Performance of Vertical Shafts Constructed In Swelling Shale Using Finite Element Method

Hayder Mohammed Salim Al-Maamori[1], and M. Hesham El Naggar[2]
[1] *PhD, PEng, Post-Doctoral Fellow, mitacs elevate,*
[2] *PhD, PEng, Professor,*
Civil and Environmental Engineering Department, The University of Western Ontario, London, Ontario, Canada

ABSTRACT
Micro-tunneling technique is employed to construct pipelines and tunnels in different types of ground including swelling shales. However, these shales exhibit time-dependent deformation behavior, generally known as rock swelling that causes internal stresses in the underground structures in addition to those caused by the in-situ stresses. These combined stresses may exceed the allowable tensile or compressive strength of the concrete forming the pipeline, the tunnel liner and the walls of vertical shafts, which may result in cracks in these structures. Vertical shafts are used to initiate construction in micro-tunneling technique, then they are utilized as manholes for inspection and monitoring purposes of the constructed pipeline or tunnel. This paper investigates the induced stresses in vertical shafts caused by the time-dependent deformations of swelling shales from south-western Ontario region. The induced stresses in vertical shafts are calculated numerically using the finite element method and the computer program PLAXIS-2D. The vertical shaft is simulated using an axi-symmetrical model to accurately predict its behavior. The time-dependent deformation parameters of the host rock were derived from experimental test program that followed the theoretical model proposed by Lo and Hefny (1996) and they are used to define the rock swelling model of PLAXIS-2D. The results of the performed numerical analyses in this research may aid in adopting the suitable construction procedure to decrease the induced stresses in the walls of vertical shafts or it may lead to adjust their structural design, if needed.

RÉSUMÉ
La technique du micro-tunnel est utilisée pour construire des pipelines et des tunnels dans différents types de sol, y compris des schistes gonflants. Cependant, ces schistes présentent un comportement de déformation dépendant du temps, généralement connu sous le nom de gonflement des roches, qui provoque des contraintes internes dans les structures souterraines en plus de celles provoquées par les contraintes in situ. Ces contraintes combinées peuvent dépasser la résistance à la traction ou à la compression admissible du béton formant la conduite, du revêtement du tunnel et des parois des puits verticaux, ce qui peut entraîner des fissures dans ces structures. Les puits verticaux sont utilisés pour initier la construction en technique de micro-tunnel, puis ils sont utilisés comme trous d'homme pour l'inspection et la surveillance du pipeline ou du tunnel. Cet article examine les contraintes induites dans les puits verticaux provoquées par les déformations dépendantes du temps des schistes gonflants du sud-ouest de la région de l'Ontario. Les contraintes induites dans les puits verticaux sont calculées numériquement en utilisant la méthode des éléments finis et le programme informatique PLAXIS-2D. L'arbre vertical est simulé en utilisant un modèle axi-symétrique pour prédire avec précision son comportement. Les paramètres de déformation dépendant du temps de la roche hôte ont été dérivés d'un programme d'essais expérimentaux qui a suivi le modèle théorique proposé par Lo et Hefny (1996) et sont utilisés pour définir le modèle de gonflement des roches de PLAXIS-2D. Les résultats des analyses numériques réalisées dans le cadre de cette recherche peuvent aider à adopter la procédure de construction appropriée pour réduire les contraintes induites dans les parois des puits verticaux ou, le cas échéant, à ajuster leur conception structurelle.

1 INTRODUCTION

The swelling behavior of shales, such as the Queenston shale, from the region of south-western Ontario has been extensively investigated. Most of the earlier researches have focused on identifying the main factors that can influence the swelling behavior of these shales, define its controlling mechanism and design laboratory tests to measure and quantify it. Those researches have been conducted over a period of four decades where different aspects of the swelling phenomena of these shales were considered (Al-Maamori et al., 2016, Hefny et al., 1996; Lo and Hefny, 1996; Lee and Lo, 1993; Lo and Lee, 1990; Lo and Yuen, 1981; Yuen, 1979; and Lo et al., 1978 and 1975). The general conclusion was that the swelling of these shales occurs when all or some of the following conditions are met: i) the relief of the initial in-situ stress; ii) the accessibility to water and iii) an outward gradient of the pore fluid salinity from the rock to the ambient fluid (Hefny and Lo, 1996). In most of underground constructions in swelling shales, these conditions do exist. Therefore, the swelling effects of these shales are expected to occur and the measures to control them are essentially required.
Three types of laboratory tests were developed earlier to measure and quantify shale swelling. These tests are: i) the free swell test; ii) the semiconfined swell test, and iii) the null swell test (Lo and Hefny, 1999, and Lo et al., 1978). Lo and Hefny (1996) developed a model to represent the results of these tests in a mathematical form. This model was fairly similar to Grob (1972)'s swelling model, originally developed for swelling rocks that contain swelling clay minerals. In parallel to the swelling phenomena, the

strength of swelling shales from the region of south-western Ontario was also investigated thoroughly. Due to their sedimentary forming nature, these shales exhibit different strength in the orthogonal directions with respect to their bedding. This feature was investigated and the strength anisotropy of these shales was clearly defined (Lee, 1988; Lo et al., 1987; Lo and Yuen, 1981; Wai et al., 1981; Lo and Hori, 1979, and Yuen, 1979).

Another important aspect of the behavior of these shales is their strength degradation caused by increasing their moisture content. Although, the strength degradation is a common feature in most rocks, it can be more evident in swelling shales due to the developed micro cracks in their structure when they are exposed to water (Al-Maamori et al., 2018; Wasantha and Ranjith, 2014; Dan et al., 2013; Liang et al., 2012; Gorski et al., 2007; Paterson and Wong, 2005; Claesson and Bohloli, 2002; Baud et al., 2000; Paterson, 1978, and Colback and Wiid, 1965). In order to predict the swelling zone of the Queenston shale, Al-Maamori, et al., (2017) developed a laboratory test that account for the swelling effects of the shale and on the same time measures the depth of penetration of water and lubricant fluids used in micro-tunneling applications. From this test, a useful correlation was established to predict the depth of the swelling zone of the Queenston shale over years of swelling. Al-Maamori et al. (2018) numerically investigated the swelling effects of the Queenston shale and predicted the generated stresses in the walls of a pipeline or a tunnel over a period of thirty years. The numerical model incorporated the swelling parameters that were derived from swelling laboratory tests (Al-Maamori et al., 2016 and 2017), including: depth of swelling zone, shale strength, and the strength degradation that occurs along the swelling zone. The Lo and Hefny (1996)'s swelling model was utilized to simulate the swelling behavior in a two-dimensional finite element analysis in PLAXIS-2D environment. Full description of the numerical model and the steps that were followed in the validation process of the model can be found in (Al-Maamori, et al., 2018).

In this paper, the effects of the swelling behavior of the Queenston shale on the walls of vertical shafts constructed as part of micro-tunneling process was investigated. The rock swelling model in PLAXIS-2D was utilized using the axi-symmetry modeling function. Parameters of the Queenston shale, such as: strength, size of the swelling zone and its reduced strength, the swelling parameters, and the permeability were adopted from Al-Maamori et al., (2018). The results of the numerical study are presented and discussed.

1.1 Micro-tunneling Process

The micro-tunneling technique is widely used to construct new underground structures and infrastructures in major cities of south-western Ontario region. In this construction technique, the boring machine must be lowered to the elevation of the proposed pipeline or utility tunnel to commence the drilling process. Therefore, an access into the ground must be prepared. Usually, the accesses are prepared in the form of vertical shafts that are drilled and constructed along the rout of the proposed pipeline or the utility tunnel. They serve as launching and receiving shafts during the micro-tunneling construction process, and as inspection manholes along the service life of the structure. Drilling these vertical shafts usually performed using different excavation techniques that depend on the nature of the excavated ground. Vertical shafts need to be deep enough to reach the proposed elevation of the pipeline or the utility tunnel, and in most cases, they may extend through the bedrock. Some of these vertical shafts may have large depths below the natural ground level that may reach 30 m or more. At such depths, the walls of these shafts must support the lateral earth pressure and the hydrostatic pressure of the ground water. In the case of vertical shafts drilled in swelling rocks, such as the Queenston shale, their walls need to be strong enough to support the additional pressure caused by the inward swelling deformations of these shales.

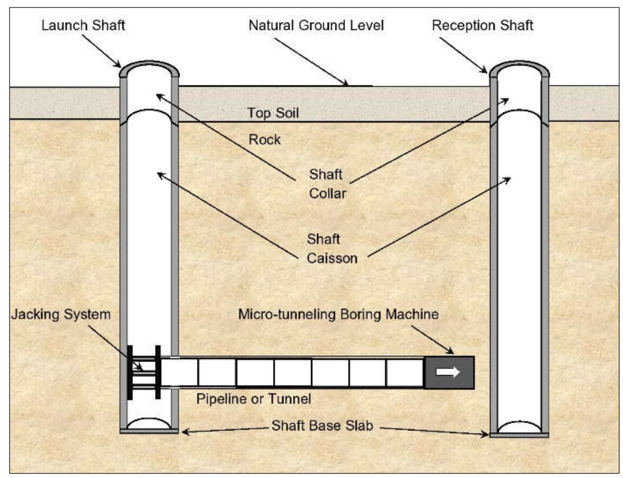

Figure 1. Schematic illustration of Vertical Shaft and micro-tunnelling construction technique

As indicated in Figure 1, it is essential that vertical shafts are wide enough to accommodate the hydraulic jacking system and the micro-tunneling boring machine. The common procedure followed in the construction of vertical shafts in micro-tunneling applications is to cast the concrete of the upper part of the shaft above the ground level (i.e. the collar), which would be then pushed into the ground with the continuous excavation of the inner and underneath soil. The collar supports the lateral earth pressure generated from for the overburden soil layer and the hydrostatic pressure of the ground water. It is supplied with steel reinforcement that overlaps with the successive concrete pours that are usually 2 m -2.5 m in height. When the drilling reaches the bedrock, the caisson is constructed using the underpinning process. The caisson is the lower part of the vertical shaft that supports the lateral earth pressure within the rock layer. It is usually constructed in successive concrete pours of 1.5 m - 2.5 m height. For the caisson portion, the rock is excavated first to the required level and to the same outer diameter of the collar portion, followed by pouring the concrete of the caisson's wall. This procedure is repeated several times until the bottom level of the shaft is reached. Finally, the bottom slab of the shaft is casted. Typically, several shafts are constructed following this procedure along the rout of the pipeline or the utility tunnel. The hydraulic jacking system is installed at the bottom of the launch shaft and the micro-tunneling boring machine is located at the front of the hydraulic system to commence the drilling process. When the micro-tunneling boring machine advances through the shale, the segments of the pipeline or the utility tunnel are simultaneously installed from its behind. The installation continues until the micro-tunneling boring machines reaches the receiving shaft. This process is repeated several times to complete the installation of the pipeline or the utility tunnel.

2 NUMERICAL MODELING METHODOLOGY

The computer program "PLAXIS" has a user-defined model that simulates the swelling behavior of rocks based on Grob's swelling model (1972). This model was originally developed based on the swelling behavior of the clay minerals included in the rock. This model was adopted in PLAXIS computer program as a user-defined model in two-dimensional environment by Schädlich et al., (2012 and 2013). Al-Maamori et al. (2018) and Al-Maamori (2016) demonstrated that this model is identical to Lo and Hefny (1996) mathematical model of swelling rocks in the region of south-western Ontario. It was revealed that this model can predict the deformations and stresses in tunnels and pipelines constructed in swelling shales, using the parameters derived from the free swell tests, the semi-confined swell test and the null swell test. This paper investigates the development of the swelling-induced stresses in the walls and the base of vertical shafts used in micro-tunneling applications. An axi-symmetrical model of vertical shafts was developed in PLAXIS-2D environment and the finite element analysis was conducted utilizing the adopted rock swelling model, and the swelling parameters that were derived based on Lo and Hefny (1996) swelling model.

Table 1. Properties of the Queenston shale used in the analyses

Parameter	Value
Unit Weight (kN/m³)	25.74
Stress Ratio:	
(horizontal stress in X direction /vertical stress)	4.00
(horizontal stress in Y direction /vertical stress)	2.44
Friction angle, φ (degrees)	20.6
Cohesion, C (kPa)	2590
Tensile strength (kPa)	4700
Elastic Modulus, vertical direction (kPa)	$14.21*10^6$
Elastic Modulus, horizontal direction (kPa)	$11.48*10^6$
Poisson's ratio, vertical direction	0.23
Poisson's ratio, horizontal direction	0.25
Vertical swelling potential (%)	0.525
Horizontal swelling potential (%)	0.210
Critical stress, vertical direction (kPa)	2070
Critical stress, horizontal direction (kPa)	1600
Permeability (m/day)	$16.4 *10^{-6}$

In the presented analyses, the swelling parameters of the Queenston shale were adopted from (Al-Maamori et al., 2018). The strength of the Queenston shale was adopted based on strength tests performed on the same shale (Al-Maamori et al., 2017). The depth of the swelling zone within the Queenston shale layer and its permeability were predicted based on the correlations presented in (Al-Maamori et al., 2017). These values are summarized in Table 1. In the following analyses, the properties of the Queenston shale layer was adopted for samples collected from the Niagara Falls region in Canada.

3 RESULTS AND DISCUSSION

In order to account for the swelling effects, the in-situ stresses, the lateral earth pressure and the overburden pressure, a cylinder of 120 m diameter and 100 m depth was modeled. The model includes 95 m of Queenston shale layer overlain by 5 m thick of the overburden soil. The ground water level was located at 2 m below the natural ground level. Using the axi-symmetry function available in PLAXIS-2D, only half cylinder was modeled, i.e. a 100 m deep cylinder of 60 m radius. Boundaries were assigned to the sides of the cylinder with restricted movement in the horizontal direction and free movement in the vertical direction, while the movement of the cylinder base was vertically restricted. The excavation boundaries and the top of the model were assigned to free movement in both vertical and horizontal directions. The default, very fine mesh size was used with additional refinement near the boundaries of the excavation to generate the finite element mesh with 2509 triangular elements. Figure 2 shows the finite element mesh of the model. The stress ratio of the Queenston shale layer (i.e. the horizontal in situ stress / the vertical in situ stress) was taken as 4.00 in the major horizontal in situ stress direction and 2.44 in the direction of minor in situ horizontal stress (Al-Maamori et al., 2014). The excavation of the overburden soil was modeled in two phases with each phase 2.5 m of the soil been removed in three days. This period was selected based on the observations made in several projects in the region. The same sequence was followed in the excavation of the Queenston shale layer with adding another phase for casting the concrete of the caisson portions of the shaft after each excavation phase.

Excavating the Queenston shale layer in each excavation phase was modeled in seven days, as an average duration required to perform the excavation in similar field cases. The collar portion of the shaft was modeled as one piece due to its steel reinforcement, while the caisson was modeled as individual pieces of pure concrete without steel reinforcement. When the final depth of the shaft was reached, the base slab was activated in the following phase. In the analyses, the thickness of the shaft walls (i.e. the collar and caisson) was taken 0.3 m and the base slab thickness was taken 0.15 m. The duration of the swelling phases in the presented analyses was 30 years. This period was selected where the majority of the swelling deformations of the shale may occur.

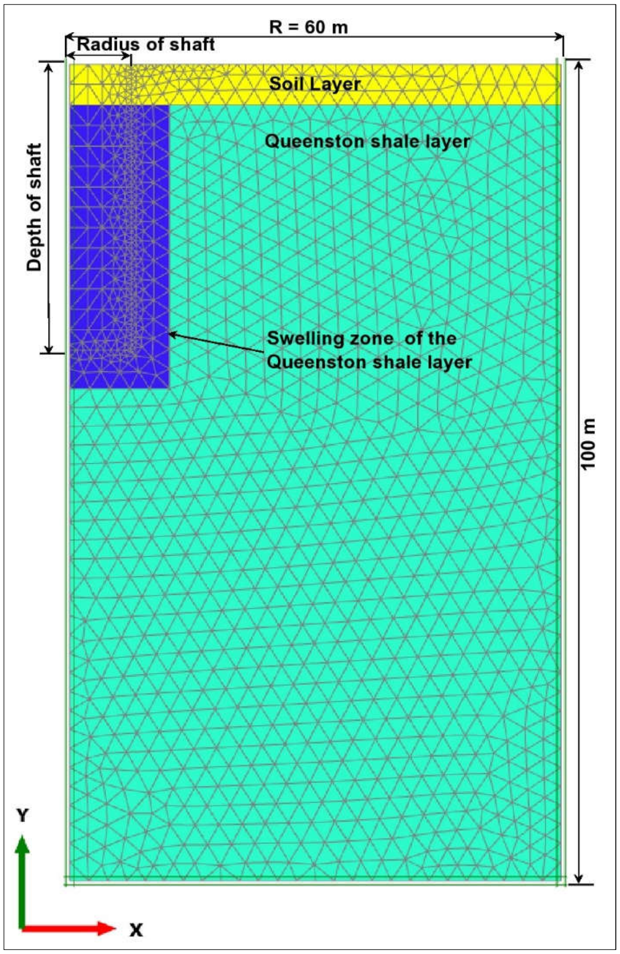

Figure 2. Finite element mesh of the model

3.1 Shaft Inward Deformation

The collar and caisson inward deformation after 30 years of swelling is shown in Figure 3 for 30 m deep shafts of 2.5 m, 5.0 m and 7.5 m radii. The inward deformation for the three radiuses increased with increasing the depth of the shaft, and its maximum value occur at the lower part of the shaft (i.e. the caisson). Increasing the radius of the shaft causes more inward deformation due to the larger excavated ground and the larger swelling zone. Figure 3 shows that the soil portion of the shaft (i.e. the collar) has almost negligible inward deformation for the three presented cases.

3.2 Shaft base slab upward deformation

Figure 4 presents the upward deformation of the base slab of 30 m deep shaft with three radius values: 2.5 m, 5.0 m, and 7.5 m. The maximum upward deformation occurs at the central part of the slab while the minimum deformation occurs near the caisson. For the shaft of large radius (i.e. 7.5 m), it can be noted that there is a small downward deformation near the caisson. This deformation can be attributed to the self-weight of the shaft and to the settlement effects of the overburden soil.

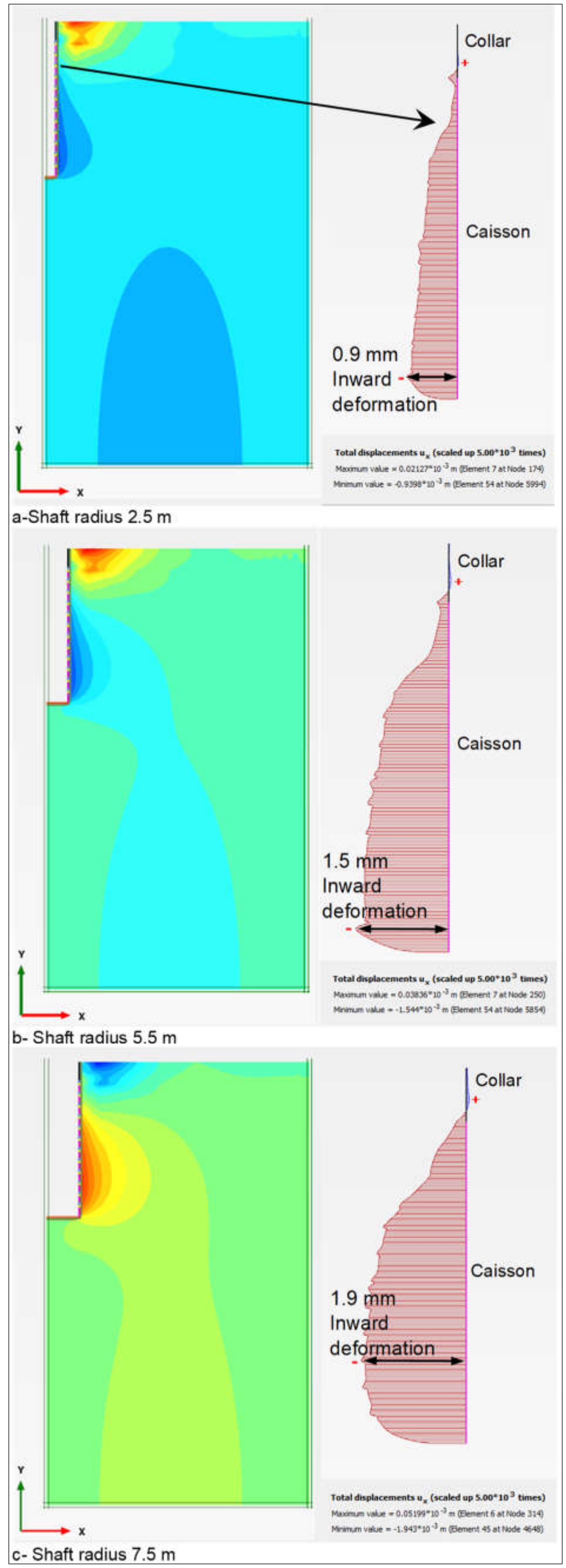

Figure 3. Inward deformation of 30 m deep shaft

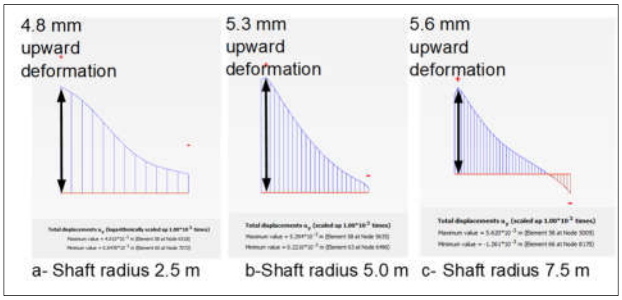

Figure 4. Upward deformation of 30 m shaft's base slab

3.3 Hoop force (N_z), shear force and bending moment

Figure 5 displays typical distribution of the hoop and shear forces, and the bending moment along the walls of a 30 m deep shaft. The forces and moments shown in this figure are those generated in the collar and the caisson of the shaft after 30 years of swelling. The generated hoop force is compression along the collar and the caisson. It gradually increases with increasing the depth below the natural ground level. The decrease in its value at the bottom of the shaft may be attributed to the restriction caused by the underneath Queenston shale.

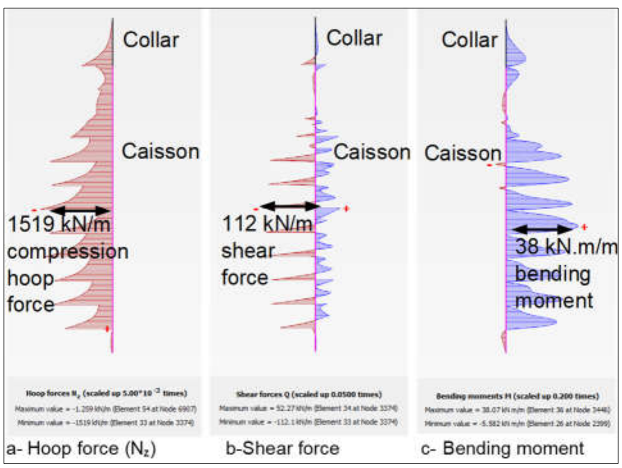

Figure 5. Distribution of the hoop force, shear force, and bending moment along walls of 30 m deep shaft

The hoop force causes the compressional and tensile stresses at the walls of the shaft in the direction perpendicular to the model. The shear force increases with increasing the depth at the collar portion causing maximum bending moment near its bottom. At the caisson portion, the shear force has large values at the top and bottom of each caisson slab causing the maximum bending moment near its center. The maximum bending moment occurs at the location of maximum shear force close to mid depth of the shaft.

3.4 Tensile stresses in the shaft

The concrete strength of the shaft walls and base slab considered in the analyses was 32 MPa, and therefore its tensile strength would be 3.2 MPa (Neville, 1996).
For the presented case in Figure 6, the shaft has a depth of 30 m and a wall of 0.3 m thick at both collar and caisson portions, and a base slab of 0.15 m thick. As it can be noted from Figure 6, the tensile stresses in the shaft wall and the base slab are well below the concrete tensile strength. The highest tensile stresses occur at the caisson portion, where the swelling deformation of the Queenston shale is expected to be at its maximum value.

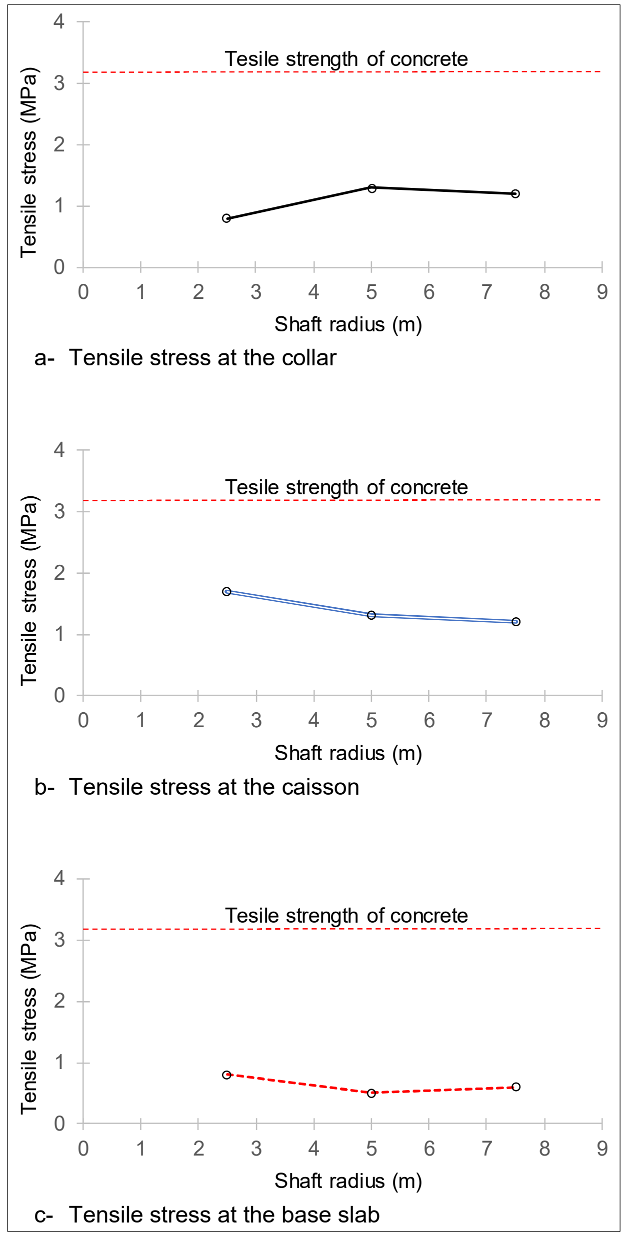

Figure 6. Tensile stress at the extreme fiber of the collar, caisson, and the base slab of 30 m deep shaft

Figure 7 shows the variation in the tensile stress at the extreme fibers of the walls of the collar and the caisson with increasing the shaft's depth. It can be clearly indicated that with increasing the depth of the shaft, the tensile stresses in the collar and the caisson increases. With increasing the shaft's depth, the swelling of larger mass of the Queenston shale is triggered. For the particular presented case, shafts with depths greater than 27 m, it is expected that the tensile stress at the wall of the caisson may exceed the tensile strength of the concrete. At such large depths, tension cracks are expected to be developed at the outer and the inner face of the caisson wall.

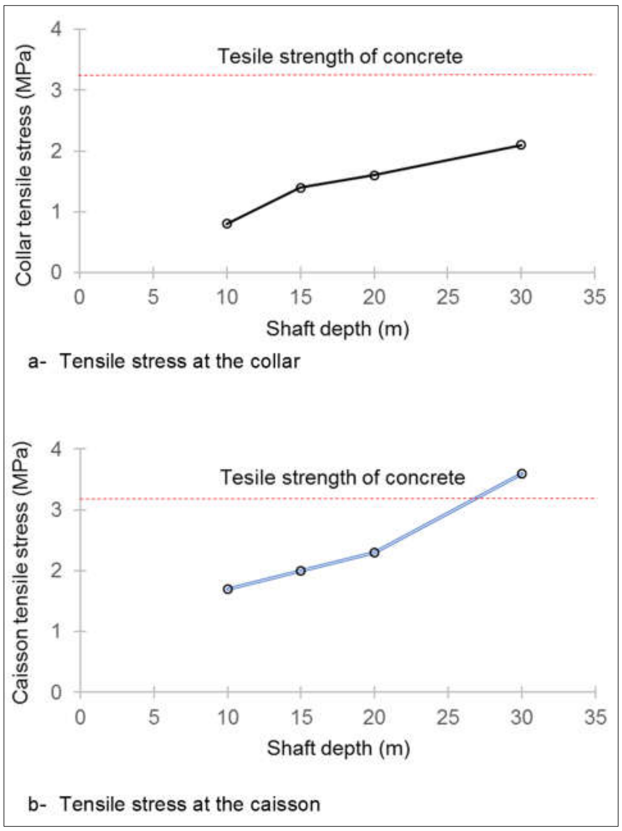

Figure 7. Variation of tensile stresses at the shaft walls

4 SUMMARY AND CONCLUSIONS

The computer program PLAXIS-2D was used to model vertical shafts of micro tunneling applications constructed in Queenston shale. The developed model was constructed based on the swelling parameters derived based on Lo and Hefny (1996)'s swelling model. The model provides an insight of the expected deformations and stresses that may be generated in the walls and base of the shaft due to the swelling effects of the Queenston shale layer. The maximum inward deformation along walls of the shaft was shown to be developed at the lower portion of the caisson, causing the highest shear force and largest bending moment. The maximum upward deformation occurs at the central part of the base slab. For the presented case of shaft with 0.3 m thick wall (i.e. collar and caisson) and 0.15 m thick base slab, it is expected to have safe performance at depths from 10 m to 27 m. Similar shafts with depth greater than 27 m are expected to experience tension cracks at their outer and inner faces.

Acknowledgment

This research work has been done under the MITACS Elevate post-doctoral industrial fellowship at the Civil and Environmental Engineering Department, the University of Western Ontario in London, ON, Canada. The authors would like to acknowledge mitacs-Canada and Ward and Burke Microtunnelling Ltd., "the industrial partner in this research" for their financial and technical support.

5 REFERENCES

Al-Maamori, H.M.S. 2016. Investigation of time-dependent behaviour of micro-tunneling construction in Queenston shale. PhD Thesis. London, Canada: Department of Civil and Environmental Engineering, University of Western Ontario; 2016.

Al-Maamori, H.M.S., El Naggar, M.H., and Micic, S. 2018. Numerical modeling of time-dependent deformation and induced stresses in concrete pipes constructed in Queenston shale using micro-tunneling technique. *Journal of Rock Mechanics and Geotechnical Engineering*, 10: 290-309.

Al-Maamori, H.M.S., El Naggar, M.H., and Micic, S. 2017. Depth of penetration of lubricant fluids and water in Queenston shale of southern Ontario. *Canadian Geotechnical Journal,* 54(2):248-57.

Al-Maamori, H.M.S., El Naggar, MH., Micic S. 2014. A compilation of the geo-mechanical properties of rocks in southern Ontario and the neighbouring regions. *Open Journal of Geology*, 4(5):210-27.

Al-Maamori, H.M.S., El Naggar, M.H., Micic, S., Lo, K.Y. 2016. Influence of lubricant fluids on swelling behaviour of Queenston shale in southern Ontario. *Canadian Geotechnical Journal,* 53(7):1059-80.

Baud, P., Zhu, W., and Wong, T. 2000. Failure mode and weakening effect of water on sandstone. *Journal of Geophysical Research,* 105(B7):16371-89.

Claesson, J. and Bohloli, B. 2002. Brazilian test: stress field and tensile strength of anisotropic rocks using an analytical solution. *International Journal of Rock Mechanics and Mining Sciences*, 39(8):991-1004.

Colback, P.S.B. and Wiid, B.L.1965. The influence of moisture content on the compressive strength of rock. In: Proceedings of the 3rd Canadian rock mechanics symposium. Society of Exploration Geophysicists, p. 65-83.

Dan, D.Q., Konietzky, H., and Herbst, M. 2013. Brazilian tensile strength tests on some anisotropic rocks. *International Journal of Rock Mechanics and Mining Sciences,* 58:1-7.

Gorski, B., Conlon, B., Ljunggren, and B., A.T.2007. Forsmark site investigation: determination of the direct and indirect tensile strength on cores from borehole KFM01D. Report P-07e76. Stockholm, Sweden: Svensk Kärnbränslehantering AB, Swedish Nuclear Fuel and Waste Management Co.

Grob, H. 1972. Schwelldruck im Belchentunnel. In: Proceedings of International Symposium für Untertagebau, Luyern; p. 99e119 (in German).

Lee, Y.N. 1988. Stress-strain-time relationship of Queenston shale. PhD Thesis. London, Canada: Civil and Environmental Engineering Department, University of Western Ontario.

Lee, Y.N. and Lo, K.Y. 1993. The swelling mechanism of Queenston shale. In: TAC annual conference proceedings. TAC; p. 75-97.

Liang, W., Yang, X., Gao, H., Zhang, C., Zhao, Y., and Dusseault, M.B. 2012. Experimental study of mechanical properties of gypsum soaked in brine. *International Journal of Rock Mechanics and Mining Sciences,* 53:142-50.

Lo, K.Y., Cooke, B.H.,and Dunbar, D.D. 1987. Design of buried structures in squeezing rock in Toronto, Canada. *Canadian Geotechnical Journal,* 24(2):232e41.

Lo, K.Y. and Hefny, A. 1996. Design of tunnels in rock with long-term time-dependent and nonlinearly stress-dependent deformation. In: TAC annual conference proceedings. TAC; p. 179-214.

Lo, K.Y. and Hori, M. 1979. Deformation and strength properties of some rocks in Southern Ontario. *Canadian Geotechnical Journal* , 16(1):108-20.

Lo, K.Y., Lee, C.F., Palmer, J.H.L., and Quigley, R.M. 1975. Stress relief and time-dependent deformation of rocks. Final Report, National Research Council of Canada Special Project No. 7303. London, Canada: Faculty of Engineering Science, University of Western Ontario.

Lo, K.Y. and Lee, Y.N. 1990. Time-dependent deformation behaviour of Queenston shale. *Canadin Geotechnical Journal*, 27(4):461e71.

Lo, K.Y., Wai, R.S.C., Palmer, J.H.L., and Quigley, R.M. 1978. Time-dependent deformation of shaly rocks in Southern Ontario. *Canadian Geotechnical Journal,*15(4):537e47.

Lo, K.Y. and Yuen, C.M.K. 1981. Design of tunnel lining in rock for long term time effects. *,* 18(1):24e39.

Neville, A.M. 1996. Properties of concrete. 4th ed. Harlow, UK: Pearson Education Limited.

Paterson, M.S. and Wong, T.F. 2005. Experimental rock deformation: the brittle field. 2nd ed. Berlin-Heidelberg-New York: Springer-Verlag.

PLAXIS. PLAXIS 2D 2016-Material models manual. nl/plaxis2d/manuals/, http://www.plaxis.

Schädlich, B., Marcher, T., and Schweiger, H.F. 2012. Application of a constitutive model for swelling rock to tunnelling. *Geotechnical Engineering Journal of the SEAGS & AGSSEA,* 43(4):4-27.

Schädlich, B., Schweiger, H.F., and Mrcher, T. 2013. Modelling swelling rock behaviour in tunnelling. Plaxis Bulletin, Spring Issue: 1-9.

Wasantha, P.L.P. and Ranjith, P.G. 2014. Water-weakening behavior of Hawkesbury sandstone in brittle regime. *Engineering Geology*, 178:91-101.

Yuen, C.M. 1979 Rock-structure time interaction in lined circular tunnels in high horizontal stress field. PhD Thesis. London, Canada: Civil and Environmental Engineering Department, University of Western Ontario.