Figure 8 shows profiles of the key parameters for the St. Clair till at the site as required for the effective stress FEA. A detailed discussion pertaining to the estimation of each parameter is beyond the scope of this paper; however, a summary is provided in the following sections.

Coefficient of earth pressure at rest (K'_0)

The profile of the coefficient of earth pressure at rest, K'_0 , was assessed using correlations with the results from both the dilatometer test (DMT) and cone penetration test (CPT) conducted at the site, as well as laboratory results from incremental, zero lateral strain, triaxial consolidation, and one-dimensional consolidation tests on both horizontal trim orientation (HTO) and vertical trim orientation (VTO) specimens (Becker et al. 1987). In addition, values found in the literature (from hydraulic fracturing tests) and assessments made based on empirical correlations were used. Only the final best estimate profile is shown. K'_0 is difficult to quantify, however, and it was found that estimates even within the normally consolidated portion of the silty clay till stratum could range from a low of about $K'_{0\ n/c} = 0.6$ (Jaky 1944) to a high of about $K'_{0\ n/c} = 0.8$ (Masood and Mitchell 1993). In light of this, the results of the sensitivity analysis of the early stages of excavation and the ability to predict the initial slope failures justified the best estimate profile shown in Fig. 8.

Overconsolidation ratio (OCR)

The overconsolidation ratio (OCR) profile is based on the results of one-dimensional consolidation tests performed on samples taken from various elevations within the silty clay till stratum as well as values found in the literature. Additionally, correlations of OCR or σ'_{vp} with either the value of the tip stress, q_c , or pore pressure, u_{max} , measured in the CPTs were also used (Mayne and Holtz 1988). Only the final best estimate profile is presented, showing that the silty clay till stratum at the site has a heavily overconsolidated crust underlain by a lightly overconsolidated stratum becoming normally consolidated with depth. However, some scatter exists in the data especially within the upper lightly overconsolidated portion of the stratum and as such, the results of the sensitivity analysis of the early stages of excavation and the ability to predict the initial slope failures were again used to justify the best estimate profile shown in Fig. 8.

Effective friction angle (ϕ')

The profile of ϕ' was based on the results of isotropic consolidated undrained (CIU) and anisotropic consolidated undrained (CAU) triaxial compression tests with pore pressure measurements performed on specimens of the silty clay till sampled from the site. The value of ϕ' within the upper desiccated and weathered crustal portion of the stratum was reduced to account for the influence of the presence of fissuring on the operative strength in this region.

Compression index (C_c) and recompression–swelling index (C_s)

The profiles of both C_c and C_s were based on the results of one-dimensional consolidation tests performed on samples taken from various elevations within the silty clay till stratum as discussed above. Additionally, comparisons made with empirical correlations found in the literature (Terzaghi and Peck 1967; Kulhawy and Mayne 1990) showed good agreement with the profiles shown.

Other parameters

In addition to the profiles shown in Fig. 8, the following parameters were also used in the analysis. Poisson's ratio, ν' , was assumed to be equal to 0.4 based on the results of an isotropic consolidated drained (CID) triaxial extension test performed on a specimen of the till and based on similar values used for the FEA of excavations found in the literature. The hydraulic conductivity, k_v , of the intact silty clay till stratum was estimated to be 2.2×10^{-10} m/s from the results of flexible wall permeameter tests, one-dimensional oedometer tests, and the literature values. However, it was found that the value of k_v could range from a low of 1.0×10^{-10} m/s to a high of 5.0×10^{-10} m/s. Sensitivity analyses were performed to assess the impact of this range of values. Researchers (D'Astous et al. 1989) have shown that the hydraulic conductivity could be as much as two orders of magnitude higher within the upper 4 to 6 m of the fissured crustal zone of the till. The effect of the value of $k_{v(\text{crust})}$ on the behaviour of the excavated slopes was also investigated. A ratio of $k_h:k_v = 2$ was adopted based on flexible wall permeameter tests performed on horizontally and vertically trimmed specimens. This was similar to the values found in the literature. Finally, the value of the aspect ratio, R_c^* , for the "elliptical cap" constitutive model (Chen and Mizuno 1990) was taken to be $R_c^* = 2$ based on a yield surface study of data from triaxial tests. This value was consistent with the literature values based on the ratio of s_v/p' for the till (Humphrey and Holtz 1988). Finally, for all analyses, the failure criterion match between the smooth Drucker–Prager cone in the "elliptical cap" model to the irregular hexagon of the Mohr–Coulomb model (Naylor 1978; Griffiths 1986) was based on an extension side match. This extension match was adopted based on both the results of the yield surface study on the till and general conclusions in the literature that, in excavation analysis, the passive soil response or the unloading properties of the soil control behaviour.

Exsolved natural gases in Sarnia area

Southwestern Ontario, and specifically Lambton County in which the town of Sarnia is located, has been an oil- and gas-producing area since the 1800s. In fact, construction of North America's first recorded oil well was completed at Oil Springs (located about 30 km from Sarnia) in 1858. Although the area is no longer developed for commercial production, naturally occurring gases (mostly methane and hydrogen sulphide) are still frequently encountered in the Lambton area when drilling near the base of the overburden and into the underlying bedrock. The phenomenon has been described by Weaver (1994) and has been documented by

Intera Technologies (1992). At the Sarnia approach cut, the venting of natural gas from the soil pore space during borehole drilling was noted by Golder Associates in 1992 and 1993 (Golder Associates Ltd. 1992, 1994), during the field work for this research in 1995 (Dittrich 2000), and by others. Gas venting typically manifests itself in the form of a violent ejection of the drill fluid from the top of augers or drill casing as the borehole approaches the lower portion of the silty clay till stratum (i.e., up to as much as about 1.5 m above the interface of the silty clay till – sand and silt till). At some locations, the gas venting was described in Golder Associates Ltd. (1994) and Dittrich (2000) to be "at a high pressure" for several hours (1 to 3 h) before decreasing to a much reduced pressure lasting up to several days (as many as 3 days).

Effect of gases on soil behaviour

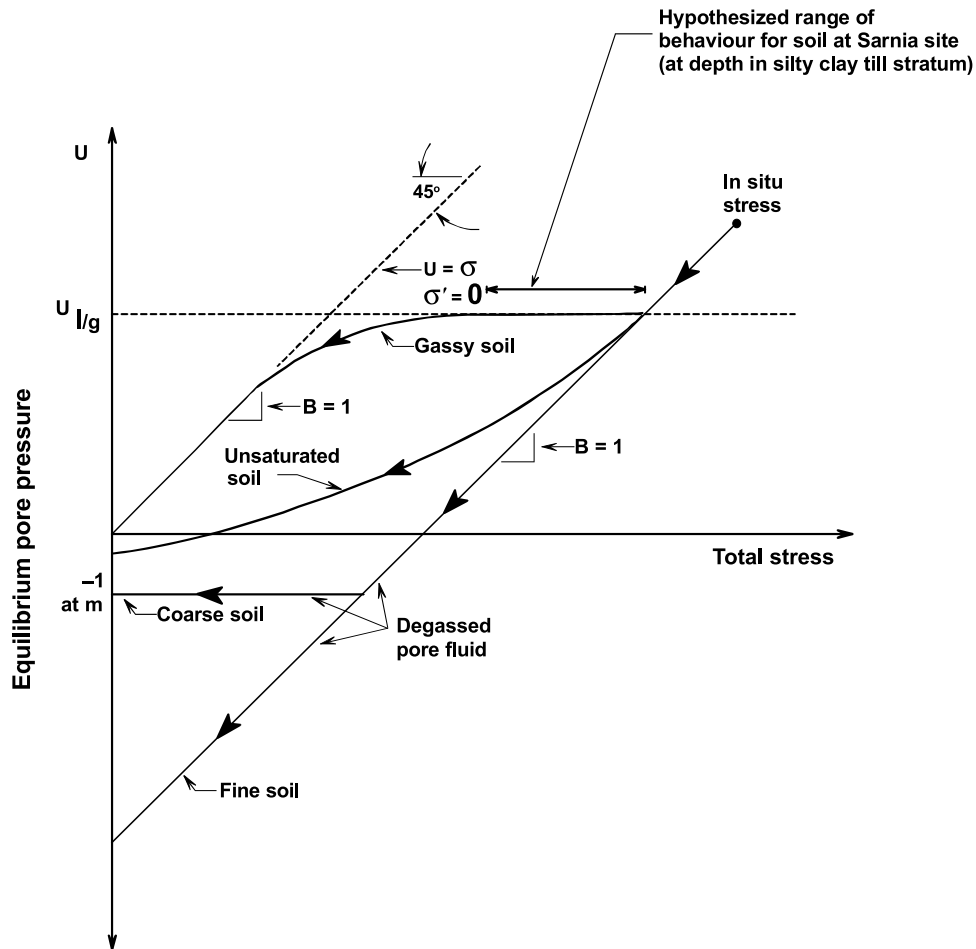
The effect of the presence of natural gases on the geotechnical properties of soils has been investigated by a number of researchers and a good summary of the work done to date is found in Grozic et al. (1999).

The effect of unloading-type stress paths on the behaviour of gassy soils has been investigated in detail (including laboratory studies) by Sobkowicz and Morgenstern (1984). They showed that when unloading a gassy, fine grained soil, the expected generation of negative excess pore pressure, Δu , and the accompanying short-term decrease in u_T leading to the condition that $\Delta\sigma' \approx 0$, will not necessarily occur. In fact, for the condition where an exsolved gas phase is already present in a soil's pore space before unloading begins (or for the condition where gas comes out of solution in response to the unloading), Sobkowicz and Morgenstern (1984) have shown that $\Delta u = 0$ during most of the unloading stage. This response will continue until a limiting condition of $u_T = \sigma_T$ is reached after which the pore pressures decrease to maintain the condition of $\sigma' = 0$ (imminent rupture of soil structure due to gas pressure) for further decreases in the total stress.

During unloading, if $\Delta u = 0$ and σ_T is decreasing, then σ' will decrease. If the effective stresses in a soil are decreasing, then the soil is weakening, and if the shear stresses in the soil are increasing at the same time, then the likelihood of failure occurring will be high. The differences in soil behaviour in terms of pore pressure response during unloading, for saturated coarse-grained and fine-grained soils, unsaturated soils, and gassy soils are illustrated in Fig. 9. Sobkowicz and Morgenstern (1984) reason that the condition that $\Delta u = 0$ during unloading of a gassy soil is a result of the presence of the gas phase in the soil pore space reducing the bulk modulus or stiffness of the soil-water system. They explain that the gas fraction in the soil can reduce the stiffness of the soil–water system to such a degree that external stress changes are absorbed by the "soft" system instead of being directly transferred to the pore water. This results in very little development of excess pore pressure, Δu . Brandes (1999) has also shown that the presence of gas bubbles, even in small amounts, (ie. at gas-bubble concentrations of less than 1%), will cause a large reduction ($\sim 95\%$ decrease) in the bulk modulus of a soil.

Numerous researchers have investigated the potential effects that the presence of gas can have on the geotechnical

Fig. 9. Undrained equilibrium behaviour of an element of soil on unloading: effect of gas in pore fluid (adapted from Sobkowicz and Morgenstern 1984).



behaviour of soils. However, the majority of this work has been limited to either theoretical speculation of the response of gassy soils or to measured and observed behaviour of laboratory-prepared specimens. There is a paucity of published work in which a case study demonstrates the effect that the presence of a gas-permeated soil stratum can have on overall soil behaviour in response to excavation. This Sarnia case study provides a unique opportunity to examine this mechanism.

Modelling effect of gases in FEA

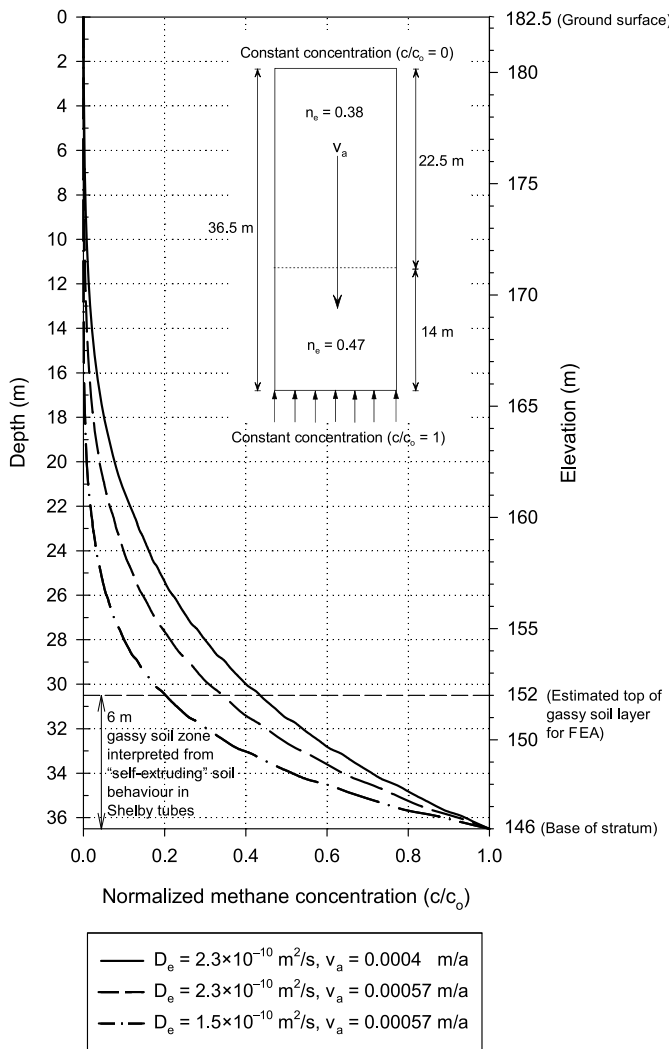
The effect that the presence of exsolved gases in the pore space has on the geotechnical behaviour of the soil, or similarly, the condition that the excess pore pressure, Δu , remains equal to zero in the lower layers of the silty clay till stratum, can be achieved several different ways in the FEA. Modelling approaches could range from the development of a rigorous constitutive model that captures the interaction and behaviour of the soil, pore-water, and pore-gas system, to assigning an artificially low bulk modulus to the soil elements defined in the gassy soil zone or to assigning an artificially high value of hydraulic conductivity, k_v , to the elements defined in the gassy soil zone of the finite element mesh. The approach adopted for this research was to select a high enough value of hydraulic conductivity that the excess

pore pressure would remain at or close to zero within the zone of gassy soil elements throughout the FEA. After several iterations of analysis during which a range of different values of k_v were utilized, it was found that specifying the value of k_v to be three orders of magnitude higher than the actual value for the silty clay till was sufficient to satisfy the requirement of maintaining zero excess pore pressure within the gassy soil zone for the boundary conditions at this site.

Upon further examination of Fig. 9, it can be seen that, in addition to the condition of zero excess pore pressure development within the gassy soil zone upon unloading, two other ranges of behaviour are possible and need to be considered, as discussed below.

First, the pore pressure response, $\Delta u = 0$, can only occur if the gas is exsolved (or exsolves) in the pore space upon or during the unloading (i.e., the pore pressure is less than or equal to the liquid-gas saturation pressure, u_{lg}). At the Sarnia site, it is postulated that the gas present in the pore space is mostly exsolved (i.e., already out of solution). As discussed in the next section, it is understood that natural gas is produced at zones deep in the underlying bedrock (more than 1000 m below ground surface). As described by Kuo (1997), from these depths the gas may migrate upwards as a separate phase and (or) as a solution gas dissolved in the formation waters. However, as the gassy fluid migrates up-

Fig. 10. POLLUTE estimate of extent of dissolved methane migration through porewater of St. Clair till at 13 500 years.



wards through the fractures in the bedrock, the decrease in pore pressure at higher elevations will likely cause gases (or additional gases) to exsolve from the groundwater. Further, even if the gas remained in solution after migrating to the base of the silty clay till (and further up into the lower portions of the clay stratum), the original excavation at the Sarnia site (between 1889 and 1891) would have caused a large reduction in total stress, a consequent decrease in pore pressures (if the gas were still dissolved), and caused gas (or additional gas) to come out of solution at this point.

Second, once $u_T = \sigma_T$ or $\sigma' = 0$, the condition that Δu remain equal to zero is no longer valid. For reductions in total stress beyond this point, Sobkowicz and Morgenstern (1984) suggest that the pore pressures will once again begin to decrease. To check whether the condition $\sigma' = 0$ would be reached within the gassy soil zone during the excavation at the Sarnia cut, a fully coupled, effective stress FEA was performed, modelling the original stages of excavation. In the analysis, the lower 6 m of the silty clay till stratum (to be discussed in the next section) was assigned an artificially high value of k_v to ensure that Δu remained equal to zero

at all times in the gassy soil layer. The results of the analysis show that the lowest value of effective stress occurs below the base of the deepest section of the cut and at the very top of the gassy soil layer and is on the order of approximately 80 kPa (much greater than $\sigma' = 0$).

Based on the above discussions and findings, it appears that in the coupled FEA, controlling the development of excess pore pressures (i.e., such that $\Delta u = 0$) in the gassy soil zone is all that is required to effectively model the gassy soil behaviour at the Sarnia site.

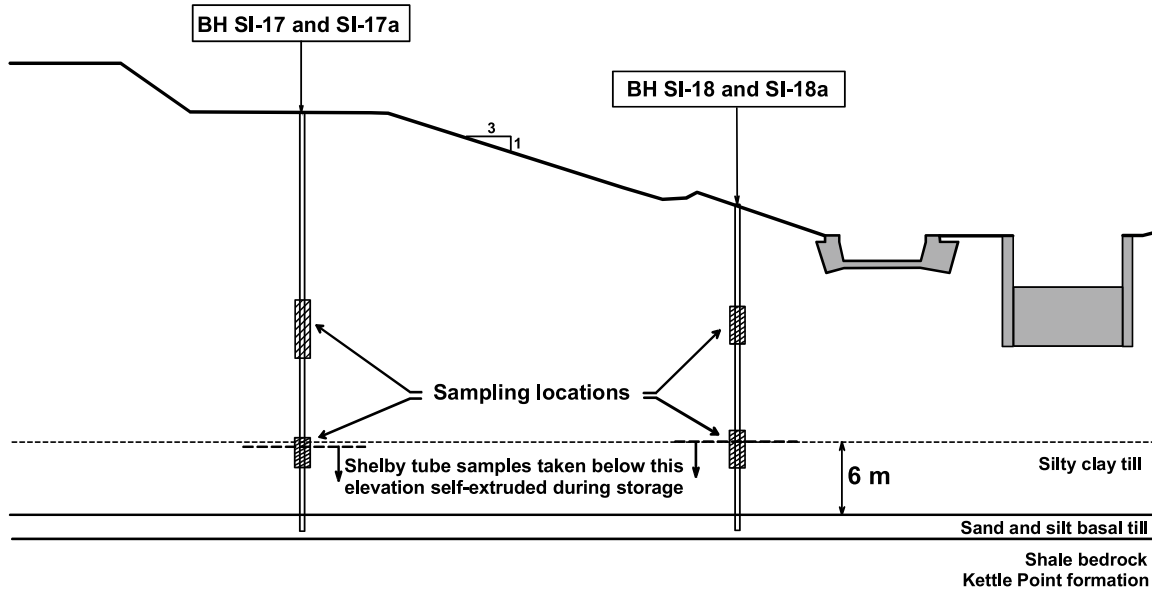
Extent of gas migration in St. Clair till at Sarnia

The natural gases in the Sarnia area are petrogenic, generated deep within the bedrock strata at depths of ~1200 to 1500 m below ground surface (Sanford et al. 1985) and produced from the Middle Ordovician carbonates of the Trenton and Black River groups (Carter et al. 1996). As natural gas is less dense than water, it tends to rise upward through fractures in the bedrock and collect in the highest elevation areas of high permeability that are bounded or capped by areas of much lower permeability. At the Sarnia site, such a geologic configuration exists as shown in Fig. 5 at the interface of the sand and silt basal till aquifer (high permeability) with the overlying silty clay till stratum (low permeability) and explains why gases are usually encountered close to this interface. The rationale that the gases will collect at the highest elevation of this interface is the basis for the location of the gassy soil zone as defined in the finite element model as shown in Figs. 6 and 7.

The silty clay till stratum at the site has low hydraulic conductivity ($\sim 2.2 \times 10^{-10}$ m/s) and performs adequately as a “cap” allowing the gases to collect beneath it. However, it is not impermeable and over long periods of time the gas (either exsolved or dissolved in the groundwater) would diffuse up into the pore space. Because of the direction of migration, it would be expected that the deepest layers of the silty clay till stratum would be the most permeated by the gas, with its presence decreasing with increasing elevation.

The elevation of the upper extent of the gassy soil zone at the Sarnia site has been assessed using two approaches: one theoretical and one practical. The theoretical approach involved using the finite layer program POLLUTE (Rowe and Booker 2005) to estimate the concentration of methane that could exist in the soil pore water 13 500 years after the stratum’s formation. A time frame of 13 500 years was selected for the analysis as this corresponds approximately to the time when many of the major soft clay deposits in the northern USA and southern Canada (including southwestern Ontario) were deposited during the retreat of the Wisconsin ice sheet during the last ice age (Quigley 1980). POLLUTE is capable of modelling one-dimensional advective diffusive transport of either dissolved or exsolved gas (or other molecules) through porous media. The problem geometry was defined as a one-dimensional 36.5 m thick soil layer (based on the average dimensions of the stratum at the Sarnia site) and the details of the analysis and input parameters are described in Dittrich (2000). The estimated profiles of the extent of dissolved methane migration into the pore space (Fig. 10) show that the higher concentrations are predicted to extend primarily over the lower 4 to 8 m of the St. Clair till stratum.

Fig. 11. Location of self-extruding Shelby tube samples (in section).



The practical approach involved careful observation and examination of the behaviour of the silty clay till specimens sampled in Shelby tubes during the 1995 investigation at the site as part of the research by Dittrich (2000). During the field work, a total of 22 Shelby tubes were obtained from four boreholes drilled into the south slope of the approach. In each of the boreholes, the tube sampling was done continuously between two different sets of elevations (an upper set between about an elevation of 164 and 159 m and a lower set between about an elevation of 153 and 150 m). Immediately after removal from the boreholes, all tubes were sealed at both ends using a heated and liquified microcrystalline (nonshrinking) wax that formed a plug at least 1 cm thick upon cooling. All tubes were stored in coolers with ice and taken each day to a laboratory where they were stored in a temperature-controlled room (at 10 °C). During a 2 week storage period, the wax seals in five of the 22 Shelby tubes broke as a result of expansion of the soil specimens' length by about 2 to 6 mm. The occurrence can most likely be explained by bubble-cavity expansion within the pore space of the soil due to the unloading caused by sampling. This phenomenon only occurred in Shelby tubes sampled as part of the lower sample set, and only seals of the tubes sampled below elevation 152 m (i.e., the lower 6 m of the silty clay till stratum as shown in Fig. 11) broke.

Based on the consistency between these two approaches, the gassy soil layer is specified to extend 6 m above the base of the silty clay till stratum in the FEA. However, for the cross section employed and the results presented herein, the gassy soil layer is specified to only exist beneath the south slope of the cut (as shown in Figs. 6 and 7). This approach was adopted because the majority of the gassy soil zones encountered during drilling investigations in the approach were located at the higher elevations of the contact between the sand and silt basal till aquifer and the overlying silty clay till "cap." This is as would be expected because, as the natural gas migrates upwards, it should tend to naturally accumulate below the highest elevations of the basal

till contact. For the section chosen for analysis, the highest elevation of the basal till contact is below the south slope.

Finite element results and discussion

Using the cross section shown in Fig. 6, the complete history of the excavation and construction within the Sarnia approach cut between 1889 and 1891 was initially modelled in the FEA. Once completed, the analysis was "marched forward" in time until the total pore pressure distribution in the solution reached a steady state. At this point, the 1993 stages of construction were modelled in the following order: (i) excavation of the north slope; (ii) excavation and construction of the north diaphragm wall panel; (iii) excavation and construction of the south diaphragm wall panel; and (iv) construction of the crosswalls.

Predicted plasticity zones

Figures 12, 13, and 14 show a comparison of the plasticity zones predicted to exist in the silty clay till stratum beneath the approach cut slopes after the excavation of the north slope, the north panel, and the south panel of the diaphragm wall, respectively. The upper cross sections on the figures show the predicted plasticity zones for the analysis where the gassy soil behaviour is not included. The lower cross sections on the figures show the predicted plasticity zones for the analyses that incorporate the gassy soil behaviour.

It can be seen that including the gassy soil layer has a major impact on the shape, location, and extent of plasticity predicted to develop beneath the south slope. Superimposed on this figure is the location of the actual 1993 field slip surface (as interpreted from the movements measured in the slope inclinometer (SI) installations and softened zones identified in the results of the CPTs). For the gassy soil case, the agreement between the location of the field-interpreted slip zone and the zones of plasticity predicted by the FEA to develop beneath the south slope is quite good.

Fig. 12. Predicted plasticity zones after excavation of north slope — effect of gassy soil layer. N/C, normally consolidated; O/C, overconsolidated.

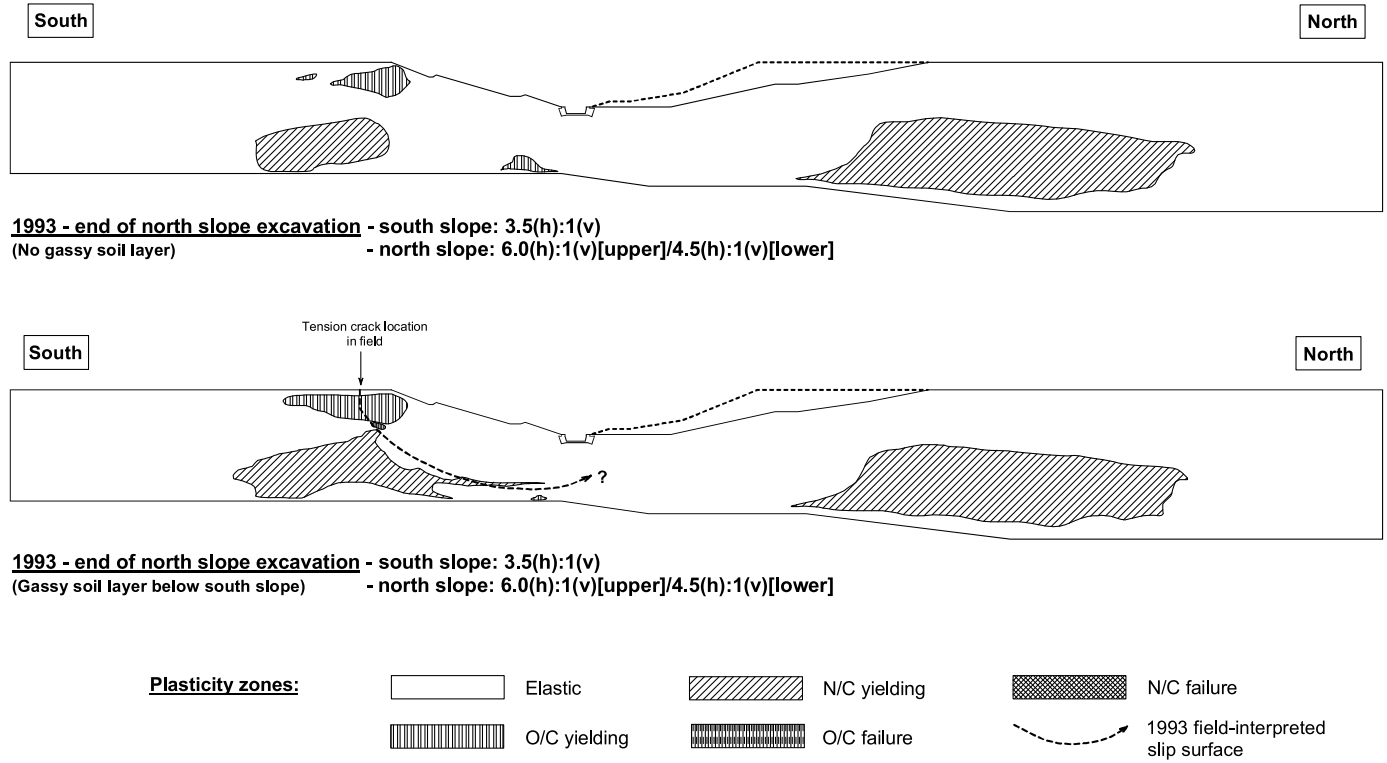
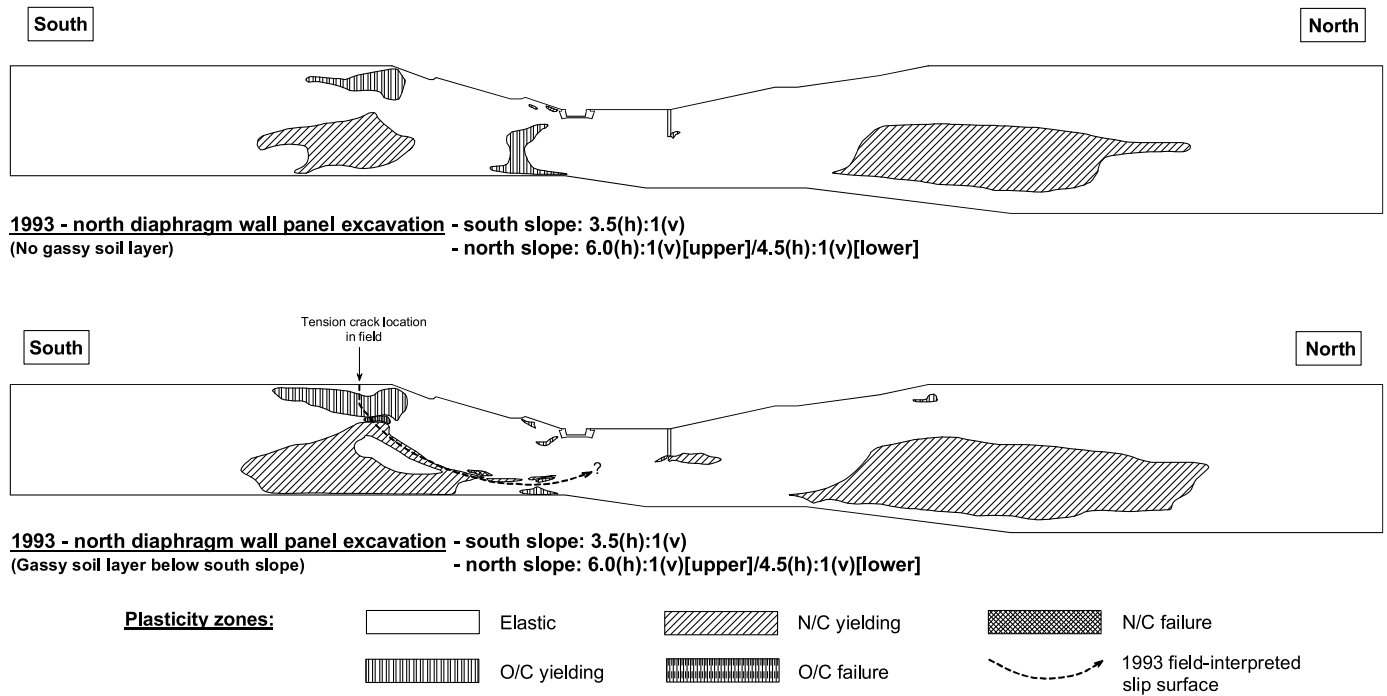


Fig. 13. Predicted plasticity zones after excavation of north panel of diaphragm wall — effect of gassy soil layer.



Predicted deformations

The horizontal deformations predicted by the FEA to occur at the locations of the field SI installations on the south and north slope of the Sarnia approach and the end of the 1993 construction of the diaphragm wall structure are presented in Fig. 15. The results from the analyses including

and neglecting the effects of the gassy soil layer are shown for comparison.

The analysis including the effects of the gassy soil layer predicts that on the south slope much more concentrated zones of movement occur. These zones are located just above elevation 150 m at the SI locations on the lower

Fig. 14. Predicted plasticity zones after excavation of south panel of diaphragm wall — effect of gassy soil layer.

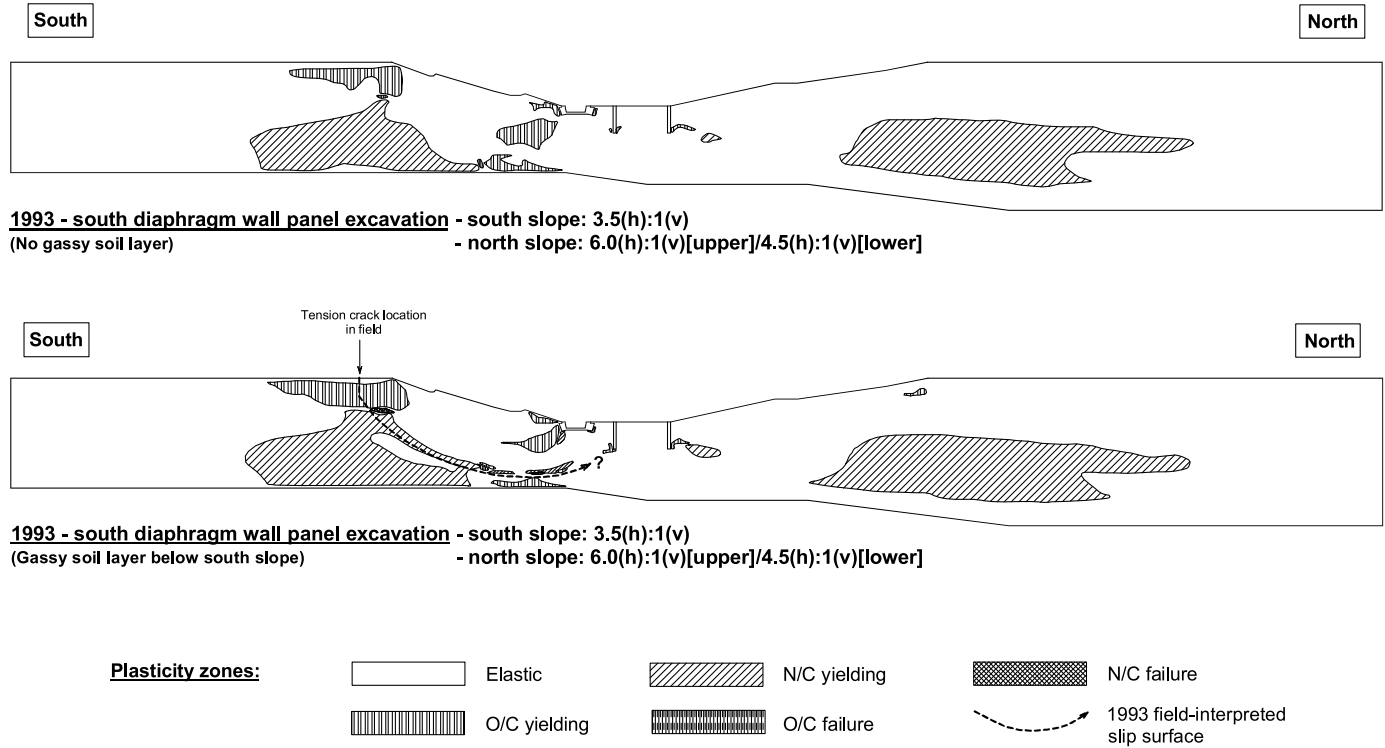
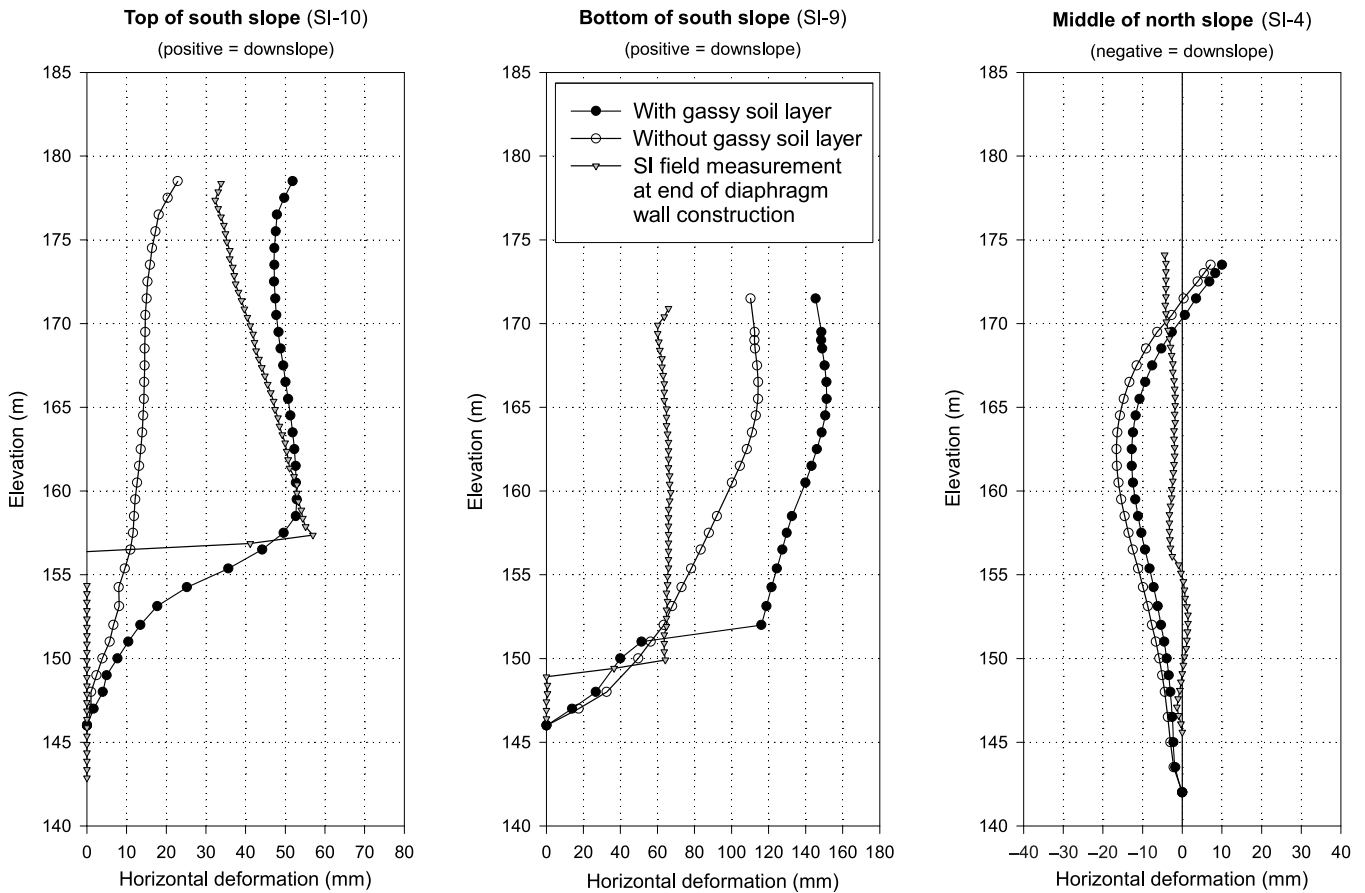


Fig. 15. Comparison of predicted deformations at SI installation locations, end of 1993 construction.



bench of the slope (for example, SI-9) and just above elevation 157 m at the SI locations on the upper bench of the slope (for example, SI-10). The results (in terms of the location of concentrated zone of movement) are very similar to the observations noted in the field at these two SI locations, as shown previously at about one-third of the way through construction in Fig. 4 and as shown at the end of the diaphragm wall construction in Fig. 15. Additionally, the profiles of the predicted deformations at both SI locations on the south slope appear to reflect a more rigid block-type movement (consistent with field SI measurements) for the gassy soil analysis compared with the analysis that does not include the gassy soil layer. As shown in Fig. 15, the predicted magnitude of down-slope deformation near the top of the slope (near SI-10) is similar to the total down-slope movements measured in the SIs near the end of construction (i.e., about 60 mm). However, the magnitude of deformation predicted near the bottom of the slope (near SI-9 located closer to the diaphragm wall at the base of the cut) is overestimated in the analysis by a factor of about two. The overprediction of the movements in this area is likely a result of the two-dimensional, plane strain analysis not being able to capture the three-dimensional nature of the construction of the adjacent diaphragm wall panels, which were specifically excavated and concreted in a series of short segments to minimize the potential and (or) magnitude of movement of the slopes.

On the north slope, both the gassy soil and nongassy soil analyses predict similar responses as shown in Fig. 15. Importantly, the magnitude of the final deformations predicted at the middle of the north slope (near SI-4) are substantially less than those predicted for the south slope, which is consistent with the observations and measurements made in the field.

Conclusions

A method of identifying the extent of gas permeation into the St. Clair till at the Sarnia approach cut based on diffusion modelling and field sampling has been discussed. An approach to modelling the effects of the presence of ex-solved natural gases within fine-grained soils subjected to unloading has been presented based on the work of Sobkowicz and Morgenstern (1984).

Including the effects of gassy soil behaviour into a fully coupled, effective stress FEA resulted in substantially better predictions of the plasticity zones and the trend of deformations on the slopes of the Sarnia approach cut in response to the 1993 construction at the site. Using this approach, the deep-seated deformations observed at the site and measured in the slope inclinometers during the unloading of the north slope and construction of the diaphragm wall were accurately predicted in the FEA. The results highlight the importance of a mechanism not usually considered in this type of analysis.

Acknowledgement

The research reported in this paper was funded in part by the Natural Science and Engineering Research Council of Canada.

References

- Becker, D.E., Crooks, J.H.A., Been, K., and Jefferies, M.G. 1987. Work as a criterion for determining in situ and yield stresses in clays. *Canadian Geotechnical Journal*, **24**(4): 549–564. doi:10.1139/t87-070.
- Becker, D.E., Dittrich, J.P., Walker, A.J., and Ruel, M. 1996. Design and performance of approach cut slopes, CN St. Clair River Tunnel project. *In Proceedings of the 49th Canadian Geotechnical Conference*, St. John's, N.L., 23–25 September 1996. *Edited by J.I. Clark*. Canadian Geotechnical Society, Richmond, B.C. pp. 37–44.
- Biot, M.A. 1941. General theory of three-dimensional consolidation. *Journal of Applied Physics*, **12**(2): 155–164. doi:10.1063/1.1712886.
- Brandes, H.G. 1999. Predicted and measured geotechnical properties of gas-charged sediments. *International Journal of Offshore and Polar Engineering*, **9**(3): 219–225.
- Carter, J.P., and Balaam, N.P. 1990. AFENA - A general finite element algorithm: user's manual. Custom version (modified for this research) [computer program]. School of Civil Engineering and Mining Engineering, University of Sydney, Sydney, Australia.
- Carter, T.R., Trevall, R.A., and Easton, R.M. 1996. Basement controls on some hydrocarbon traps in southern Ontario, Canada. *In Basements and Basins of Eastern North America*. Special Paper 308. *Edited by B.A. van der Pluijm, and P.A. Catacosinos*. Geological Society of America, Boulder, Colo. pp. 95–107.
- Chen, W.F., and Mizuno, E. 1990. Non-linear Analysis in Soil Mechanics - Theory and Implementation. *In Developments in Geotechnical Engineering* 53. Elsevier Science Publishing Company Inc., New York.
- D'Astous, A.Y., Ruland, W.W., Bruce, J.R.G., Cherry, J.A., and Gillham, R.W. 1989. Fracture effects in the shallow groundwater zone in weathered Sarnia-area clay. *Canadian Geotechnical Journal*, **26**(1): 43–56. doi:10.1139/t89-005.
- Dittrich, J.P. 2000. Slope behaviour during excavation of the Sarnia approach to the St. Clair Tunnel. Ph.D. Thesis, The University of Western Ontario, London, Ont.
- Dittrich, J.P., Rowe, R.K., and Becker, D.E. 1997. A history of failures at the St. Clair River Tunnel. *In Proceedings of the 50th Canadian Geotechnical Conference*, Ottawa, Ont., 20–22 October 1997. BiTech Publishers Ltd., Richmond, B.C. pp. 234–244.
- Dittrich, J.P., Becker, D.E., Rowe, R.K., and Lo, K.Y. 2000. Analysis of the 1890's excavation of the Sarnia approach to the St. Clair River Tunnel. *In Proceedings of the 53rd Canadian Geotechnical Conference*, Montréal, Que., 15–18 October 2000. Bi-Tech Publishers Ltd., Richmond, B.C. pp. 759–768.
- Dittrich, J.P., Rowe, R.K., Becker, D.E., and Lo, K.Y. 2002. Analysis of the 1990's Excavation of the Sarnia Approach for the New St. Clair Tunnel Project. *Proceedings of the 55th Canadian Geotechnical Conference*, Niagara Falls, Ont, 20–23 October 2002. BiTech Publishers Ltd., Richmond, B.C. pp. 299–306.
- Golder Associates Ltd. 1992. Site investigation, CN – St. Clair River Tunnel project, Sarnia, Ontario, and Port Huron, Michigan. Golder Associates Ltd., Mississauga, Ont. Report No. 921-1322H and Addendum Report submitted to Hatch/Mott MacDonald Joint Venture.
- Golder Associates Ltd. 1994. Additional geotechnical investigation and stability analysis – south slope – St. Clair River Tunnel project, Sarnia, Ontario. Golder Associates Ltd., Mississauga, Ont. Report No. 921-1322F.
- Griffiths, D.V. 1986. Some theoretical observations on conical failure criteria in principal stress space. *International Journal of Solids and Structures*, **22**(5): 553–565. doi:10.1016/0020-7683(86)90044-2.
- Grozic, J.L., Robertson, P.K., and Morgenstern, N.R. 1999. The be-

- behaviour of loose gassy sand. *Canadian Geotechnical Journal*, **36**(3): 482–492. doi:10.1139/cgj-36-3-482.
- Humphrey, D.N., and Holtz, R.D. 1988. Cap parameters for clayey soils. In *Numerical Methods in Geomechanics. Edited by G. Swoboda*. Balkema, Rotterdam, the Netherlands. pp. 441–446.
- Intera Technologies Ltd. 1992. Hydrogeologic study of the fresh-water aquifer and deep geologic formations, Sarnia, Ontario, Vol. 1. Prepared for the Ontario Ministry of the Environment. Intera Technologies Ltd., Ottawa, Ont.
- Jaky, J. 1944. A nyugalmi nyomás tényezője (The coefficient of earth pressure at rest). *Magyar Mernok es Eptesz-Egylet Kozlonye*, **78**(22): 355–358. [In Hungarian.]
- Kulhawy, F.H. 1977. Embankments and Excavations. In *Numerical methods in geotechnical engineering. Edited by C.S. Desai, and J.T. Christian*. McGraw-Hill, Inc., New York.
- Kulhawy, F.H., and Mayne, P.W. 1990. Manual on estimating soil properties for foundation design. EL-6800, Research Project 1493–6. Prepared for the Electric Power Research Institute, Palo Alto, Calif.
- Kuo, L.C. 1997. Gas exsolution during fluid migration and its relation to overpressure and petroleum accumulation. *Marine and Petroleum Geology*, **14**(3): 221–229. doi:10.1016/S0264-8172(97)00003-2.
- Masood, T., and Mitchell, J.K. 1993. Estimation of in situ lateral stress in soils by cone penetration test. *Journal of Geotechnical Engineering*, **119**(10): 1624–1639. doi:10.1061/(ASCE)0733-9410(1993)119:10(1624).
- Mayne, P.W., and Holtz, R.D. 1988. Profiling stress history from piezocone soundings. *Soils and Foundations*, **28**(1): 16–28.
- Naylor, D.J. 1978. Stress–strain laws for soil. In *Developments in Soil Mechanics – 1. Edited by C.R. Scott*. Applied Science Publishers, Ltd., London.
- Qigley, R.M. 1980. Geology, mineralogy, and geochemistry of Canadian soft soils: a geotechnical perspective. *Canadian Geotechnical Journal*, **17**(2): 261–285. doi:10.1139/t80-026.
- Rowe, R.K., and Booker, J.R. 2005. POLLUTE Pollutant migration through a nonhomogeneous soil. Version 6.3 [computer program]. Distributed by GAEA Environmental Engineering, Ltd., Whitby, Ont.
- Sanford, B.V., Thompson, F.J., and McFall, G.H. 1985. Plate tectonics – a possible controlling mechanism in the development of hydrocarbon traps in southwestern Ontario. *Bulletin of Canadian Petroleum Geology*, **33**(1): 52–71.
- Sobkowicz, J.C., and Morgenstern, N.R. 1984. The undrained equilibrium behaviour of gassy sediments. *Canadian Geotechnical Journal*, **21**(3): 439–448. doi:10.1139/t84-048.

Terzaghi, K., and Peck, R.B. 1967. *Soil mechanics in engineering practice*. 2nd ed. John Wiley and Sons, Inc., New York.

Weaver, T.R. 1994. Groundwater flow and solute transport in shallow Devonian bedrock formations and overlying Pleistocene units, Lambton County, southwestern Ontario. Ph.D. thesis, University of Waterloo, Waterloo, Ont.

List of symbols

B	Skempton pore-pressure coefficient
C_c	compression index
C_s	swelling index
c	concentration
c_0	initial source concentration
D_e	diffusion coefficient
h_T	total hydraulic head
J_2	second invariant of deviatoric stress tensor
K'_o	coefficient of lateral earth pressure at rest
$K'_{o\ n/c}$	coefficient of lateral earth pressure at normally consolidated condition
k_h	hydraulic conductivity (horizontal)
k_v	hydraulic conductivity (vertical)
n_e	effective porosity
OCR	overconsolidation ratio
p'	mean effective stress
q_c	tip resistance (measured by cone penetration test)
R'_c	aspect ratio for elliptical cap yield surface in $\sqrt{2}J_2 - \sigma'_m$ stress space
s_u	undrained shear strength
u	pore pressure
Δu	excess pore pressure
$u_{l/g}$	liquid–gas saturation pressure
u_{max}	pore pressure maximum (measured by cone penetration test)
u_T	total pore pressure (= excess + initial)
v_a	Darcy velocity
γ_{sat}	saturated unit weight
ν'	drained Poisson's ratio
σ'	effective stress
$\Delta\sigma'$	change in effective stress
σ'_m	mean effective stress
σ'_{yo}	in situ vertical effective stress
σ'_{vp}	vertical preconsolidation pressure
σ_T	total stress (= σ)
ϕ'	effective friction angle