Effects of Tunnelling On the Bearing Capacity of Shallow Foundations

Zakhem, Anna- Maria and El Naggar, Hany
Department of Civil and Resource Engineering - Dalhousie University, Halifax, NS, Canada

[1] Graduate Student, Department of Civil and Resource Engineering, Dalhousie University, Canada.
[2] Associate Professor, Department of Civil and Resource Engineering, Dalhousie University, Canada.

GEOQuébec 2015
Challenges from North to South
Des défis du Nord au Sud

ABSTRACT
Urban tunnelling can induce distortion to the existing three-dimensional stress regime due to the development of ground arching and the associated stress redistribution. In some cases this may decrease the bearing capacity of nearby foundations which could cause damage to the overlying structures especially in soft soils and/or densely populated areas. Thus, the effect of tunneling on the performance and capacity of nearby foundations may be significant and should be considered in their design. This paper uses the finite element method with an elastic-plastic model and the Plaxis 3D software to study this interaction problem considering a number of parameters such as the tunnelling processes, construction sequence, proximity, and the burial depth. Accordingly, recommendations for the design of shallow foundations in close vicinity to shallow tunnels were proposed.

RÉSUMÉ
Le creusement de tunnels urbains peut induire une distorsion du régime de contrainte tridimensionnel en raison du développement d'une voûte et de la redistribution des contraintes associées. Dans certains cas, cela peut diminuer la capacité portante des fondations proches qui pourraient causer des dommages aux structures en surface, en particulier dans les sols mous et/ou les zones densément peuplées. Ainsi, l'effet des tunnels sur la performance et la capacité des fondations à proximité peut être important et doit être pris en compte dans leur conception. Ce document utilise la méthode des éléments finis avec un modèle élasto-plastique et le logiciel Plaxis 3D pour étudier ce problème d'interaction en examinant un certain nombre de paramètres, tels que les processus de creusement, la séquence de construction, la proximité et la profondeur d'enfouissement. En conséquence, des recommandations pour la conception des fondations superficielles à proximité immédiate de tunnels peu profonds ont été proposées.

1 INTRODUCTION

Several researchers have investigated the interaction problem between shallow foundations and urban tunnels taking in consideration a number of parameters such as the tunnelling process, construction sequence, proximity, foundation type, burial depth, the associated ground movement and possible damage to adjacent buildings. However none of them developed any solution that can be used to account for the tunnelling effect on the bearing capacity of foundations. Safe tunnel design dictates the necessity to ensure adequate stability for the tunnel and to control the effect of tunnelling-induced ground settlement on the nearby buildings. Also, the nearby foundations should have enough capacity to sustain the applied loads. Therefore, it is necessary to evaluate the effect of tunnelling on the bearing capacity of nearby foundations and will be useful to provide simple design equations to account for this effect.

This project will involve comprehensive numerical modeling using the finite element (FE) code Plaxis 3D. The effect of tunnelling on the bearing capacity of foundations will be investigated using 3D models. The FE mesh will be refined around and in close vicinity of the tunnel and near locations where non-linear behaviour is anticipated to assure high accuracy of the results. Interface elements allowing for both slippage and gapping to occur from the Plaxis library will be used to model the interface between the soil and the tunnel lining. Elasto-plastic constitutive models will be used to model the soil and the interface. The tunnel will be considered to be buried at different depths and proximities to the foundation. A wide range of material properties of the individual elements and soil (compressibility, strength, and modulus of elasticity) will be considered. This research will lead to the development of new design equations for shallow foundations in close vicinity of tunnels. This paper reports the results of the first phase of the study.

2 PROBLEM DEFINITION

This section presents the development of the FE models that were used to carry out the numerical analyses presented in this paper. The considered problem involves strip foundations resting on a thick clay layer underlain by bedrock at a great depth (Figure 1). The 3D FE models were established using the computer program PLAXIS 3D (PLAXIS bv, 2013) considering an appropriate size mesh and a number of elements following a sensitivity study. Terzaghi's conventional bearing capacity equation was used to calibrate the FE models. Then the calibrated FE models were employed to perform the parametric study to investigate the effect of tunnelling on the performance and capacity of nearby foundations. Also, the effect of the pre-existing foundations on the forces and deformations developed in the tunnel lining was investigated.

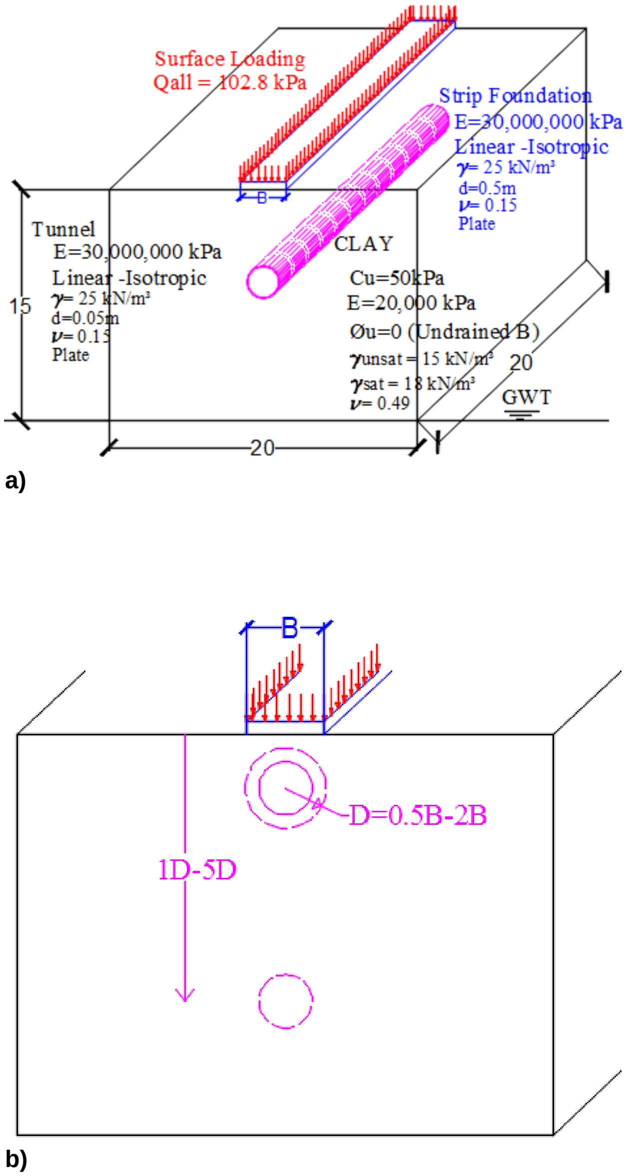

Figure 1. a) Geometry of a strip foundation on a deposit of clay with a tunnel located underneath the center of the foundation, b) Tunnel diameter varies from 0.5B to 2B at different depth (ranges from 1D to 5D)

3 THREE-DIMENSIONAL FE ANALYSES

3.1 Geometry

The considered FE model is 20 m wide that extends 20 m in the y direction and it is 15 m deep. These dimensions are sufficient to allow for any possible collapse mechanism to develop and to avoid any influence from the model boundaries. Figure 1 shows the geometry of the considered problem. Different cases of tunnels with tunnel diameter, D, varying from 0.5B to 2B, where B is the width of the strip foundation, were considered. The tunnels were assumed to be buried at different depths ranging from 1D to 5D.

3.1.1 Soil Stratigraphy

The Soil layer is assumed to be horizontal throughout the model and so just one borehole is sufficient to describe the soil layer. The ground water table is located well below the foundation level so there is no influence of the water table on the ultimate bearing capacity of the foundation. The analyses were conducted assuming undrained conditions. The unit weight γ_{unsat} = 15 kN/m^3. The Mohr-Coulomb model is selected as the material model. A perfectly-plastic model is a constitutive model with a fixed yield surface, i.e. a yield surface that is fully defined by model parameters and not affected by (plastic) straining. For stress states represented by points within the yield surface, the behaviour is purely elastic and all strains are reversible. The Mohr-Coulomb model involves only five basic parameters: (1) Young's modulus, E = 20,000 kPa, (2) Poisson's ratio, ν = 0.49, (3) Undrained cohesion, C_u = 50 kPa, (4) Undrained friction angle, φ_u = 0°, and (5) Dilatancy angle, ψ = 0°.

3.1.2 Strip Foundation

The 2.0 m wide strip foundation considered in this analysis is located at the middle of the clay deposit. It consists of a 0.5 m thick concrete of unit weight, γ = 25 kN/m^3. The strip foundation was modeled using plate elements from the PLAXIS library with a linear isotropic behaviour. The Young's modulus, E_1 = 30,000,000 kPa, and the Poisson's ratio, ν_{12} = 0.15.

Karl Terzaghi developed the conventional bearing capacity theory in the 1920's and it has been used, with several refinements, ever since. Considering the model shown in Figure 2 below, the failure mechanism contains three zones: 1) A wedge zone that, in essence, becomes part of the footing, 2) A radial zone assumed to be bounded by a log spiral segment and a passive zone. Terzaghi obtained the critical surface by trial and error and presented the results in the following equation (in terms of effective stress) for strip foundations.

$$q_{ult} = c'N_c + \sigma'_D N_q + \gamma' B N_\gamma \qquad [1]$$

Where,

c' = effective cohesion of supporting soil;

N_c, N_q, N_γ = Bearing capacity factors which depend on ϕ';

σ_D' = Effective vertical stress at the base of the foundation;

γ' = Effective unit weight of the supporting soil; and

B = Width of the foundation.

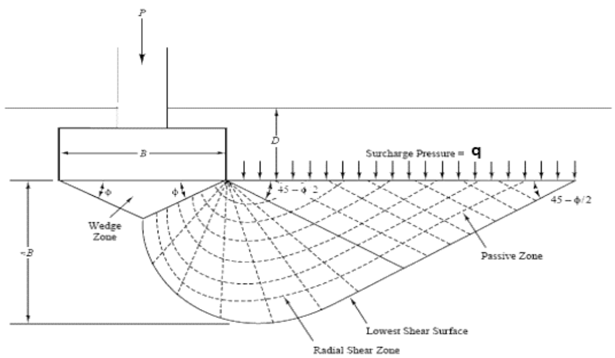

Figure 2 Terzaghi's model for bearing capacity of soils

This formula has been substantially generalized by numerous investigators to account for different footing shapes, depth and load inclination (Cox et al. 1961; Meyerhof 1963; Hansen 1970; De Beer 1970; Vesic 1973), location of the ground water table (Meyerhof 1955), sloping ground surface (Meyerhof 1957; Hansen 1970), mode of shear (Vesic 1963, 1973), inclined or eccentric loading (Meyerhof 1953; Vesic 1973; Taiebat and Carter 2002) and soil compressibility (Vesic 1973).

In the case of a surface strip footing resting on an undrained clay ($\varphi_u = 0^\circ$), the value of the bearing capacity factors N_q and N_γ will be 1 and 0, respectively. Consequently, the ultimate bearing capacity of a footing could be expressed by the following equation:

$$q_{ult} = C_u N_c \qquad [2]$$

Where,

C_u = Undrained cohesion of the soil

$N_c = \pi + 2$

$$q_{allowable} = \frac{q_{ult}}{FOS} \qquad [3]$$

Thus, the allowable bearing capacity of the strip foundation considered in this paper is 102.8 kPa.

3.1.3 The Tunnel Lining

The tunnel lining was assumed to obey the linear elasticity model based on Hook's law of isotropic elasticity. According to the considered cases the tunnel was modeled for lining thicknesses equal to 0.05D. For all cases, the tunnel lining was modeled as concrete with unit weight, γ = 25, elastic modulus, E = 30 GPa and Poisson's ratio, ν = 0.15. The diameter of the considered tunnels ranges from 0.5B to 2B and for each diameter the position of the depth of the tunnel's center was varied from 1D to 5D.

3.1.4 Interfaces

The friction behavior at the soil–tunnel interface was modeled using the five noded interface elements from the PLAXIS's library. The interface elements were assigned material properties similar to that of the interfacing medium. The roughness of the interaction the concrete and the soil was modeled by using a strength reduction factor of 0.85.

By using an interface, node pairs are created at the interface of the structure and the soil, forming a node pair, one node belongs to the structure and the other node belongs to the soil (Figure 3). The interaction between these two nodes consists of two elastic-perfectly plastic springs; one spring to model the gap displacement and the other one to model the slip displacement.

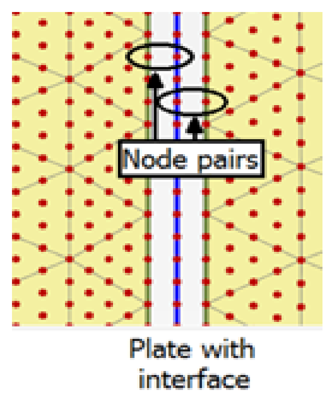

Figure 3 Interfaces.

3.2 Model Conditions

PLAXIS 3D allows for a fully automatic mesh generation procedure, in which the geometry is divided into volume elements and compatible structure elements, if applicable. The mesh generation takes full account of the position of the geometry entities in the geometry model, so that the exact position of layers, loads and structures is accounted for in the finite element mesh. A local refinement will be considered in the strip and tunnel volumes.

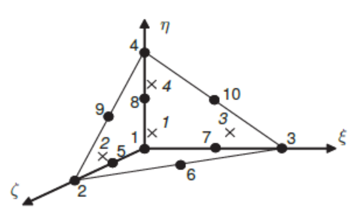

Figure 4 3-D soil elements (10-node tetrahedrons)

The model was built using about 60,000 3D 10-node tetrahedral elements (Figure 4). The average size of the element was approximately 110 mm. The large number of small size elements assured high accuracy of the results at locations where non-linear behavior is anticipated. The load was applied using uniform prescribed displacement applied at the top of the foundation, and the corresponding load was evaluated. Figure 5 shows the generated mesh of the model.

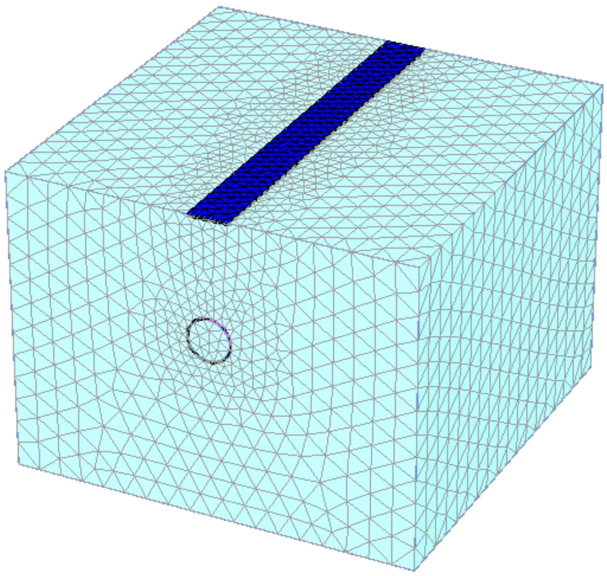

Figure 5 Generated mesh

3.3 Simulation of the Construction Sequence

The staged construction technique was used to simulate the construction process in two phases as follows:

3.3.1 Initial Conditions

The initial in-situ conditions were first generated in this step. The initial conditions comprise the initial geometry configuration and the initial stress state, i.e. effective stresses, pore pressures and state parameters, if applicable.

3.3.2 Construction and Loading the Strip Foundation phase

In this phase the foundation construction was simulated by activating the plate elements representing the strip foundation including the uniform distributed surface loading and the interface elements between foundation and the soil.

3.3.3 Excavation and Construction of the Tunnel

The soil volume corresponding to the inside of the tunnel is deactivated. The plate and the interface corresponding to the tunnel are activated.

3.3.4 Bearing Capacity of the Foundation

To determine the ultimate bearing capacity of the foundation, a surface prescribed displacement of 0.2m is imposed to the strip foundation to visualize the reaction forces.

4 RESULTS AND DISCUSSIONS

A parametric study was conducted to investigate the effect of the tunnel's size, proximity and burial depth on the performance of strip foundations. Also, the effect of the existing structure on the developed moments and thrusts in the tunnel lining was studied.

In the parametric study, the tunnel was assumed to be located just below the centerline of the foundation. In addition, burial depths ranging between 1D to 5D were investigated.

To illustrate some of the characteristics of this Tunnel-Soil-Structure interaction problem the results of the analyses are presented in a normalized form. The normalized displacements, U_c , thrust, T_c , and moment, M_c , are (El Naggar et al., 2008):

$$U_{c,(s)} = \frac{uE_g}{\sigma_v R\left(1 + \nu_g\right)} \quad [4]$$

$$\Delta D_z = \frac{\sigma_v R\left(1 + \nu_g\right)}{\delta D\, E_g} \quad [5]$$

$$T_{c,(s)} = \frac{T}{\sigma_v R} \quad [6]$$

$$M_{c,(s)} = \frac{M}{\sigma_v R^2} \quad [7]$$

Where,

E_g = Soil elastic modulus;
σ_v = Total stress at spring line depth;
R = Radius, centerline of the liner;
ν_g = Poisson's ratio of the ground;

4.1 Effect of the Existing Structure on the Displacements of the Tunnel Lining

Figure 6 shows the normalized vertical change in diameter of the tunnel at the crown location at the ultimate loading capacity of the foundation.
It can be seen from Figure 6 that the normalized vertical change in diameter, ΔD_z , decreases as the burial depth of the tunnel increases. Furthermore, it increases as the diameter of the tunnel increases. Similar trends in

behaviour can also be observed in Figure 7 for the normalized horizontal change in diameter at the springline. It can be noticed that the maximum deformation of the lining is less than the tolerable deformation (less than 1% of the diameter). It remains on the safe side. This behaviour is expected as the tunnel is located away from the foundation. The stress concentration from the foundation does not interfere with the tunneling zone and consequently, less deformation occur.

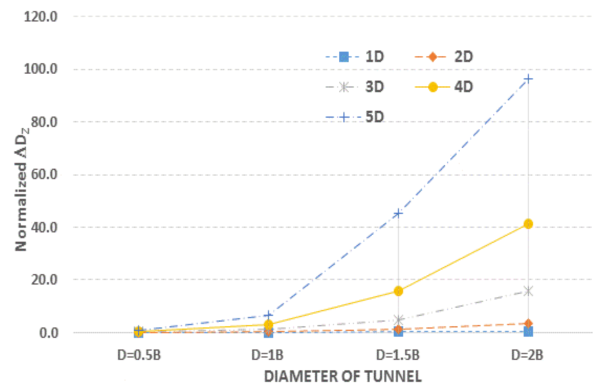

Figure 6 Normalized vertical change in diameter of the tunnel at the crown location

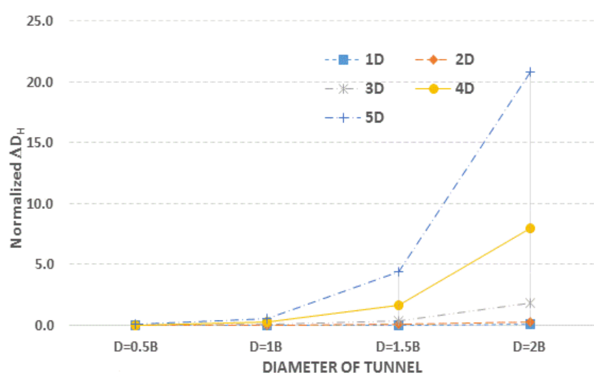

Figure 7 Normalized horizontal change in diameter of the tunnel at the springline location

4.2 Effect of the Existing Structure on the Developed Moments and Thrusts in the Tunnel Lining

Figures 8 and 9 show the normalized thrust at the crown and springline locations, respectively. It can be noticed from Figures 8 and 9 that when the tunnel is located within the overstressed zone (due to the load of the building), the attracted thrusts both at the crown and the springline increase considerably, especially when the tunnel is located at a depth of less than 2D under the centerline of the building. When the tunnel is located at a depth of 3D or more below the foundation, the increase in the thrust almost vanishes.

Figures 10 and 11 present the maximum normalized positive and negative moments at the crown and springline locations, respectively. It can be noticed from both Figures that the value of the moment decreases as the tunnel is located away from the zone of influence (the zone that is overstressed under the foundation). For tunnels located at a depth of 1D below the deep foundation, the maximum positive moment increases by substantially due to the presence of the existing foundation. For tunnels located a deeper depth, the increase in the moment is much less (within less than 10%).

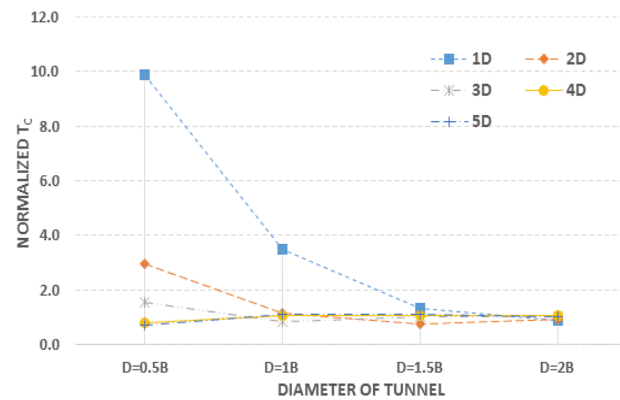

Figure 8 Normalized thrust at the crown of the lining

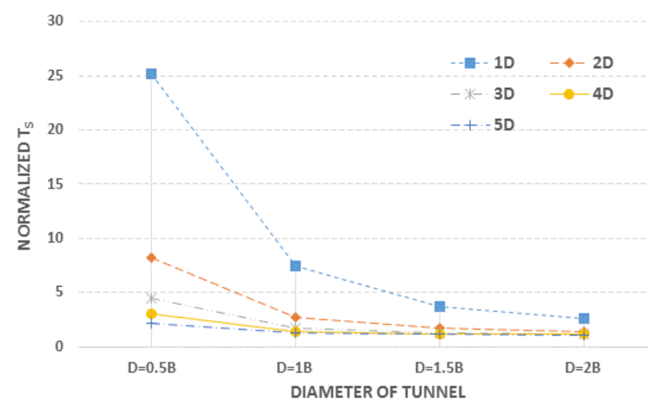

Figure 9 Normalized thrust at the spring line of the lining

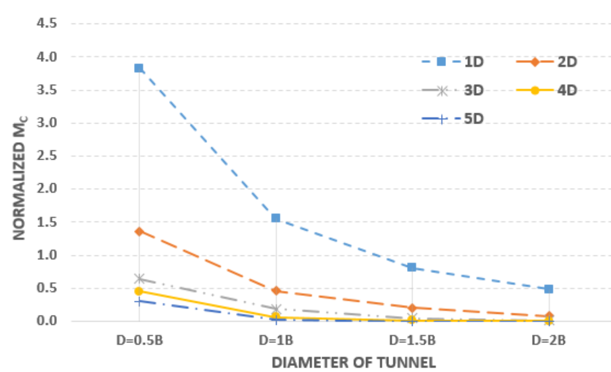

Figure 10 Normalized moment at crown of the lining

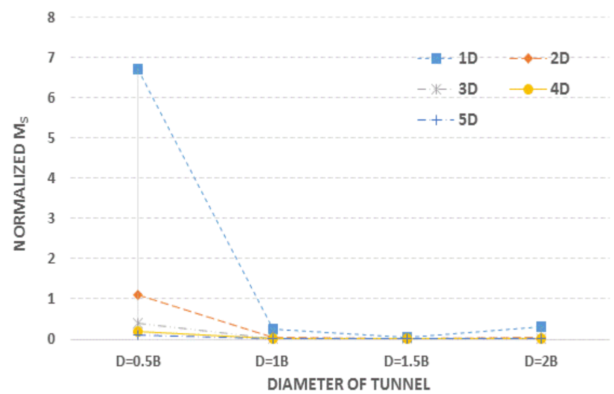

Figure 11 Normalized moment at the spring line of the lining

4.3 Bearing Capacity

Figure 12 shows the ultimate bearing capacity of the strip foundation. The diameter of the tunnel, D, varies from 0.5B to 2B, where B is the width of the strip foundation, and the tunnel's depth ranges from 1D to 5D. It can be noticed that there is slight increase of the load, almost by 4%, when the depth of the tunnel's center is at 1D and its diameter varies between 0.5B to 1B. In the other cases it is almost 2% even less.

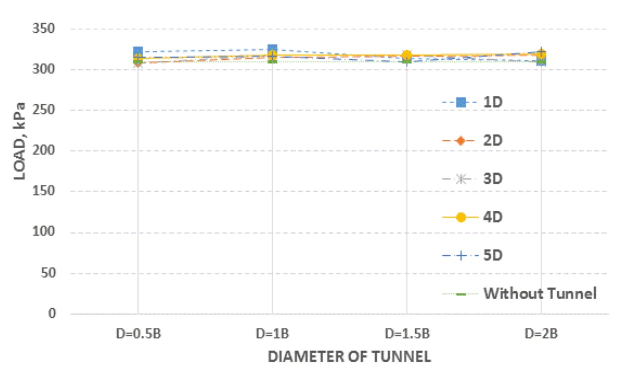

Figure 12 Bearing Capacity at the prescribed displacement 0.2 m

Figure 13 shows the bearing capacity of the strip foundation at the tolerable settlement 35 mm. It can be remarked when the diameter varies from 1D to 2D and at a depth of 1D the load is 10% to 5% more, for the the other cases it is almost the same and the change is unnoticable.

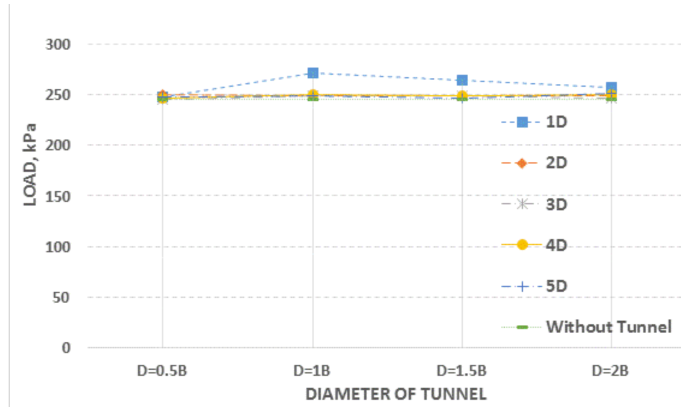

Figure 13 Bearing Capacity at tolerable settlement (35 mm)

5 CONCLUSIONS

An extensive parametric study was conducted to investigate the effect of tunnelling on the performance and capacity of nearby foundations. In addition, the effect of the pre-existing foundations on the forces and deformations developed in the tunnel lining was investigated. The conclusions drawn from this study are summarized as following:

1) When the tunnel is embedded within 2D below the centerline of the foundation (i.e., when the tunnel is located within the overstressed zone), due to the tunnel-foundation interaction the deformations, bending moment and thrusts increase substantially
2) The normalized changes in diameter in the vertical and horizontal directions respectively, decrease as the burial depth of the tunnel increases.
3) The normalized changes in diameter in the vertical and horizontal directions, respectively, increase as the diameter of the tunnel increases.
4) All of the above effect reduces substantially or vanishes when the tunnel is embedded at a depth of 3D or more below the foundation.
5) Presence of the tunnel did not affect the bearing capacity of the foundation due to the high stiffness of the tunnel lining which acted as a support below the foundation. This finding is expected to be different for the case of flexible linings and/or unlined tunnels.

6 REFERENCES

H. El Naggar, S.D. Hinchberger, and K.Y. Lo. A closed-form solution for composite tunnel linings in a homogeneous infinite isotropic elastic medium. *Can. Geotech. J. 45: 266-287.*

Einstein, H.H., and Schwartz, C.W. 1979. Simplified analysis for tunnel support. *Journal of the Geotechnical Engineering Division 105 (GT4): 499-518.*

S. Benmebarek, S. Benmoussa, L. Belounar, N. Benmebarek. Bearing capacity of shallow foundation on two clay layers by numerical approach. *Geotech Geol Eng (2012) 30:907–923.*

E. Sadrossadat, F. Soltani, S. M. Mousavi, S. M. Marandi and A. H. Alavi. A new design equation for prediction of ultimate bearing capacity of shallow foundation on granular soils. *Journal of Civil Engineering and Management, 19:sup1, S78-S90.*

PLAXIS 3D 2013. Tutorial manual, Reference manual, Material models manual and Scientific manual.

Vesic, A. S. 1973. Analysis of ultimate loads of shallow foundations. *Journal of the Soil Mechanics and Foundations Division 99(1): 45–73.*

Meyerhof, G. G. 1963. Some recent research on the bearing capacity of foundations. *Canadian geotechnical Journal 1(1): 16–26.*

K. Terzaghi and R. B. Peck. Soil mechanics in engineering practice.

B. Das. Principles of foundation engineering.

J. E. Bowles. Foundation analysis and design.