EFFECTS OF TUNNELING ON PILED FOUNDATIONS IN CLEAN SANDS USING FINITE ELEMENT METHOD AND SENSITIVITY ANALYSES

TEMİZ KUMLAR İÇERİSİNDE YER ALAN TÜNELLERİN KAZIKLAR ÜZERİNDE ETKİSİNİN SONLU ELEMANLAR YÖNTEMİ İLE ARAŞTIRILMASI VE DUYARLILIK ANALİZLERİ

Cansu GÜNDAY URAS

PROF. DR. Berna UNUTMAZ

Supervisor

Submitted to

Graduate School of Science and Engineering of Hacettepe University

as a Partial Fulfillment to thee Requirements

for the Award of the Degree of Master of Science

in Civil Engineering

2021

To my mom, dad and little sister

with love

ABSTRACT

EFFECTS OF TUNNELING ON PILED FOUNDATIONS IN CLEAN SANDS USING FINITE ELEMENT METHOD AND SENSITIVITY ANALYSES

Cansu GÜNDAY URAS

Master of Science, Civil Engineering

Supervisor: Prof. Dr. Berna UNUTMAZ

February 2021, 77 pages

Infrastructures are getting more and more popular and preferable since the urban areas are getting crowded. Ground surface is occupied by high rise buildings, hospitals, social facilities and etc. Therefore, it is essential to use the underground space for different purposes. Some of these purposes are transportation, water systems, sewer systems, and electric lines. Tunneling is a way to travel underground and it is an alternative to ease traffic in cities. Therefore, tunnels are constructed under structures with shallow or deep foundations. For this reason, effects of tunneling on a foundation become a really important topic for civil engineers.

In this manner, many researchers studied the aforementioned topic. They tried to find an answer to the big question; what are the effects of tunneling on piled foundation? In this study, the answer of the question is tried to be found by using analytical methods. Different parameters are specified according to the former researches and engineering judgements. These parameters are the ratio of location of the tunnel in x-direction to tunnel diameter (x/D), depth of the tunnel to tunnel diameter (z/D) and deformation modulus of the clean sand layer. The tunnel is excavated at once as a TBM tunnel. It is important to note that effects of tunneling on existing structures are examined in this thesis.

According to the 2-Dimensional plane strain model created for the problem, finite element models are created and a series of analyses are conducted by using PLAXIS 2D software. Results are tabulated and evaluated. In the light of the results, a simplified equation is suggested in the scope of this study.

According to the results, outcomes are evaluated and following statements are presented. Additional settlements of the foundation due to tunneling are decreased while tunnel is located further than the foundation center axis, and with increasing ‘x/D’ and ‘z/D’, and deformation modulus. Tunneling has put extra moments on piles, especially on the further piles where tunnel is located. Increasing deformation modulus has a decreasing effect on extra pile moment because the displacement is decreasing with increasing deformation modulus of the soil.

With the additional settlement values obtained from finite element method using Plaxis 2D software, an equation is proposed using Maximum Likelihood Estimation. This equation requires tunnel location in x-direction, tunnel depth, tunnel diameter and deformation modulus of the soil in order to estimate additional settlement due to tunneling. It should be kept in mind that proposed equation is only valid for cohesionless soils.

Keywords: settlement due to tunneling, settlement, tunneling, piled foundation, deep foundation

ÖZET

TEMİZ KUMLAR İÇERİSİNDE YER ALAN TÜNELLERİN KAZIKLAR ÜZERİNDE ETKİSİNİN SONLU ELEMANLAR YÖNTEMİ İLE ARAŞTIRILMASI VE DUYARLILIK ANALİZLERİ

Cansu GÜNDAY URAS

Yüksek Lisans, İnşaat Mühendisliği

Tez Danışmanı: Prof. Dr. Berna UNUTMAZ

Şubat 2021,77 sayfa

Kentsel bölgelerdeki nüfus artışıyla beraber yeraltı yapıları gün geçtikçe daha popüler ve tercih edilir olmaktadır. Yer yüzeyi; yüksek katlı binalar, hastaneler ve diğer hizmet binaları ile işgal edilmektedir. Bu nedenle yeraltı yapıları değişik amaçlarla kullanılmaktadır. Bu amaçlardan bazıları; ulaşım, su sistemleri, kanalizasyon yapıları ve elektrik hatları olarak sıralanabilir. Tüneller yer altı ulaşımı için bir yöntem olmakla beraber trafik sorununa da bir alternatif olmaktadır. Bu sebeple yapıların altına tünel inşa edilmesi gerekebilmektedir. Bu yapıların temelleri yüzeysel olabileceği gibi derin (kazıklı) de olabilmektedir. Bu nedenle tünel yapılarının temeller üzerindeki etkileri inşaat mühendisliği açısından önemli bir konu haline gelmektedir.

Bu konuda farklı araştırmacılar detaylı çalışmalar yürütmüştür. Çoğu araştırmacı tek bir büyük sorunun cevabını aramıştır: tünellerin kazıklı temellere etkileri nelerdir? Bu tez çalışmasında da çeşitli analitik yöntemler kullanılarak ilgili sorunun yanıtı aranmaktadır. Önceki çalışmalar ve mühendislik hükmü hesaba katılarak farklı parametreler oluşturulmuştur. Bu parametreler, tünelin temele olan x-aksı yönündeki yerleşiminin tünel çapına oranı (x/D), tünelin derinliğinin tünel çapına oranı (z/D) ve temiz kum zeminin deformasyon modülü değeri olarak belirlenmiştir. Tünel TBM tekniği kullanılarak tek seferde açılacak şekilde modellenmiştir.

Probleme yönelik oluşturulan model uyarınca, 2 boyutlu plane strain sonlu element modeli yaratılmış ve PLAXIS 2D yazılımı yardımıyla çeşitli analizler yürütülmüştür. Sonuçlar özetlenerek değerlendirilmiştir. Bu çalışma kapsamında ilgili sonuçlar ışığında basitleştirilmiş bir denklem sunulmaktadır.

Sonuçlara göre çeşitli çıkarımlar yapılmış ve sunulmuştur. Tünelin, temelin merkez aksından uzaklaşması ile tünel kaynaklı meydana gelen oturma değerleri düşmektedir. Aynı şekilde, artan x/D ve z/D oranları ile de tünel kaynaklı meydana gelen oturma değerleri düşmektedir. Tünel inşası temel kazıklarında ekstra momentler oluşmasına neden olmaktadır. En çok etkilenen kazıklar ise tünele en uzak konumlanan kazıklar olmaktadır. Deformasyon modülünün artışı ise tünel inşasından kaynaklı oturmaları azalttığından kazıklarda oluşan ekstra moment değerini düşürmektedir.

Tünel inşası sebebiyle oluşan ekstra oturma değerlerinin Plaxis 2D yazılımı kullanılarak elde edilmesinin ardından maksimum olabilirlik kestirimi (Maximum Likelihood Estimation) kullanılarak bir denklem oluşturulmuştur. Bu denklemde kullanılmak üzere tünelin x-yönündeki yerleşimi, tünel derinliği, tünel çapı ve zeminin deformasyon modülü değerlerine ihtiyaç duyulmaktadır. Dikkat edilmesi gereken en önemli nokta ise bu denklemin yalnızca kohezyonsuz zeminler için geçerli olmasıdır.

Anahtar Kelimeler: tünel inşası kaynaklı oturma, oturma, tünel inşası, kazıklı temel, derin temel

TEŞEKKÜR

Lisansüstü eğitimimin başından itibaren gerek yönlendirmeleri gerekse sağladığı imkanlar ile bana yardımcı olmaktan da ötesini sunan, tez aşamasında bana her türlü imkanı sağlayan ve her ihtiyacım olduğunda beni dinleyip yol gösteren değerli hocam Sayın Prof. Dr. Berna UNUTMAZ'a

Eğitim sürecimde benim için zaman yaratıp imkan sağlayan eski çalışma arkadaşım ve saygıdeğer şefim Mehmet AS'a,

Olumsuz duyguların tuzağına her düştüğümde enerjisiyle beni kendime getiren canım arkadaşım İpek TEKİN'e,

Bana olan inançlarını her fırsatta yılmadan yineleyen, yoluma yoldaş olan, bana benliğimi kazandıran, karşılıksız sevgi ve saygıları üzerimden eksik olmayan, benim muhteşem ailem; Ayşen, Hürkan ve Pınar GÜNDAY'a,

Ve tabi ki destekleriyle ve gerçekçi yorumlarıyla beni mesleğime ve eğitimime sıkıca bağlayıp devam etmeme yardımcı olan, en büyük destekçim, suç ortağım, sevgili eşim Gökhan URAS'a,

Sonsuz teşekkürlerimi sunarım...

Cansu GÜNDAY URAS

Şubat 2021, Ankara

TABLE OF CONTENT

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

Countries are getting more crowded day by day causing urbanization to increase. In urban areas, structures are constructed side by side to use the free space efficiently. With the growth of the population and lack of available construction area, infrastructures; like tunnels, water and wastewater systems, gain prominence in the current century, especially in urban areas like city centers. Therefore, inevitably, infrastructures are constructed under the buildings with shallow or deep foundations. For this reason, effects of tunneling on a foundation become a really important topic for civil engineers.

1.1 Objective and Scope

Considering the real life, many researchers studied the aforementioned topic. They tried to find an answer to the big question; what are the effects of tunneling on piled foundation? In this study, the answer of the question is tried to be found by using analytical methods.

The objective of this study is to assess the effect of tunnels on piled foundations. As this is a very wide area, the study is limited to the tunnels constructed in sandy soils under piled foundations. In the scope of this study is , there is a existing structure (or building) with a piled foundation and then the tunnel is excavated beneath it. The tunnel is excavated at once like a TBM tunnel. Within this scope, different parameters are specified according to the former researches and engineering judgements. These parameters are the stiffness of the soil (in terms of modulus of elasticity, E), the diameter of the tunnel (D), the depth of the tunnel (z), the length of the piles (L) and the location of the tunnel with respect to the piled foundation (x). These parameters were not utilized independently but in terms of z/D and x/D whose details will be discussed in the following chapters. Parametric sensitivity analyses are performed using the finite element methodology with the parameters listed above are conducted. Results are tabulated and evaluated and discussed in detail. In the light of the results, a simplified equation to calculate the tunnel-induced settlement of the foundation is suggested at the end.

After this brief introduction in Chapter 1, previous researches related to the main topic are presented in details in Chapter 2.

Analysis models, parameters used in the analyses and analysis steps are discussed in Chapter 3.

Results and discussion with graphs which conclude the analysis results are given in Chapter 4.

An overall evaluation of this study is stated and further studies are discussed in the last chapter, Chapter 5.

CHAPTER 2

A SURVEY ON THE EFFECTS OF TUNNELING ON PILED FOUNDATIONS

In this chapter, a comprehensive review of literature about the effects of tunneling on piles and piled foundations is presented. First, influence zones are mentioned. Second, centrifuge test research results and comparison of these results with empirical approaches, analytical methods and case studies are given. Last but not least, some parametric studies are presented.

2.1 Influence Zones

There are various types of investigation and articles in literature about effects of tunneling on piled foundation. Some of the researchers like Selemetas (2006), Kaalberg et al. (2006) and Cham (2016) defined three influence zones under the foundation.

Selemetas et al. (2006) studied the of full-scale piles' response to tunneling induced movements thanks to the construction of the CTRL (Channel Tunnel Rail Link) in the UK. By using the displacement occurred in piles, generalized form of the influence zones is proposed (Figure 2-1).

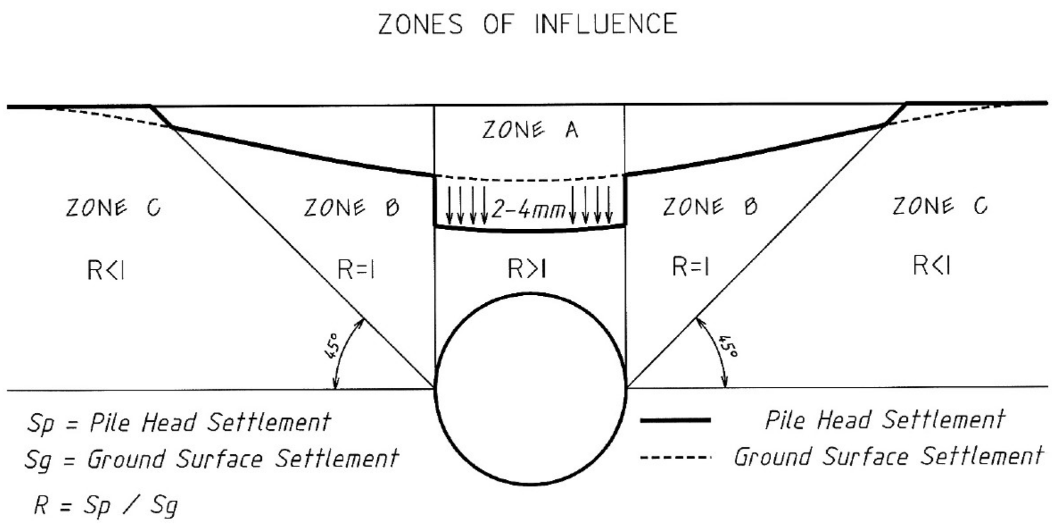


Figure 2-1: Zones of influence of piled settlement due to Earth Pressure Balance (EPB) shield tunneling in London Clay. (Selemetas et al., 2006)

Following results are found after this research.

- Settlements on ground surfaces are higher by 2-4mm when piles located in Zone A .
- Piles located in Zone B settled similar with the ground surface.
- Settlements on ground surfaces are less when piles located in Zone C.
- In Zone A, piles experienced a decrease in their base loads during tunneling with differential pile settlement.
- In Zone B and C, piles experienced very little changes in their base loads.

Kaalberg (2006) states that piles with toes founded in (Figure 2-2);

i. Zone A: settle nearly equal or more than surface-level subsidence.
ii. Zone B: settle approximately equal to the ground surface.
iii. Zone C: settle significantly less than ground level.

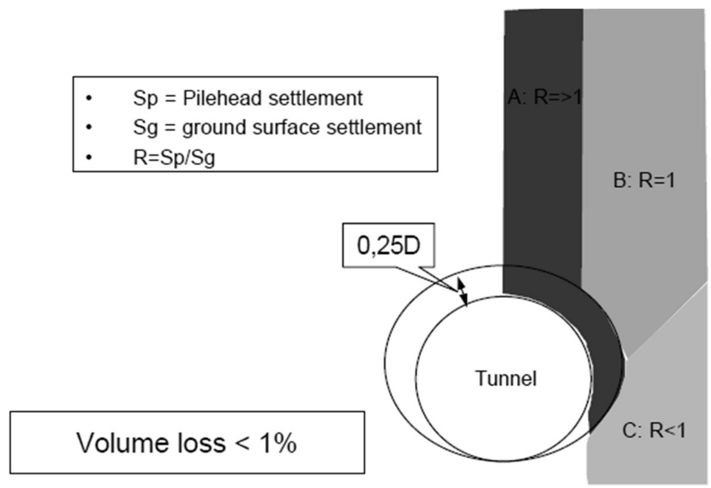


Figure 2-2: Pile Toe Influence Zones for Volume Loss <1% (Kaalberg, 2006)

Other results are stated in the related research as follows.

i. A circular cut off zone of 0.25*D has to be acknowledged for a volume loss of app. < 1%. For larger volume losses, the width of the cut off zone has to be increased.
ii. The inclination of the settlement trough, both in longitudinal as in perpendicular direction is steeper than was expected and predicted by FEM analysis based the simulation of the tunneling process by concentric contraction. This means that the influence area is smaller than predicted, but the risks for nearby foundations will be larger. The steep settlement contours result in high relative rotations perpendicular to the tunnel axis and the high rate of the settlements results in a steep longitudinal trough causing high rotations along the tunnel axis.

Cham (2016) reported that pile toes in (Figure 2-3);

i. Zone 1: settle more than the ground surface.
ii. Zone 2: settle up to ground surface settlement.
iii. Zone 3: settle less than ground surface settlement.

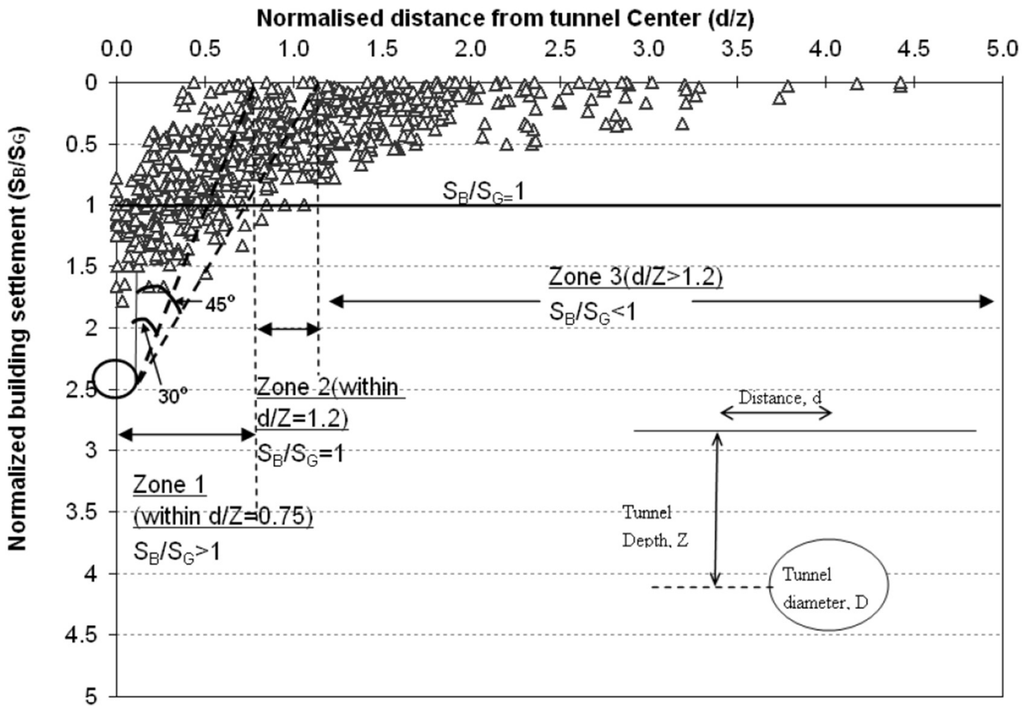


Figure 2-3: Influence Zone of Tunneling (Cham, 2016)

Although Dias (2015) states that there are numerous studies reported that pile settlement/ground settlement is depended on pile location with respect to tunnel, it is also stated in the article that whether and how greenfield displacements are related to displacements in presence of piles are arguable.

2.2 Centrifuge Test Researches, Analytical Methods and Case Studies

There are several other researchers in literature who studied effects of tunneling on piled foundations. Grant et al. (2000) designed a centrifuge model and compared the test results with the empirical approaches.

Gaussian distribution for transverse settlement of ground surface during tunneling is given in the article as;

$$S_v = S_{v,max}\, e^{\frac{-x^2}{2i^2}} \qquad \text{Equation 2-1}$$

where

S_v : settlement

x : distance from the tunnel centerline in the transverse direction

$S_{v,max}$: settlement at x=0

i : distance from the centerline to the point of inflexion of the curve

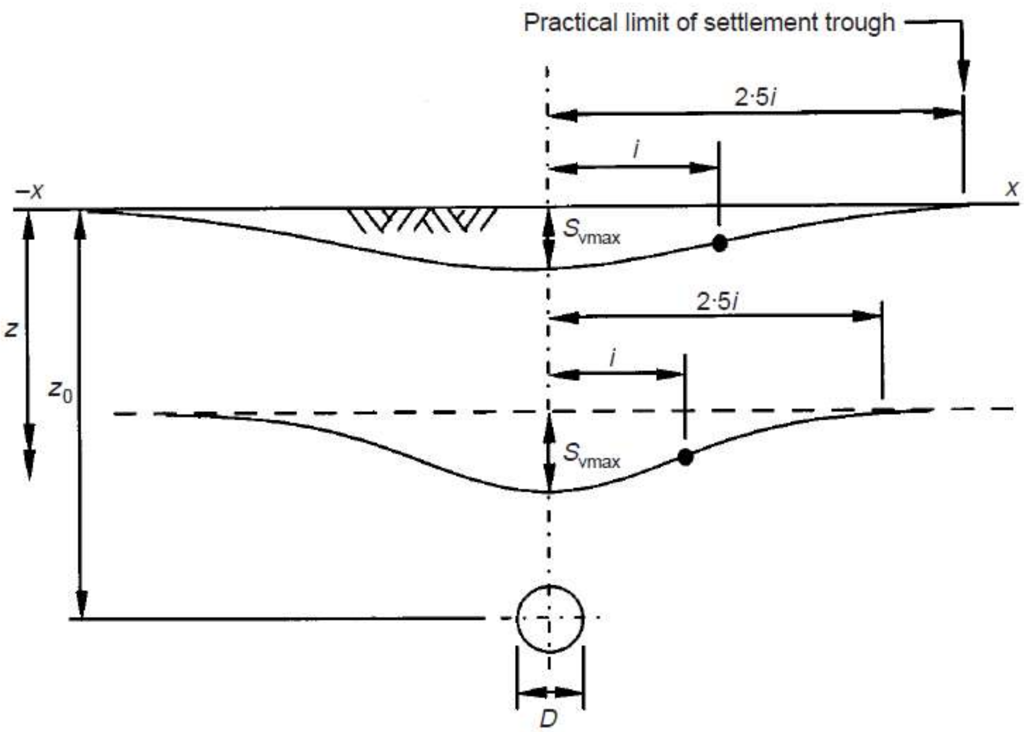


Figure 2-4: Gaussian form of Settlement profiles (Grant et al., 2000)

After several indications and equation modifications given by different researchers, horizontal movements is given as follows as an alternative.

$$S_h = \frac{S_{v,x}}{1+\frac{0.175}{0.325}*z_0} \qquad \text{Equation 2-2}$$

The centrifuge model is given in Figure 2-5 and 2-6.

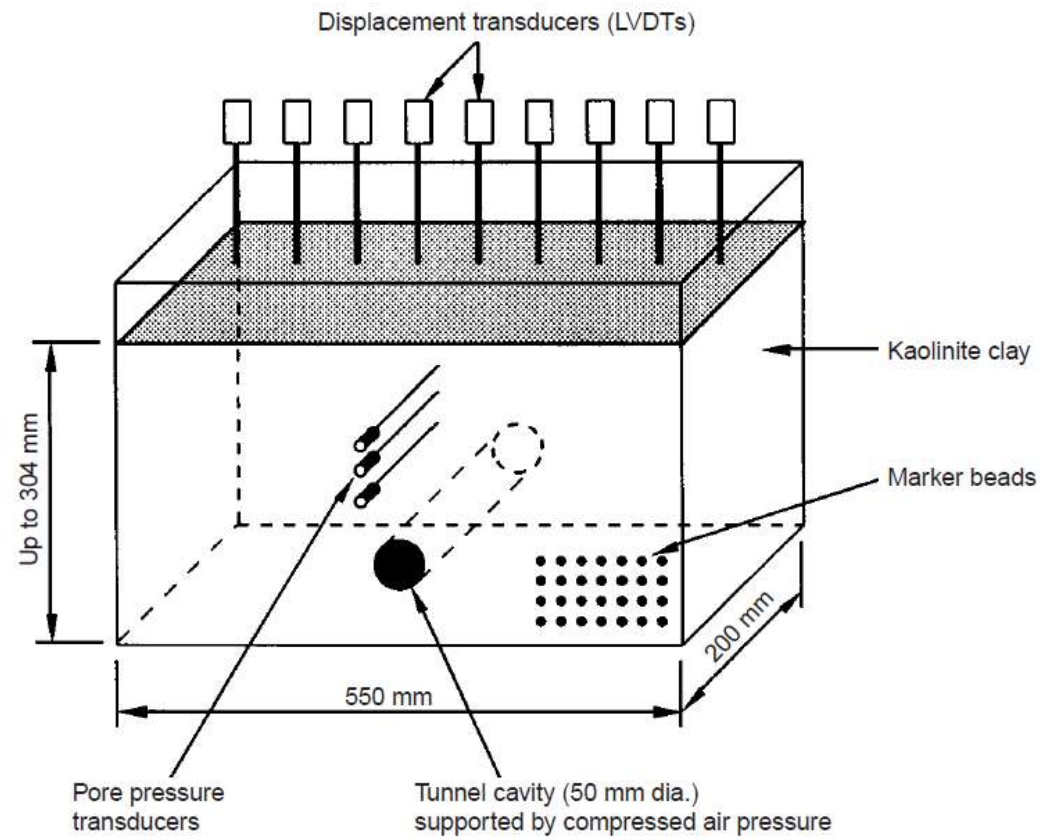


Figure 2-5: A typical plane strain centrifuge model (Grant et al., 2000)

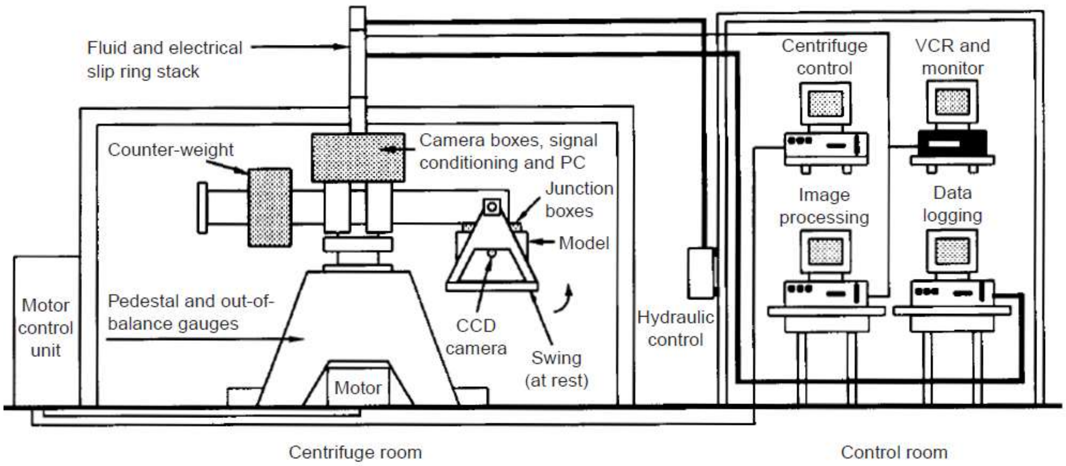


Figure 2-6: Schematic diagram of geotechnical centrifuge model-testing facility at City University, London (Grant et al., 2000)

As a result of the conducted tests, Gaussian distribution represents both surface and subsurface settlement except within about 0.5D of the tunnel crown.

According to Jacobsz et al. (2005), stress relief occurs in the ground while tunneling causing a reduction in the magnitude of loads that can be sustained on pile bases. Since base load reduces, pile shaft loads have to be increasing in order to ensure equilibrium. In addition to that, a small amount of differential settlement is observed between the pile and surrounding ground. Once the maximum skin friction capacity has been fully mobilized, rapid pile settlement follows. Piles with their bases outside the zone of influence (Figure 2-7) did not suffer large settlements even at volume losses up to 10%. Pile groups behave in a similar fashion to volume loss than individual piles. Load transfer from one pile to another within a group only occurs once the shaft capacity of a given pile has been mobilized causing its settlement to become significant. In the pile groups investigated, this usually occurred at large volumes which are undesirable in practice.

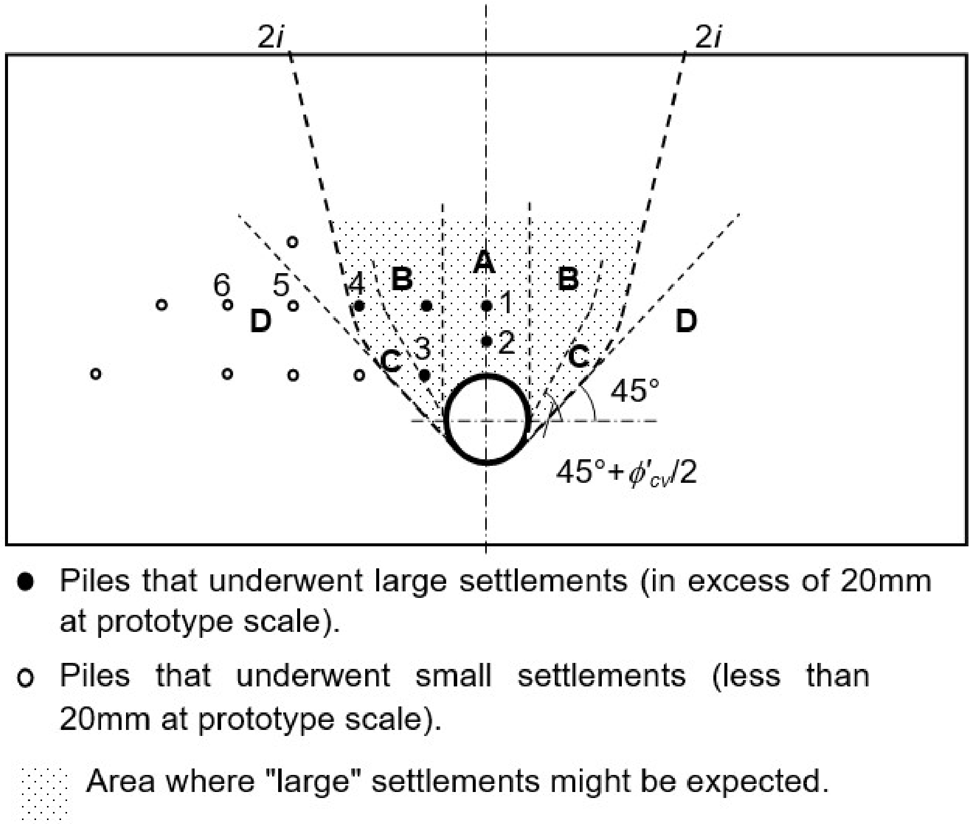


Figure 2-7: Zone of influence around tunnel in which potential for large pile settlements exists (Jacobsz et al., 2005)

Note: *"i"* refers to the distance from the tunnel center line to the inflection point on the Gaussian surface settlement trough.

Ng et al. (2012) conducted several centrifuge tests and as a result, it is stated that the relative location of a tunnel to the pile and cover-to-diameter ratios (C/D) of tunnels are main parameters considering the pile settlement induced by twin tunneling.

A closed form solution to estimate tunneling-induced ground deformations proposed by Loganathan et al. (2001) is given in Eqn. 2-3, 2-4 and 2-5.

$$U_{z=0} = \varepsilon_0 \cdot R^2 \cdot \left(\frac{4H(1-\nu)}{H^2+x^2}\right) \cdot e^{-\left[\frac{1.38x^2}{(H*cot\beta+R)^2}\right]}$$

Equation 2-3

$$U_z = \varepsilon_0 \cdot R^2 \cdot (-\frac{z-H}{x^2+(z-H)^2} + (3-4\nu) \cdot \frac{z+H}{x^2+(z-H)^2} - \frac{2z(x^2-(z+H)^2)}{(x^2+(z+H)^2)^2} \cdot e^{-\left[\frac{1.38x^2}{(H*cot\beta+R)^2}+\frac{0.69z^2}{H^2}\right]}$$

Equation 2-4

$$U_x = -\varepsilon_0 \cdot R^2 \cdot x \cdot (\frac{1}{x^2+(H-z)^2} + \frac{(3-4\nu)}{x^2+(H+z)^2} - \frac{4z(z+H)}{(x^2+(H+z)^2)^2} \cdot e^{-\left[\frac{1.38x^2}{(H*cot\beta+R)^2}+\frac{0.69z^2}{H^2}\right]}$$

Equation 2-5

where

$U_{z=0}$: ground surface settlement

U_z : sub-surface settlement

U_x : lateral soil movement

R : tunnel radius

z : depth below ground surface

H : depth of tunnel axis level

v : Poisson's ratio of soil

ε_0 : average ground loss ratio

x : lateral distance from tunnel centerline

β : limit angle = (45+φ/2)

φ: angle of shearing resistance

The equations given require only Poisson's ratio estimation of the soil and allow quick estimation of ground deformations. Numerical comparison (GEPAN) of the analytical solution is conducted using FLAC3D and the results come out reasonable (Figure 2-8).

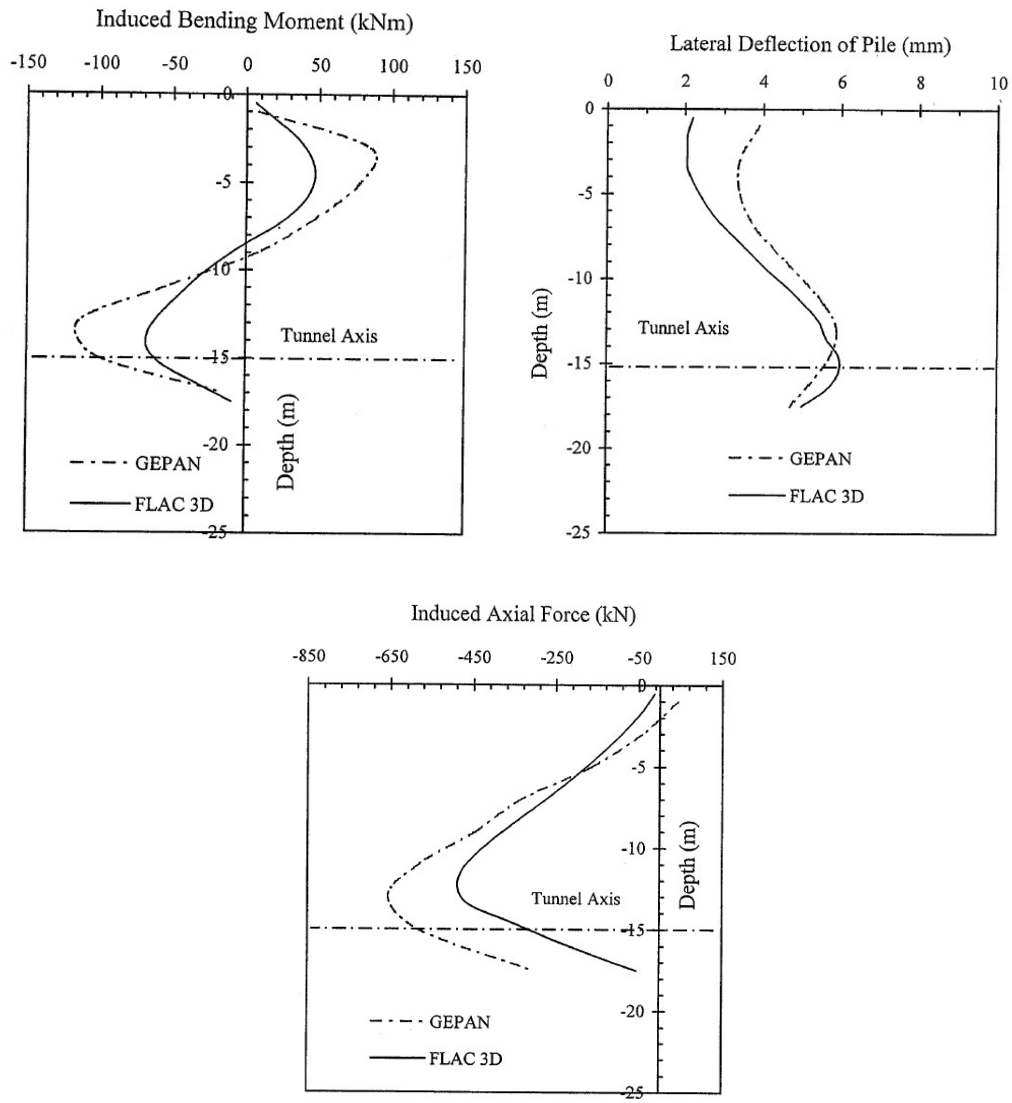


Figure 2-8: Numerical comparison of the analytical solution (Loganathan et al., 2001)

Marshall et al. (2014) proposed a revised analytical study called '*a new method'* (Eqn. 2-6&2-7) of experimental data of Marshall (2012).

$$p'_{0,mod} = \frac{p'_{mid}}{p'_{0,pile}} * p'_{0,tun} \qquad \text{Equation 2-6}$$

where

$p'_{0,mod}$: modified pressure

p'_{mid} : confining pressure half-way between pile tip and tunnel lining

$p'_{0,pile}$: confining pressure at pile tip

$p'_{0,tun}$: confining pressure at tunnel depth

$$R_{Q,S} = \frac{Q_{V_I}}{Q_0} = \frac{q_{b,V_I} * D_p + 4\overline{\tau_{S,V_I}}L}{q_{b,0} * D_p + 4\overline{\tau_{S,0}}L} \qquad \text{Equation 2-7}$$

where

$R_{Q,S}$: pile capacity reduction factor including effect on pile shaft

Q_{VI} : total load capacity of pile after tunnel volume loss

Q_0 : total load capacity of pile before tunnel volume loss

$q_{b,VI}$: reduced end-bearing capacity of pile after tunnel volume loss

$q_{b,V0}$: end-bearing capacity of pile before tunnel volume loss

D_p : pile diameter

$\overline{\tau_{S,V_I}}$: average shear stress on pile shaft after tunnel volume loss

$\overline{\tau_{S,0}}$: average shear stress on pile shaft before tunnel volume loss

L : embedded pile length

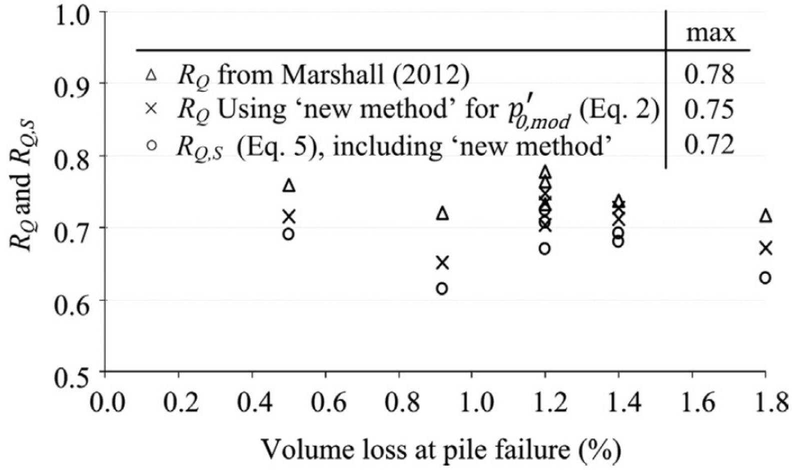


Figure 2-9: Comparison with Marshall (2012) (Marshall et al., 2014)

Note that each data set represents an individual centrifuge experiment including material properties, geometrical conditions, known tunnel volume loss at which pile failed. As it can be seen from Figure 2-9, *The New Method* provides slightly more conservative approach.

Simplified analysis methods are stated by Kitiyodom et al. (2005) and Huang et al. (2009). Results obtained from these simplified methods are compared with the results of finite difference programs and good agreements are achieved.

Kitiyodom et al. (2005) proposed an equation that is given in Eqn. 2-8.

$$[C + K_r + K_p]\{w\} = [K]\{w\} = [C]\{w_0\} \qquad \text{Equation 2-8}$$

where

[K] : global stiffness of the piled raft system

[C]$\{w\}$: nodal forces acting on the piled raft induced by the ground movements.

After the presentation of this equation, Kitiyodom et al. (2005) compared the results with previous researches.

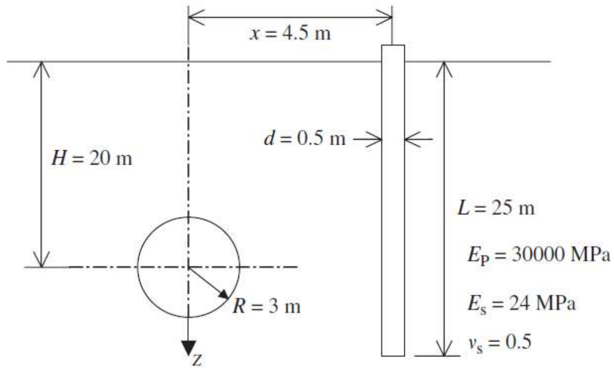


Figure 2-10: Analyzed problem of a single pile case (Kitiyodom et al., 2005)

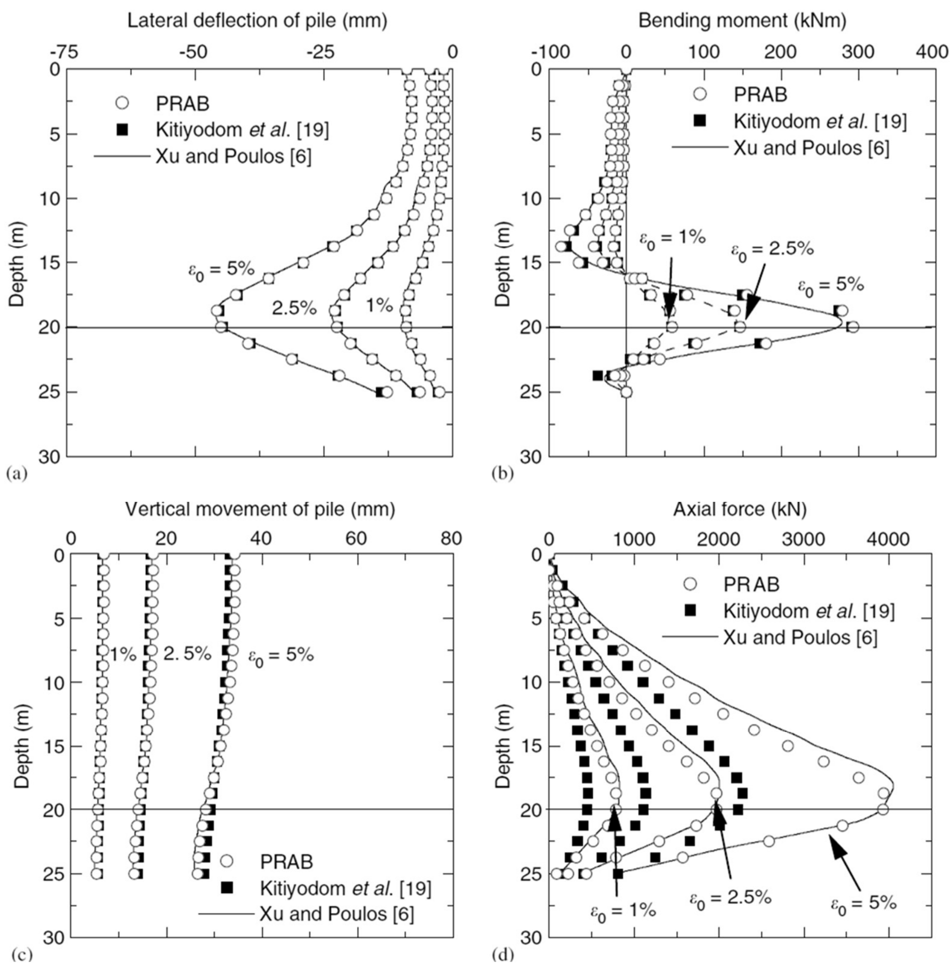


Figure 2-11: Computed responses of single pile: (a) lateral deflection; (b) bending moment; (c) vertical movement; and (d) axial force (Kitiyodom et al., 2005)

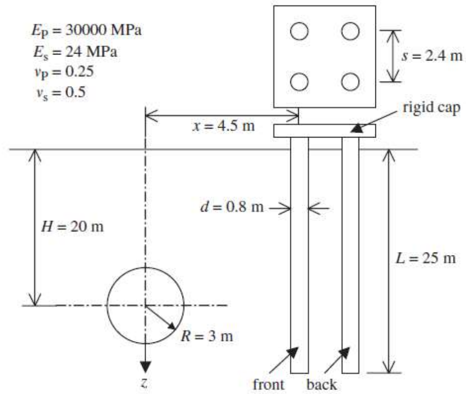


Figure 2-12: Analyzed problem of pile group case (Kitiyodom et al., 2005)

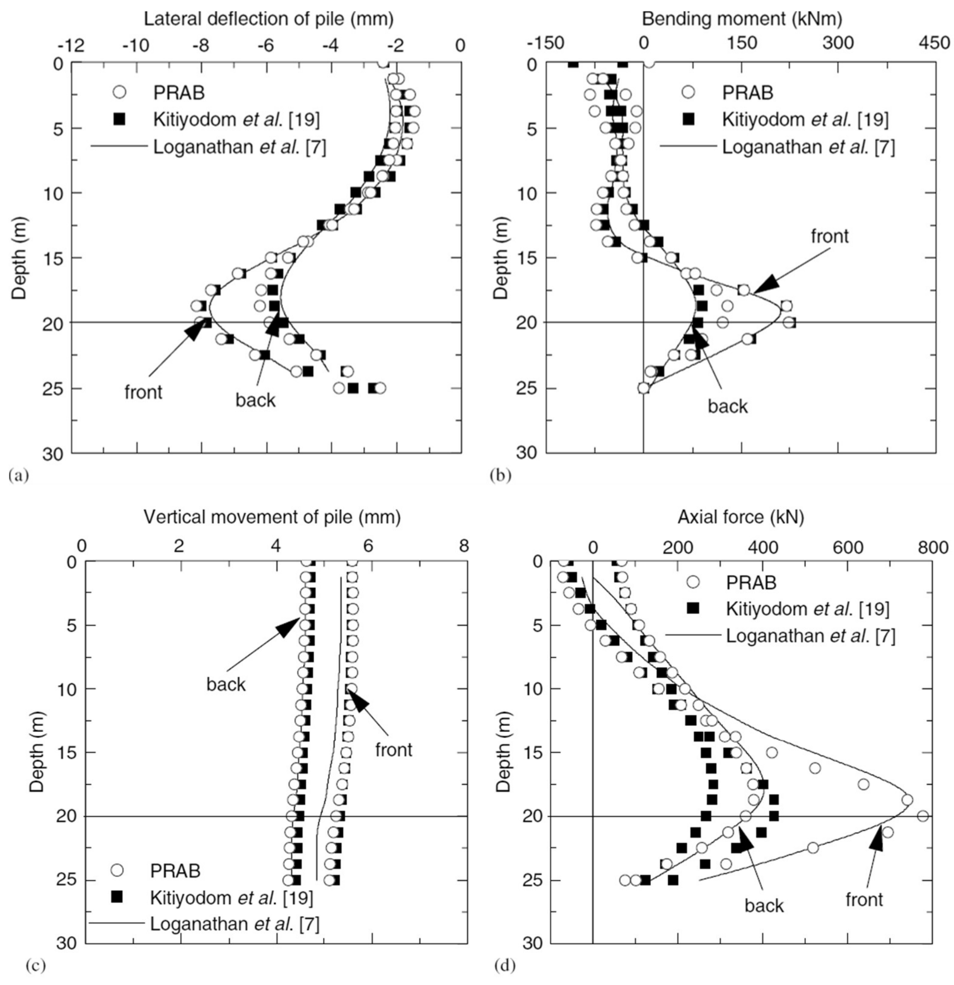


Figure 2-13: Computed responses of pile group: (a) lateral deflection; (b) bending moment; (c) vertical movement; and (d) axial force (Kitiyodom et al., 2005)

Chen et al. (1999) and Basile (2013) present and efficient and practical two-stage procedure in order to estimate the pile deformations, lateral and axial loads, and moments.

Chen et al. (1999) ended up with the factors which affects tunneling induced movements such as soil strength tunnel geometry, pile diameter, ratio of pile length to tunnel cover depth and ground loss ratio. Chen et al. (1999) also proposed equations related with the lateral and axial responses of the pile (Eqn. 2-9 to 2-13).

Lateral Response:

$$M_{max} = M_b * k_{c_u}^M * k_d^M * k_{L_p/h}^M \qquad \text{Equation 2-9}$$

$$\rho_{max} = \rho_b * k_{c_u}^\rho * k_d^\rho * k_{L_p/h}^\rho \qquad \text{Equation 2-10}$$

Axial Response:

$$+P_{max} = +P_b * k_{c_u}^{+P} * k_d^{+P} * k_{L_p/h}^{+P} \qquad \text{Equation 2-11}$$

$$-P_{max} = -P_b * k_{c_u}^{-P} * k_d^{-P} * k_{L_p/h}^{-P} \qquad \text{Equation 2-12}$$

$$\nu_{max} = \nu_b * k_{c_u}^\nu * k_d^\nu * k_{L_p/h}^\nu \qquad \text{Equation 2-13}$$

where

M_{max}: Maximum bending moment in kN.m

ρ_{max}: Maximum lateral deflection in mm

$+P_{max}$: Maximum compressive force in kN

$-P_{max}$: Maximum tensile force in kN

ν_{max}: Maximum pile head settlement in mm

M_b, ρ_b, $+P_b$, $-P_b$, v_b: basic values of bending moment, lateral deflection, compressive force, tensile force and pile head settlement, respectively as shown in Figure 2-14.

$k^M_{c_u}$, $k^\rho_{c_u}$, $k^{+P}_{c_u}$, $k^{-P}_{c_u}$, $k^\nu_{c_u}$: Correction factors for undrained shear strength of soil (Figure 2-15)

k^M_d, k^ρ_d, k^{+P}_d, k^{-P}_d, k^ν_d: Correction factors for pile diameter (Figure 2-16)

$k^M_{L_p/h}$, $k^\rho_{L_p/h}$, $k^{+P}_{L_p/h}$, $k^{-P}_{L_p/h}$, $k^\nu_{L_p/h}$: Correction factors for pile length to tunnel axis ratio (Figure 2-17)

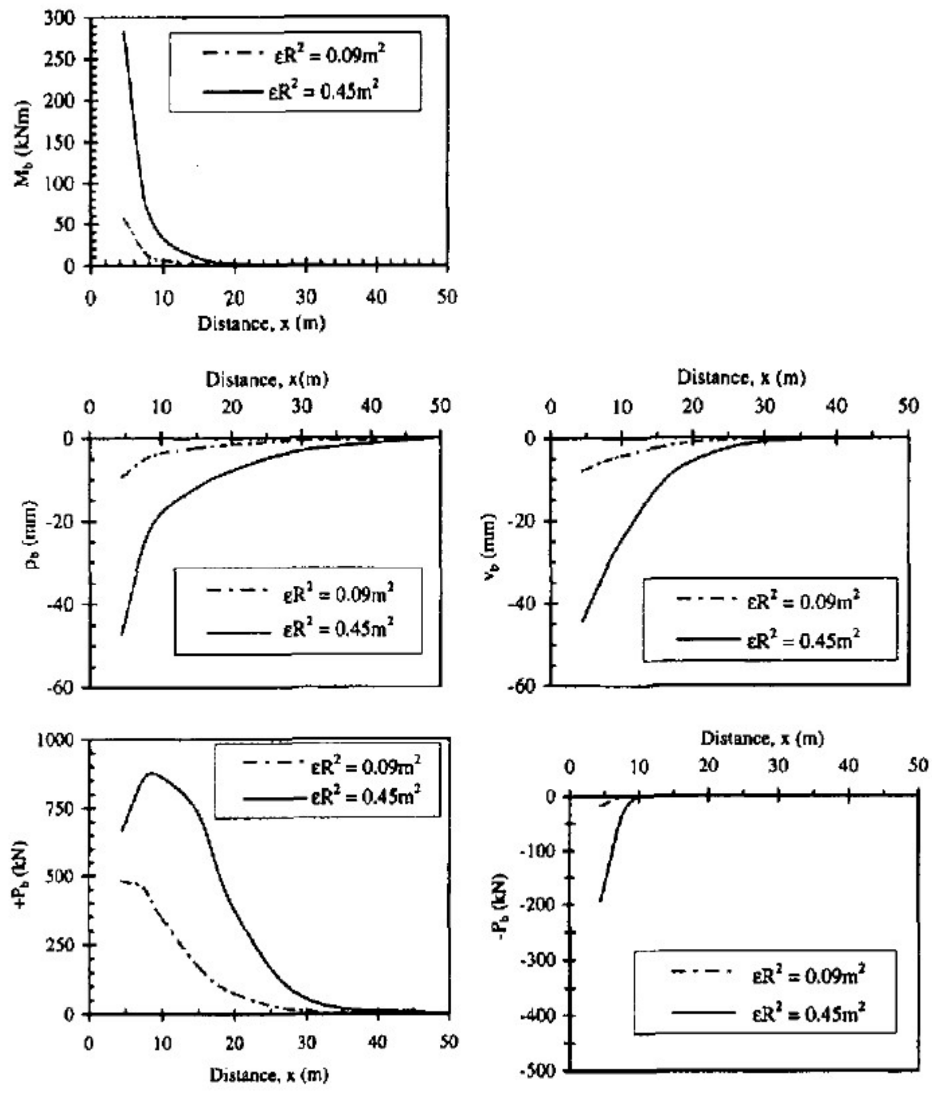


Figure 2-14: Basic values of bending moment, lateral deflection, compressive force, tensile force and pile head settlement (Chen et al., 1999)

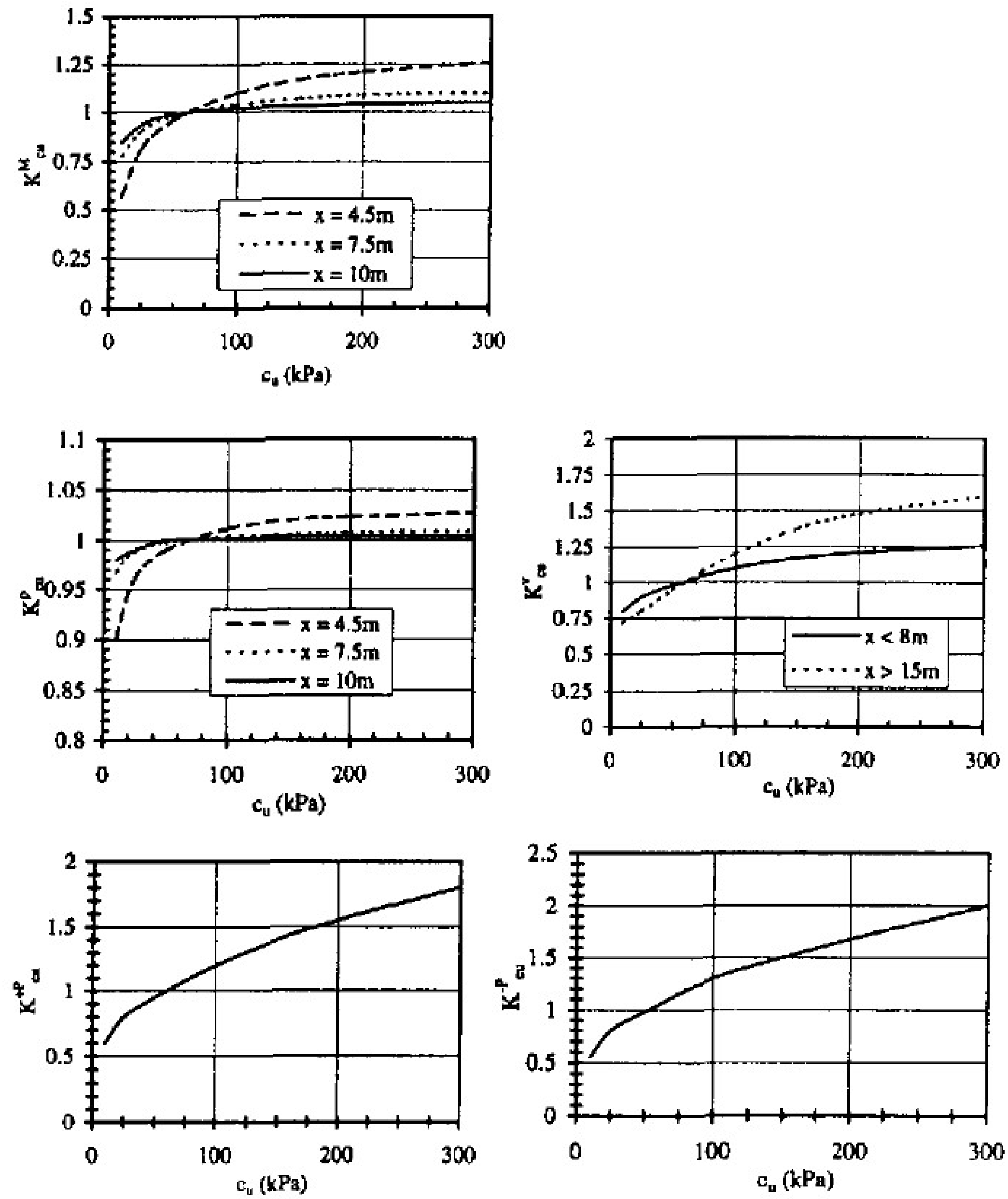


Figure 2-15: Correction factors for undrained shear strength of soil (Chen et al., 1999)

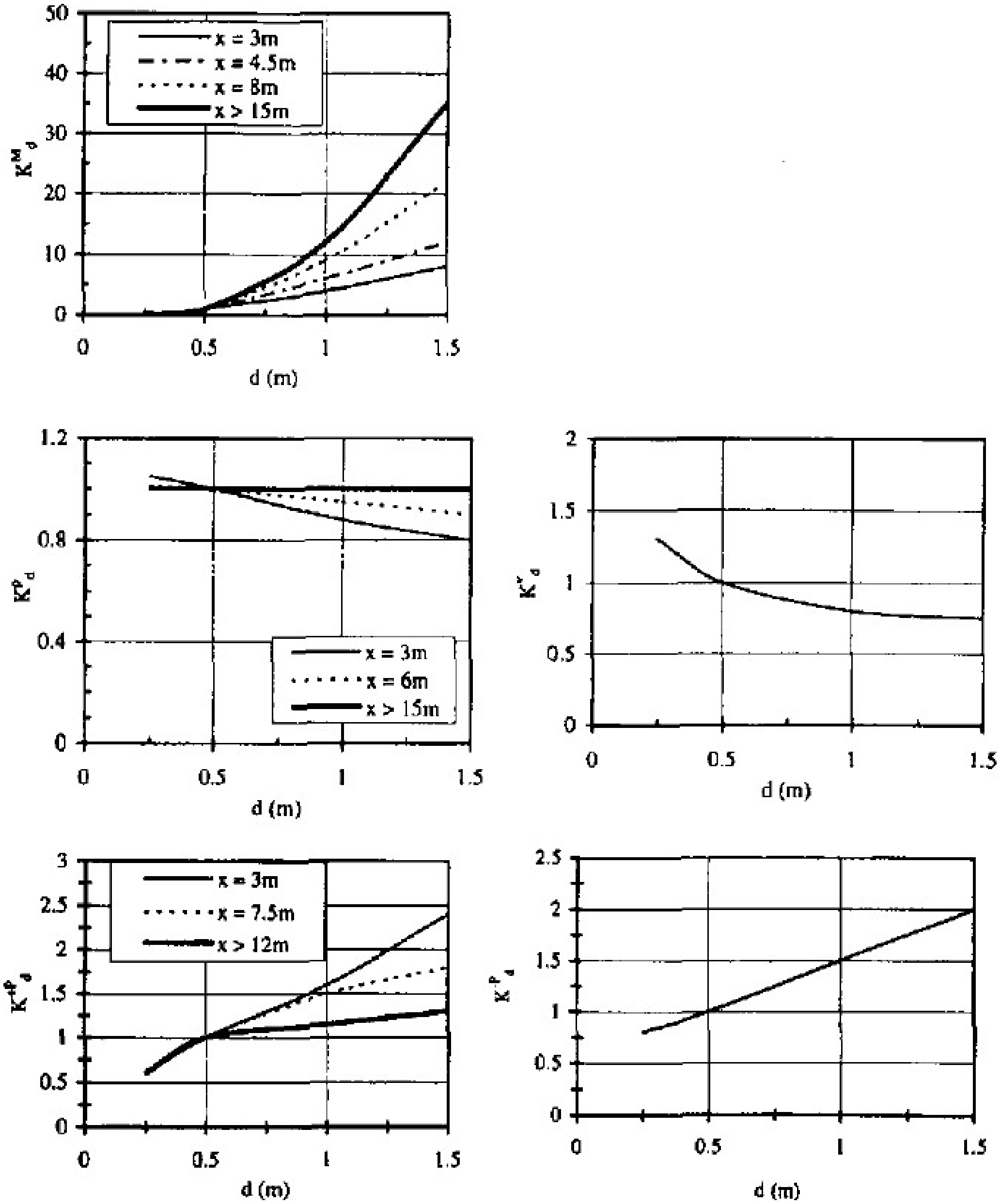


Figure 2-16: Correction factors for pile diameter (Chen et al., 1999)

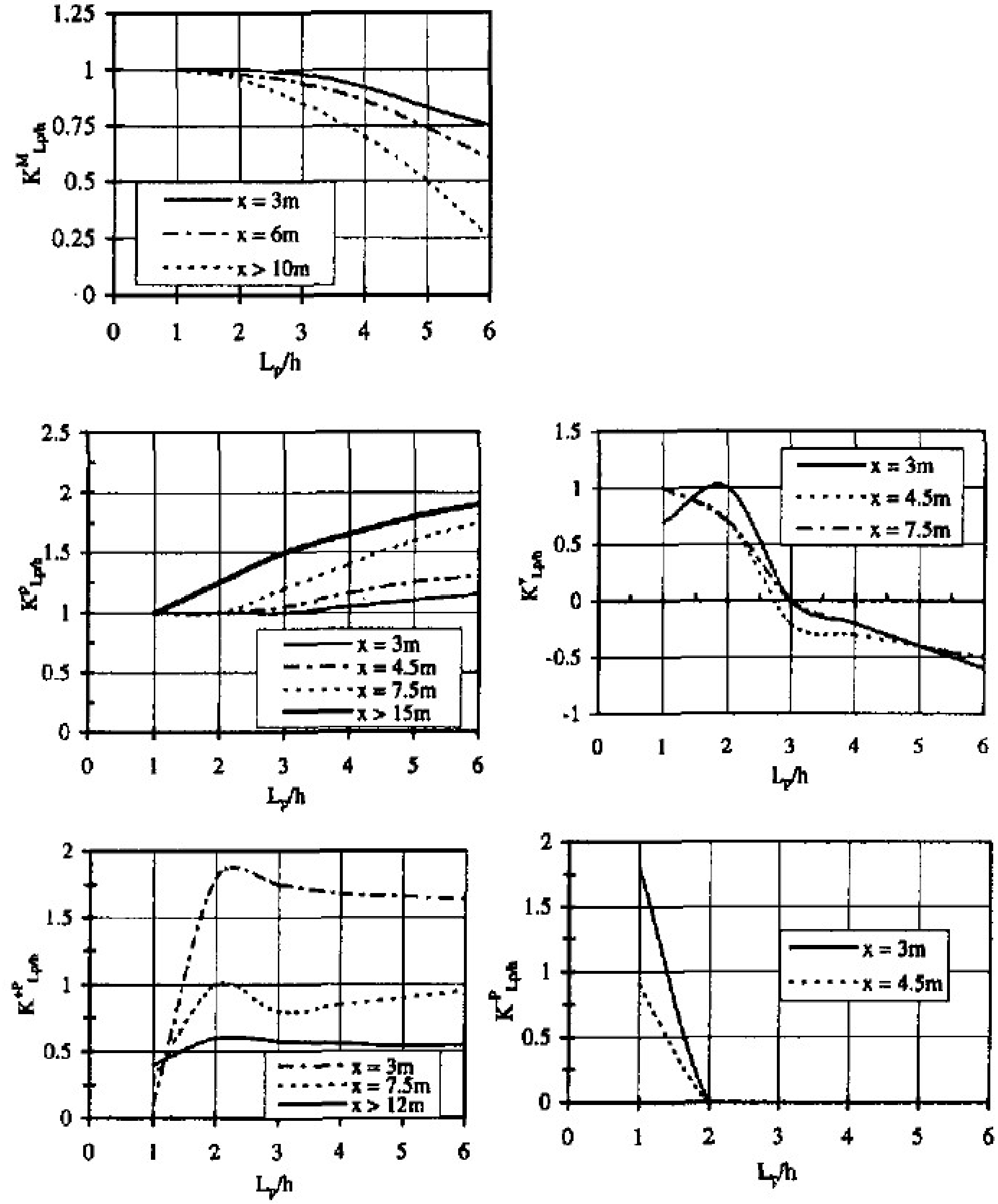


Figure 2-17: Correction factors for pile length to tunnel axis ratio (Chen et al., 1999)

Some researchers also conducted a series of 3D finite element analyses to investigate the effects of tunnels on existing pile foundations. Mroueh et al. (2002) studied elastoplastic 3D analyses using PECPLAS software. It stated that significant internal forces and deflections are induced by tunneling. The tunnel's horizontal axis with respect to the pile tip is a very important parameter according to the numerical analyses conducted within the scope of the paper.

In order to investigate tunneling-single pile interaction, Mroueh et al. (2002) created a three-dimensional finite element mesh (Figure 2-18) and conducted a series of analyses. Results are presented in Figure 2-18 through Figure 2-21.

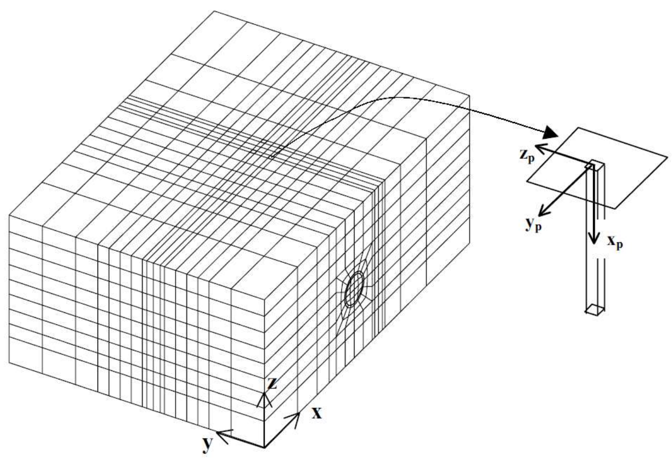


Figure 2-18: Three-dimensional finite element mesh used for the pile/tunneling interaction (Mroueh et al., 2002)

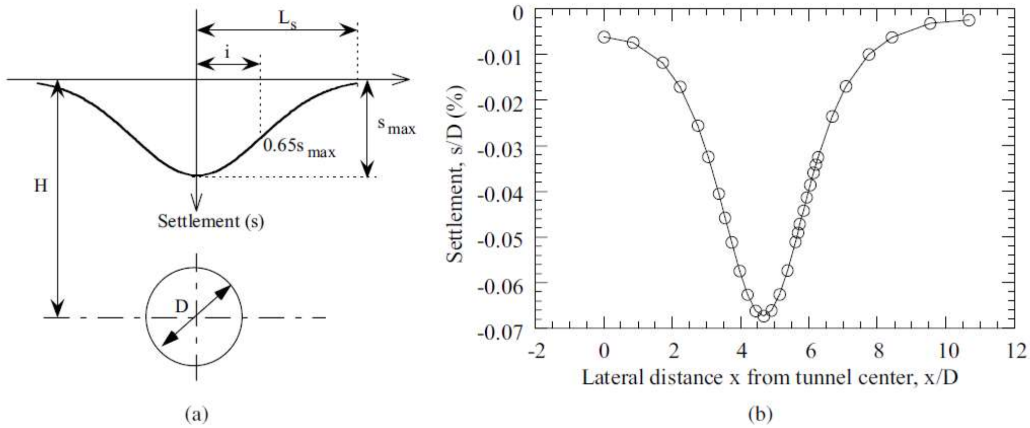


Figure 2-19: Settlement profile at the surface in a transverse section 3D behind the tunnel face (approaching plane strain condition) (Mroueh et al., 2002)

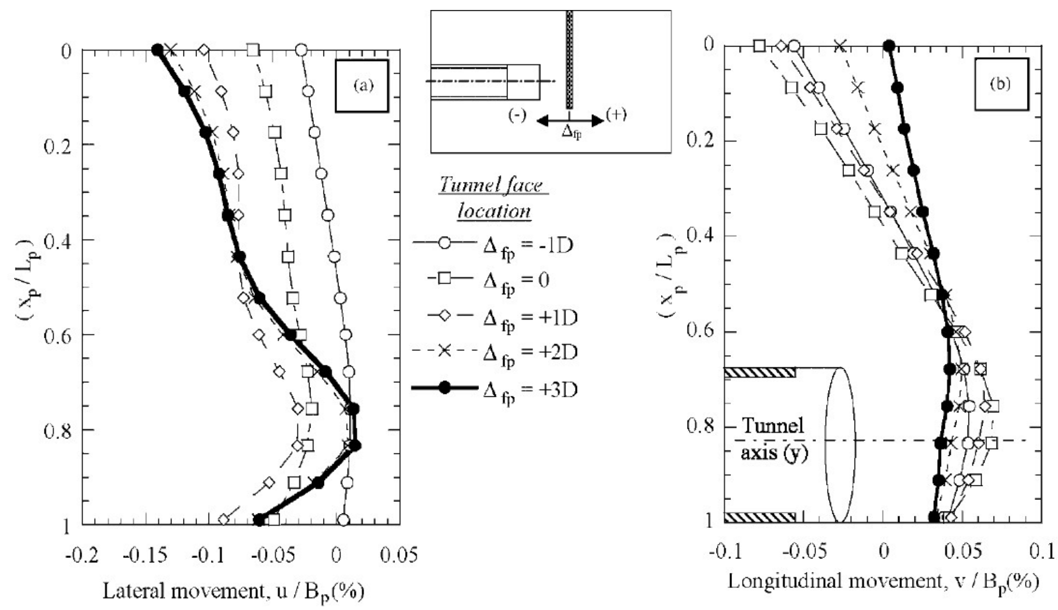


Figure 2-20: Pile deflection due to tunneling: (a) lateral section and (b) longitudinal section (Mroueh et al., 2002)

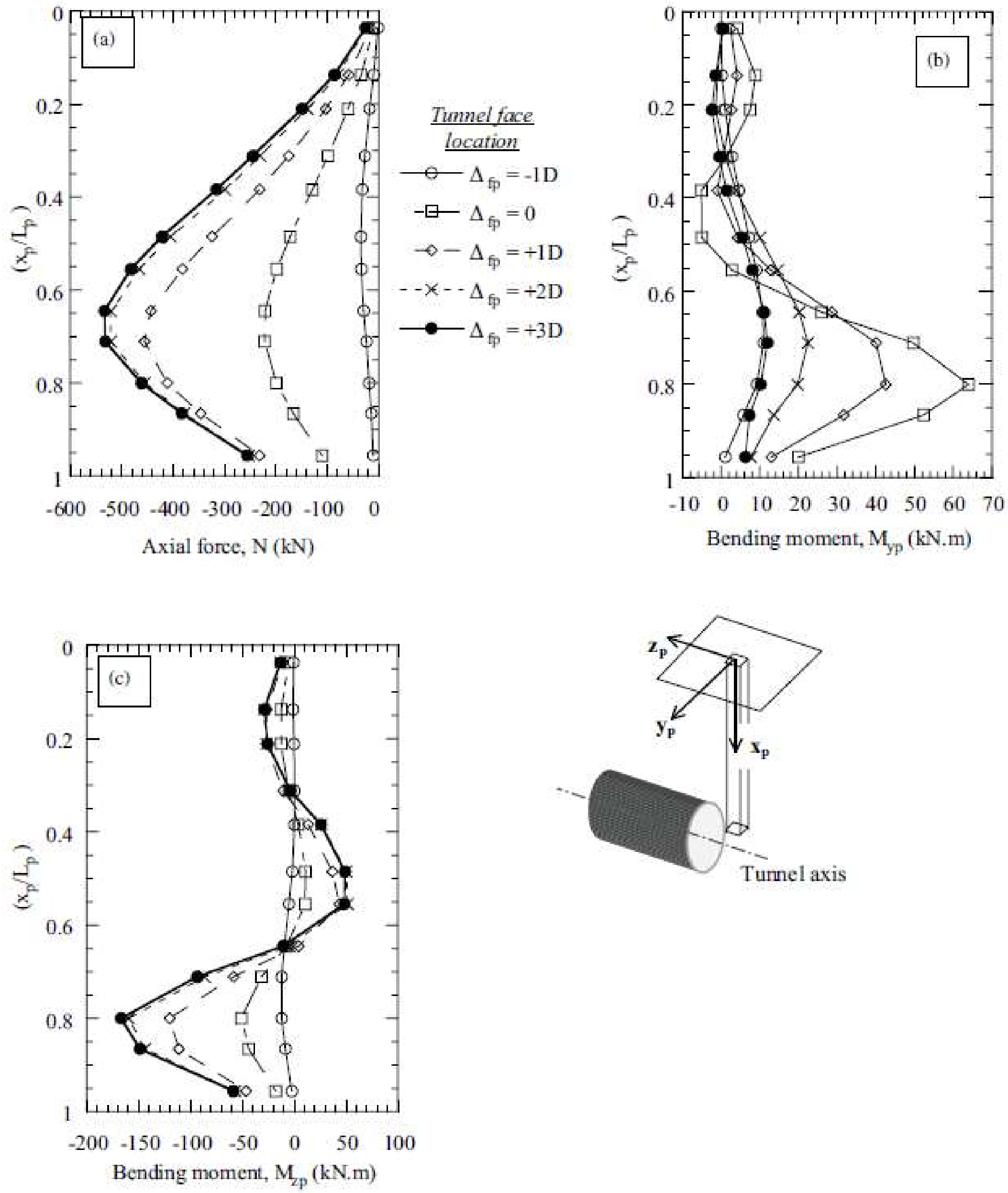


Figure 2-21: Internal forces induced by tunneling: (a) axial force (N), (b) bending moment (M_{yp}) and (c) bending moment (M_{zp}) (Mroueh et al., 2002)

Cheng et al. (2004) presented a displacement control method for finite element modelling of tunnel-soil-pile interaction and conducted a back analysis of a case study. ABAQUS (HKS, 2003) software is used for the three-dimensional total stress analyses. Typical mesh is shown in Figure 2-22 and nonlinear soil constitutive model used for parametric study is given in Figure 2-23.

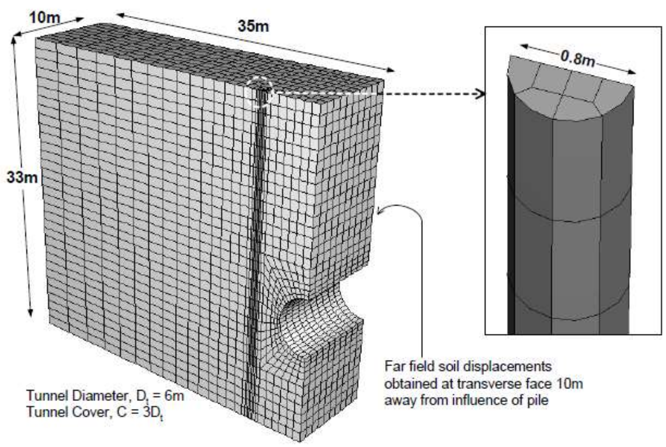


Figure 2-22: Typical mesh (Cheng et al., 2004)

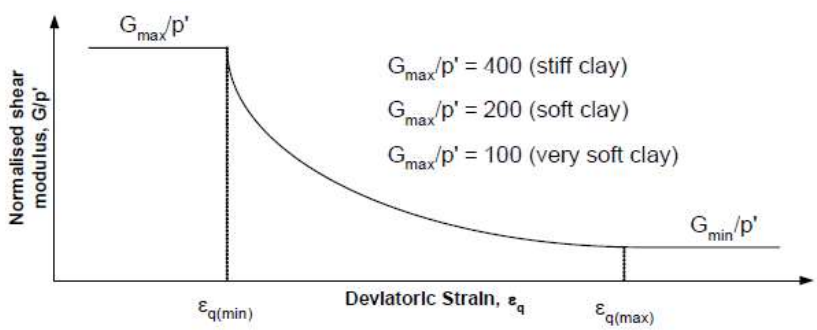


Figure 2-23: Nonlinear soil constitutive model used for parametric study (Cheng et al., 2004)

Obtained pile lateral displacement and bending moment profiles along the pile length presented in the paper (Figure 2-24). According to these figures, lateral soil displacement values match with the far field soil displacements thus showing its low bending stiffness (EI). The induced pile bending moment is maximum for the case where the pile tip is located below tunnel axis which is the case of Y_p= -1D_t . Even though the volume loss of 1% occurred, the maximum obtained moment magnitude which is M_{max}=160kN.m exceeds the pile cracking moment (M_{cr}). Besides this, M_{max} is calculated as only 25% of M_{ult}. Additional studies show that magnitudes of maximum induced bending moment remain constant for cases where $Y_p < -1D_t$.

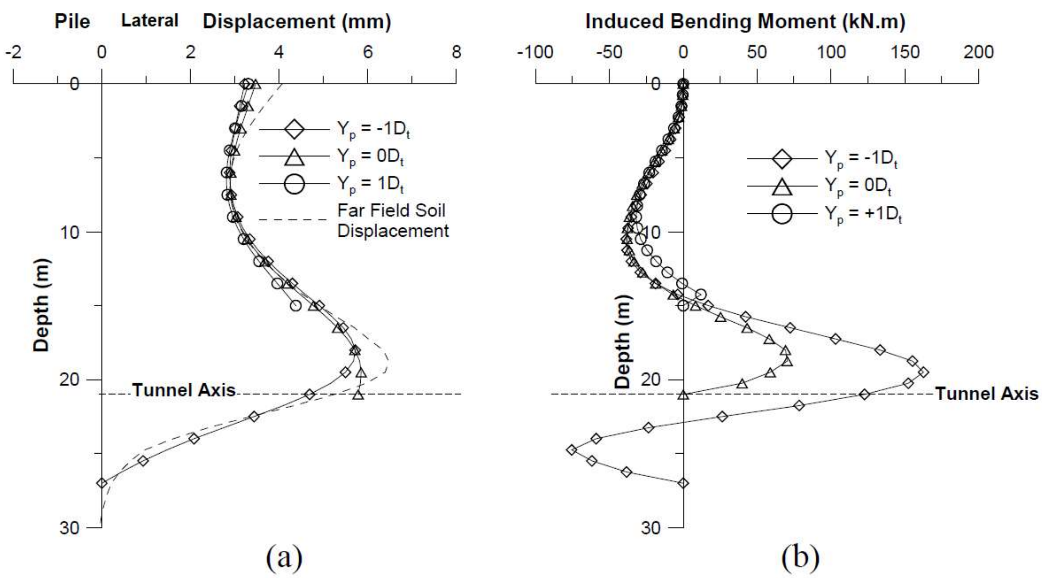


Figure 2-24: Induced (a) lateral displacement and (b) bending moment profile along pile length for G_{max}/p' = 400 (Cheng et al., 2004)

As shown in the Figure 2-25(a), settlement remains almost constant through the pile length. This is because of the high axial stiffness of a pile as expected. In the Figure 2-25(b), significant compressive force is acting on the pile at level closer to the tunnel axis. As expected, largest induced pile axial force occurs in the case where the pile tip is located below tunnel axis. In this particular research, a maximum force induced is obtained as 1790kN. This value is larger than the limiting skin friction which is 1220kN. This means limiting skin friction is achieved along significant lengths of the pile. Besides, induced concrete stress which is around 2.43MPa is smaller compared to a concrete compressive strength (30MPa).

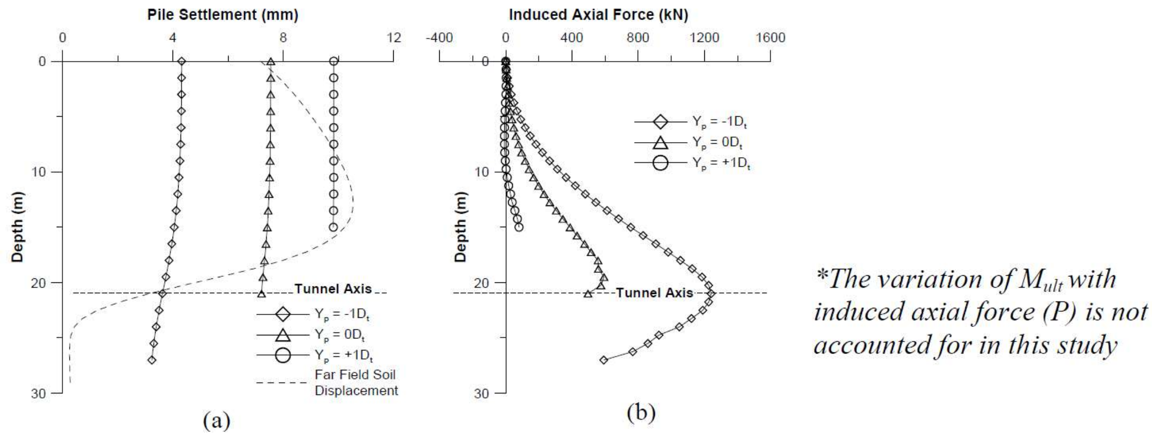


Figure 2-25: Induced (a) settlement and (b) axial force profiles along pile length for G_{max}/p' = 400 (Cheng et al., 2004)

Figure 2-26 shows that when the pile tip is located within the zone of large displacements, as bearing capacity reaction is prevented from fully develop, the pile settles more like a rigid body translation.

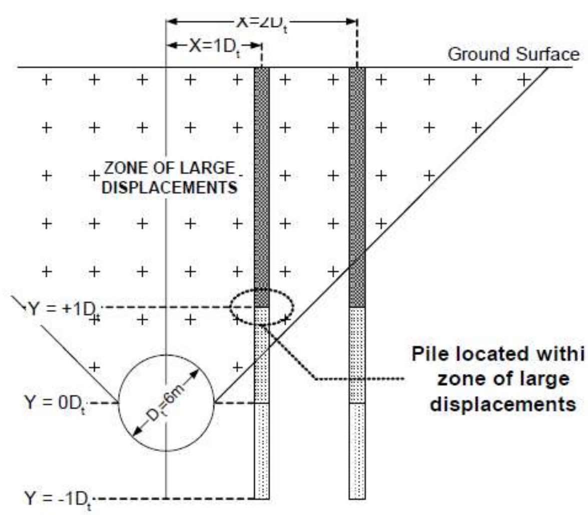


Figure 2-26: Location of pile tip relative to tunnel axis level and zone of large displacements (Cheng et al., 2004)

2.3 Parametric Studies

A series of 3D parametric numerical analyses were performed by Al-Omari et al. (2019) in order to study the response of piles to shield tunneling in the soil. The main topic for this study is to investigate longitudinal, lateral and vertical distances of the tunnel face from piles after which further tunneling process would be risky. In order to do this, several numerical analyses are conducted. Single and 3x3 group of piles with an optimum center-to-center spacing of s=3d (d: diameter of piles, D: diameter of the tunnel) were modelled. In Figure 2-27, 3-D mesh and model dimensions are given. Underground location of tunnel relative to single or group of piles and piles cap dimensions and locations of piles in groups are given in Figure 2-28.

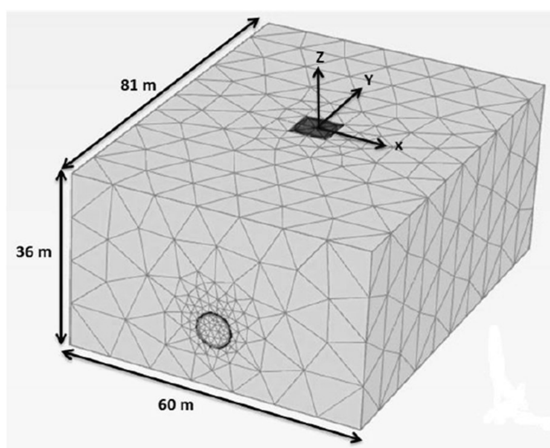


Figure 2-27: Three dimensional finite element mesh and problem dimensions (Al-Omari et al., 2019)

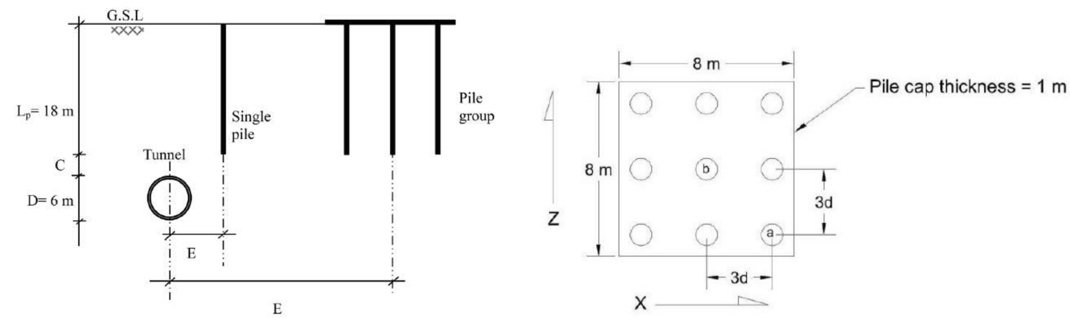


Figure 2-28: Underground location of tunnel relative to single or group of piles and piles cap dimensions and locations of piles in groups (Al-Omari et al., 2019)

Following results are obtained from the study:

i. Pile head settlement increase might occur by tunneling compared to the pile head settlement under service loads.
ii. About 89% of total settlement of single pile and 94% of total settlement for the center pile in group occur within the range of ±2D from the pile center.
iii. With the progress tunnel face towards the pile and passes the pile, in a significant range of 5D, pile head settlement increases.
iv. The maximum pile head settlement due to tunneling is only 1.27 times larger for a single pile and 1.42 times larger for a center pile of groups than that obtained from the Greenfield condition (tunneling analysis without pile presence condition).

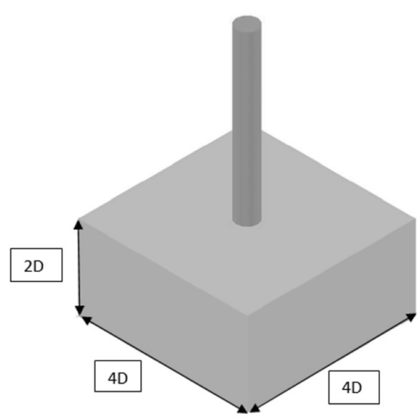


Figure 2-29: The zone of significant influence during tunnelling (Al-Omari et al., 2019)

CHAPTER 3

PARAMETRIC STUDY AND NUMERICAL ANALYSES

3.1 Software Program Selection

A software program working with finite element method is required in order to get the results of the problem defined. The most convenient and available option was *PLAXIS 2D (V20)*. Plaxis 2D is a software that performs two-dimensional analysis of deformation and stability in geotechnical engineering. Engineers in field and in researches rely on the results that Plaxis 2D provides. Plaxis 2D has a capability to model and solve different projects such as excavations, foundations, tunneling, reservoir etc.

It allows the user to model the structural elements, soil layers and loadings. After modelling the geometry and structural elements, mesh should be created to solve the problem with finite element method. Plaxis 2D allows users to get an automatically created finite element mesh almost immediately.

Construction steps can be accurately modelled by activating and deactivating the soil clusters, structural elements in each calculation steps with staged construction. A great range of geotechnical problems can be analyzed like consolidation and safety.

With these mentioned features, Plaxis 2D (V20) is chosen to be used in finite element analyses.

3.2 Parameter Selection

In the light of former researches, parameters which are used in this research are decided. A model which takes most of the important findings of the previous studies into account is tried to be constructed and the geometrical properties of the model selected is presented in Figure 3-1 below.

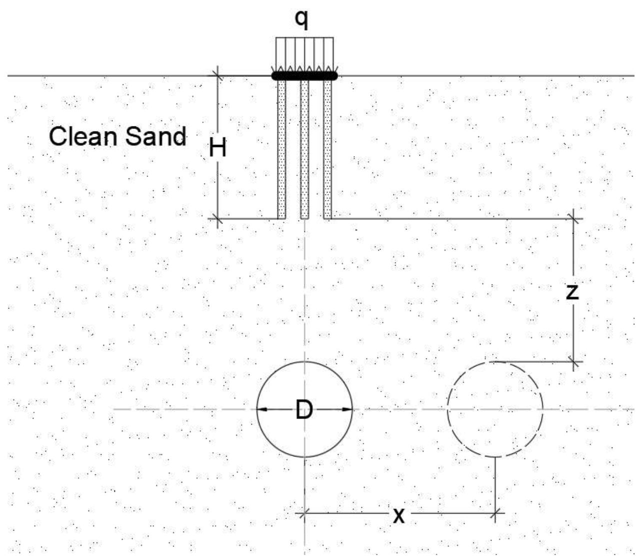


Figure 3-1: Schematic Model for Numerical Analyses

where,

H: Pile length

D: Tunnel diameter

z: Clear distance between pile toe and tunnel cover

x: Distance between tunnel center axis and the pile group center axis.

For this research, soil type is selected as clean sand in order to evaluate the immediate settlements due to the tunneling. Since the foundation is piled, it is considered that consistency of the sand layer would be loose or medium to be more realistic as pile foundations are generally not preferred in denser soils. According to Kulhawy and Mayne (1990), typical normalized elastic modulus values are suggested as presented in Table 3-1:

Table 3-1: Typical Normalized Elastic Modulus Values for Sand with Different Consistencies (Kulhawy&Mayne, 1990)

Consistency	Normalized Elastic Modulus, E_d/P_a
Loose	100 – 200
Medium	200 – 500
Dense	500 – 1000

In the light of suggested values, elastic moduli are decided as 15000 kPa, 25000 kPa and 35000 kPa for Sand I, Sand II and Sand III, respectively.

Pile length and tunnel diameter decided to be kept constant and they are selected as **H=15.0m** and **D=10.0m** respectively. Actually, the dimensionless parameter, ratio of the depth and location of the tunnel to the pile diameter (z/D and x/D) will be used as a comparative parameter while evaluating the results instead of using that parameters alone.

A constant structural load of **q=150kPa** is used in the analyses which is assumed to correspond for a typical 10-storey building.

According to the Figure 3-1, Table 3-1 and engineering judgement, parameters are decided as;

i. *Deformation Modulus, E,*
- Sand I: E=15000 kPa
- Sand II: E=25000 kPa
- Sand III: E=35000 kPa

ii. *Clear distance between pile toe and tunnel cover to tunnel diameter ratio, z/D*

- z/D= 0.25
- z/D=0.50
- z/D=1.00
- z/D= 1.50
- z/D=2.00

iii. *Distance between tunnel center axis and the pile group center axis to tunnel diameter ratio, x/D*

- x/D=0.00
- x/D=0.50
- x/D=1.00
- x/D=1.50
- x/D=2.00

Analysis numbers and parameters in the corresponding analysis are tabulated in the below table (Table 3-2).

Table 3-2: Analysis no and parameters in the corresponding analysis

Analysis Number	z/D	x/D	E (kPa)
No.1	0.25	0.00	15000
No.2	0.25	0.00	25000
No.3	0.25	0.00	35000
No.4	0.25	0.50	15000
No.5	0.25	0.50	25000
No.6	0.25	0.50	35000
No.7	0.25	1.00	15000
No.8	0.25	1.00	25000
No.9	0.25	1.00	35000
No.10	0.25	1.50	15000
No.11	0.25	1.50	25000
No.12	0.25	1.50	35000
No.13	0.25	2.00	15000
No.14	0.25	2.00	25000
No.15	0.25	2.00	35000

Analysis Number	z/D	x/D	E (kPa)
No.16	0.50	0.00	15000
No.17	0.50	0.00	25000
No.18	0.50	0.00	35000
No.19	0.50	0.50	15000
No.20	0.50	0.50	25000
No.21	0.50	0.50	35000
No.22	0.50	1.00	15000
No.23	0.50	1.00	25000
No.24	0.50	1.00	35000
No.25	0.50	1.50	15000
No.26	0.50	1.50	25000
No.27	0.50	1.50	35000
No.28	0.50	2.00	15000
No.29	0.50	2.00	25000
No.30	0.50	2.00	35000
No.31	1.00	0.00	15000
No.32	1.00	0.00	25000
No.33	1.00	0.00	35000
No.34	1.00	0.50	15000
No.35	1.00	0.50	25000
No.36	1.00	0.50	35000
No.37	1.00	1.00	15000
No.38	1.00	1.00	25000
No.39	1.00	1.00	35000
No.40	1.00	1.50	15000
No.41	1.00	1.50	25000
No.42	1.00	1.50	35000
No.43	1.00	2.00	15000
No.44	1.00	2.00	25000
No.45	1.00	2.00	35000
No.46	1.50	0.00	15000
No.47	1.50	0.00	25000
No.48	1.50	0.00	35000
No.49	1.50	0.50	15000
No.50	1.50	0.50	25000
No.51	1.50	0.50	35000
No.52	1.50	1.00	15000
No.53	1.50	1.00	25000
No.54	1.50	1.00	35000
No.55	1.50	1.50	15000
No.56	1.50	1.50	25000
No.57	1.50	1.50	35000
No.58	1.50	2.00	15000
No.59	1.50	2.00	25000
No.60	1.50	2.00	35000
No.61	2.00	0.00	15000

Analysis Number	z/D	x/D	E (kPa)
No.62	2.00	0.00	25000
No.63	2.00	0.00	35000
No.64	2.00	0.50	15000
No.65	2.00	0.50	25000
No.66	2.00	0.50	35000
No.67	2.00	1.00	15000
No.68	2.00	1.00	25000
No.69	2.00	1.00	35000
No.70	2.00	1.50	15000
No.71	2.00	1.50	25000
No.72	2.00	1.50	35000
No.73	2.00	2.00	15000
No.74	2.00	2.00	25000
No.75	2.00	2.00	35000

3.3 Soil and Structural Element Modelling in Plaxis 2D (V.20)

3.3.1 Soil Modelling in Plaxis 2D (V.20)

As mentioned before, the analysis will be performed in three different cohesionless clean sands with different stiffness values. To include the effect of stiffness of the soil when unloading-reloading and to obtain more realistic settlement values, Hardening Soil Model is chosen as for Sand I, II and III. The basic feature of the Hardening Soil model is the stress dependent soil stiffness. The model implies the relationship of below equation (Eqn. 3-1).

$$E_{50} = E_{50}^{ref}\left(\frac{c\cos(\phi)-\sigma_3'\sin(\phi)}{c\cos(\phi)-p^{ref}\sin(\phi)}\right) \qquad \text{Equation 3-1}$$

$$E_{oed} = E_{oed}^{ref}\left(\frac{c\,\mathrm{co}\ \ (\phi)-\sigma_1'\sin(\phi)}{c\cos(\phi)-p^{ref}\sin(\phi)}\right) \qquad \text{Equation 3-2}$$

where;

c: effective cohesion value of the soil [kN/m^2]

ϕ: effective internal friction angle of the soil [°]

E_{50}: confining stress dependent stiffness modulus for primary loading [kN/m^2]

E_{oed}: tangent stiffness modulus obtained from an oedometer test [kN/m^2]

m: Power for stress-level dependency of stiffness (=0.5 for sandy soils according to the Plaxis 2D (V.20) Material Models Manual)

E_{50}^{ref}: reference stiffness modulus corresponding to the reference confining pressure, p^{ref} (i.e. Secant stiffness in standard drained triaxial test) [kN/m^2]

E_{oed}^{ref}: tangent stiffness at a vertical stress (i.e. Tangent stiffness for primary oedometer loading) [kN/m^2]

p^{ref}: reference pressure (a default setting p^{ref}= 100 kN/m^2 is used.) [kN/m^2]

σ_1': effective vertical stress in the middle of the soil layer [kN/m^2]

σ_3': effective horizontal stress in the middle of the soil layer ($\sigma_1'(1 - sin\phi)$) [kN/m^2]

Unsaturated and saturated unit weights of Sand I, II and III are chosen by using suggestions of Clayton (1995). Clayton (1995) suggested that for well graded sand, bulk unit weight and saturated bulk unit weight could be taken as γ_{unsat}=18.0-21.0 kN/m^3 and γ_{sat}=20.5.0-22.5 kN/m^3, respectively.

Drainage type is chosen as “Drained” since the soil is cohesionless. All the parameters used in the analyses are presented in Table 3-3.

Table 3-3: Soil Parameters

Soil Type/ Parameters	Sand I	Sand II	Sand III	Units
Dry Unit Weight, γ_{unsat}	18.50	18.50	18.50	kN/m^3
Saturated Unit Weight, γ_{sat}	19.00	19.00	19.00	kN/m^3
E_{50}^{ref}	9650	16000	22500	kN/m^2
E_{oed}^{ref}	9650	16000	22500	kN/m^2
E_{ur}^{ref} (1)	28950	48000	67500	kN/m^2
c (2)	5	5	5	kN/m^2
ϕ (3)	25	25	25	°

Note (1): In many practical cases, it is appropriate to set $E_{ur}^{ref} = 3 * E_{50}^{ref}$, suggested by Pramthawee et al (2011). (E_{ur}^{ref}: Reference Young's modulus and reloading, corresponding to the reference pressure p^{ref}.)

Note(2): Cohesion values are assumed as 5kPa for simplicity, Elasticity modulus, E, will have more effect on deformations as compared to c, since the focus is deformations in this thesis.

Note(3): Internal friction angle values are assumed as 25° for simplicity, Elasticity modulus, E, will have more effect on deformations as compared to ϕ, since the focus is deformations in this thesis.

3.3.2 Structural Element Modelling in Plaxis 2D (V.20)

Structural elements used in numerical analyses are; foundation (pile cap), piles and tunnel. In this section, modelling of these elements is discussed. All structural elements are assumed to be concrete. Concrete type is decided as C30 with 28th day modulus of elasticity as E=3.025x10^7 kPa and unit weight γ=24.0 kN/m^3.

A square foundation with dimensions of BxL=6.0x6.0 m^2 and thickness of h=1.0m is modelled. Its moment of inertia is calculated by using Eqn. 3-3:

$$I_{foundation} = \frac{b*h^3}{12} \qquad \text{Equation 3-3}$$

3x3 pile group with diameter of Ø80cm and length of L=15.0m is modelled. All piles are modelled as individual piles in Plaxis. (See Figure 3-1). Spacing of s=2.40m is chosen to eliminate the group effect (s≥3D, D: pile diameter). Their moment of inertia is calculated by using Eqn. 3-4:

$$I_{piles} = \frac{\pi * r^4}{4} \qquad \text{Equation 3-4}$$

Note: Since Plaxis 2D models everything drawn in x-y direction as they continue through in z-direction, inputs for piles (EI, and EA) are divided into the spacing which is s=2.40m.

Tunnel diameter is chosen as D=10.0m with a tunnel thickness (tunnel lining) of t=50cm. Tunnel is modelled as circular with volume loss of 1.0%. Its moment of inertia is calculated by the formula of moment of inertia of a pipe (Eqn. 3-5).

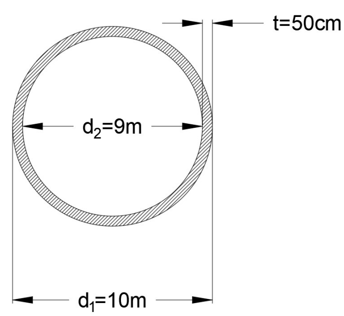


Figure 3-2: Tunnel geometry in details

$$I_{tunnel} = \frac{\pi * (d_1^4 - (d_1 - 2t)^4)}{64} \qquad \text{Equation 3-5}$$

Pile cap, pile and tunnel lining properties are tabulated below (Table 3-4).

Table 3-4: Pile cap, pile and tunnel lining properties

Material set				
Identification number		1	2	3
Identification		Pile Cap	d=80cm s=2.4m Pile	Tunnel Lining
Comments				
Colour		RGB 28, 28, 130	RGB 0, 0, 255	RGB 23, 5, 8
Material type		Elastic	Elastic	Elastic
Properties				
Isotropic		Yes	Yes	Yes
EA_1	kN/m	30.25E6	6.336E6	5.107E9
EA_2	kN/m	30.25E6	6.336E6	5.107E9
EI	kN m²/m	2.521E6	253.4E3	451.4E6
d	m	1.000	0.6928	1.030
w	kN/m/m	0.000	1.050	74.61
ν (nu)		0.1500	0.1500	0.1500
Rayleigh α		0.000	0.000	0.000
Rayleigh β		0.000	0.000	0.000
Prevent punching		No	No	No
Parameters				
Identification number		1	2	3
c	kJ/t/K	0.000	0.000	0.000
λ	kW/m/K	0.000	0.000	0.000
ρ	t/m³	0.000	0.000	0.000
α	1/K	0.000	0.000	0.000

Tunnel is modelled by using tunnel designer feature of Plaxis 2D. Settings are given in Figure 3-3.

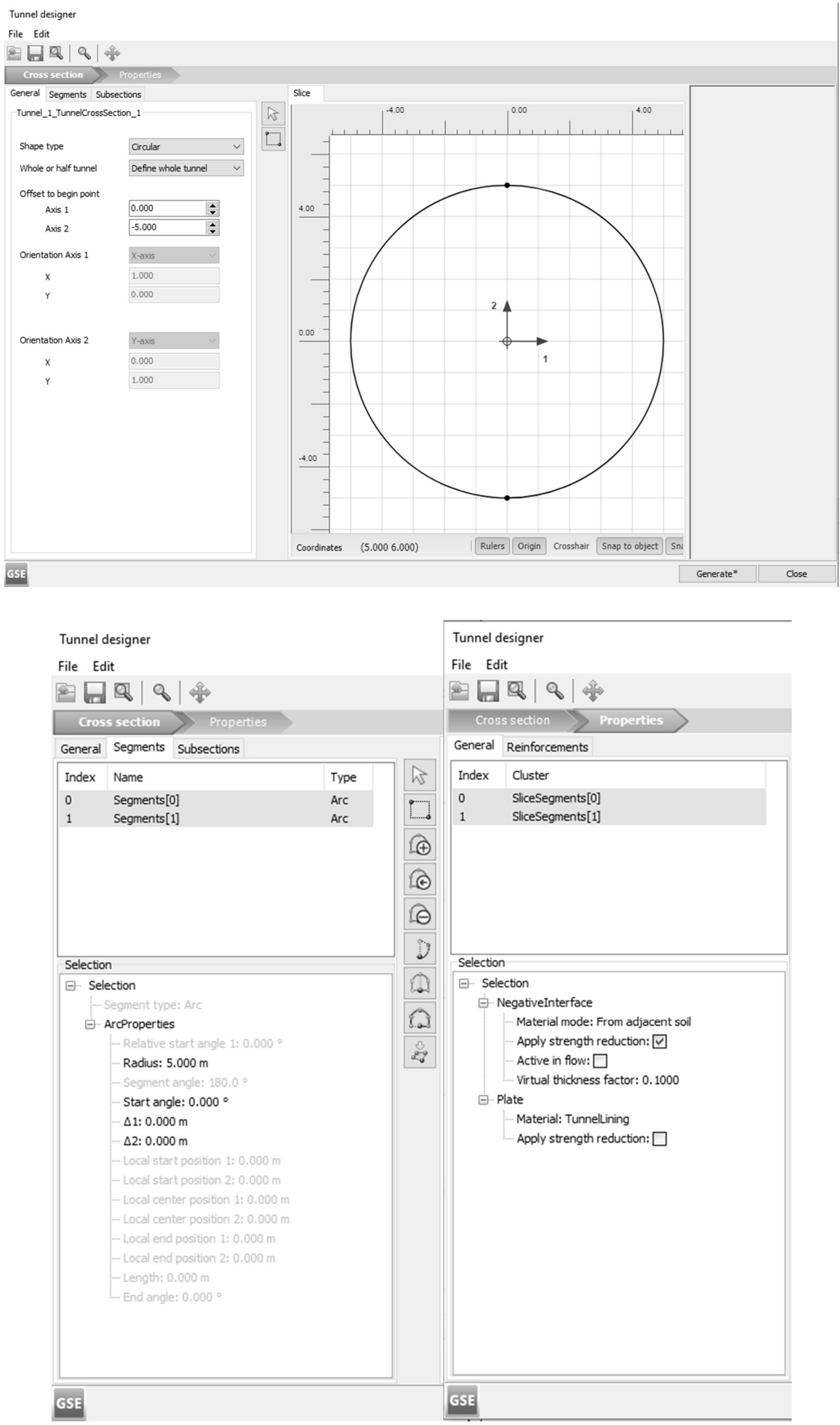


Figure 3-3: Tunnel modelling in details

Note: In this study, the tunnel thickness is selected to be 50 cm’s a very rigid tunnel. It is created so that any tunnel failure is eliminated.

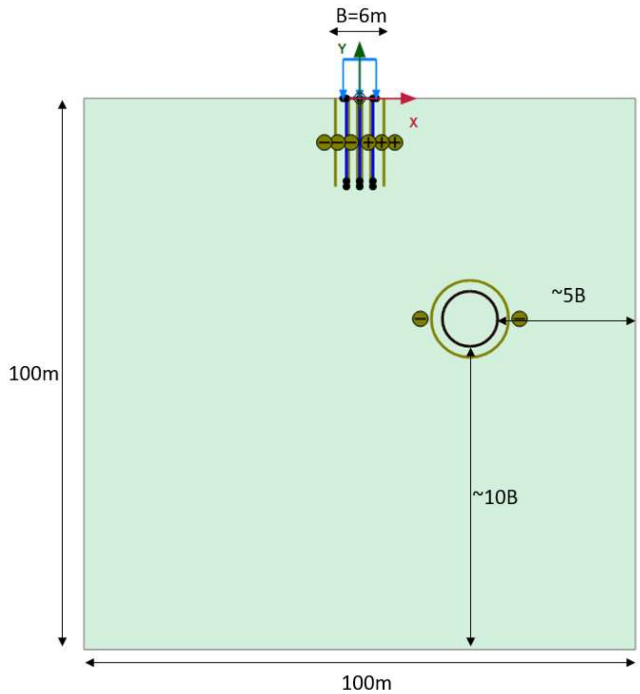


Figure 3-4: Model Geometry Dimensions

The dimensions presented in the figure above has been used in the analyses. The boundary conditions are selected to be the standard conditions suggested in PLAXIS. The mesh size has been chosen as a fine mesh to assess the results more accurately as the run time of the model does not change much as analyses are static.

3.4 Analysis Steps in Plaxis 2D (V.20) (Staged Construction)

Analysis are conducted by using loading type of Staged Construction. Groundwater table is assumed to be at the ground surface. Analysis steps are given below. As an example of the analysis performed, figures from Analysis No.1 is used.

- Initial phase: This phase is automatically added by Plaxis. It calculates initial stress conditions. According to the Plaxis 2D (V.20) Reference Manual, K0 procedure is suitable in cases with a horizontal surface and with all soil layers and phreatic levels parallel to the surface same as the model created in this study.

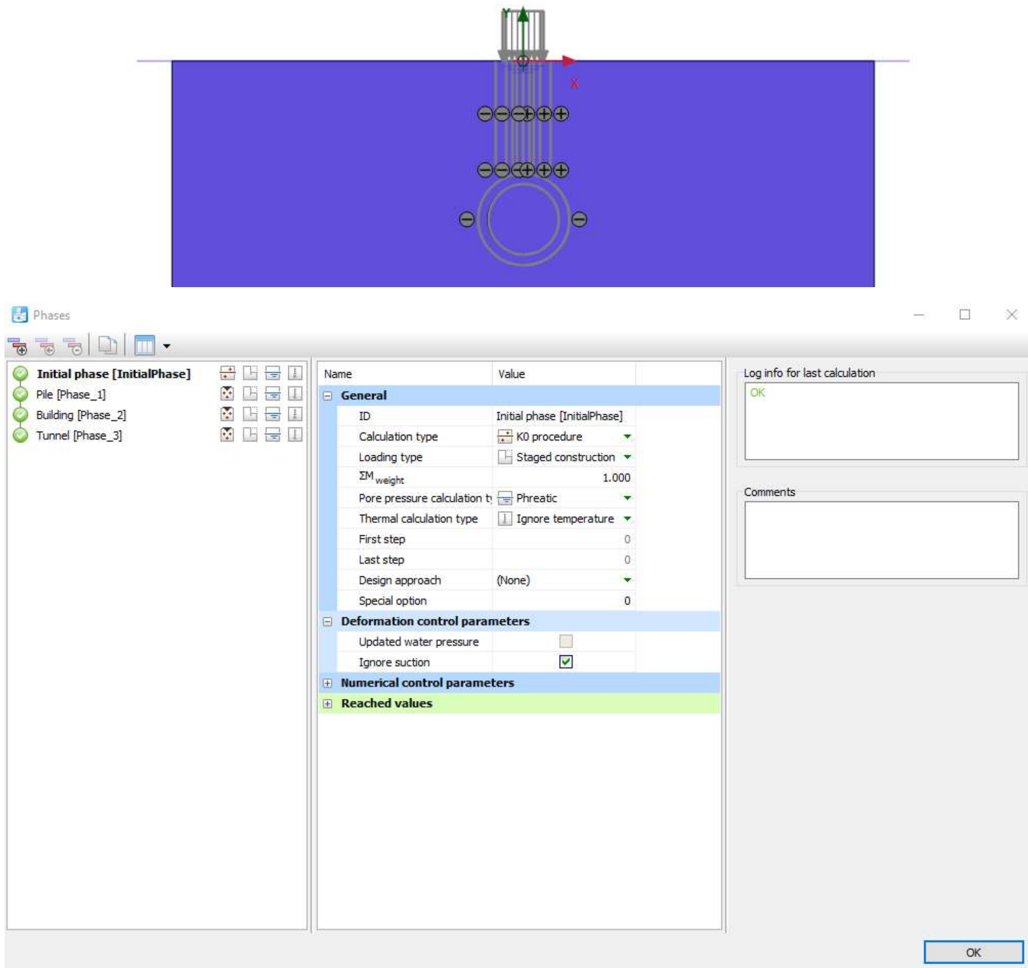


Figure 3-5: Initial Phase for Analysis No.1

- Phase_1 Pile construction: After the initial phase piles are installed. In order to achieve this, piles and their interfaces are activated in this step.

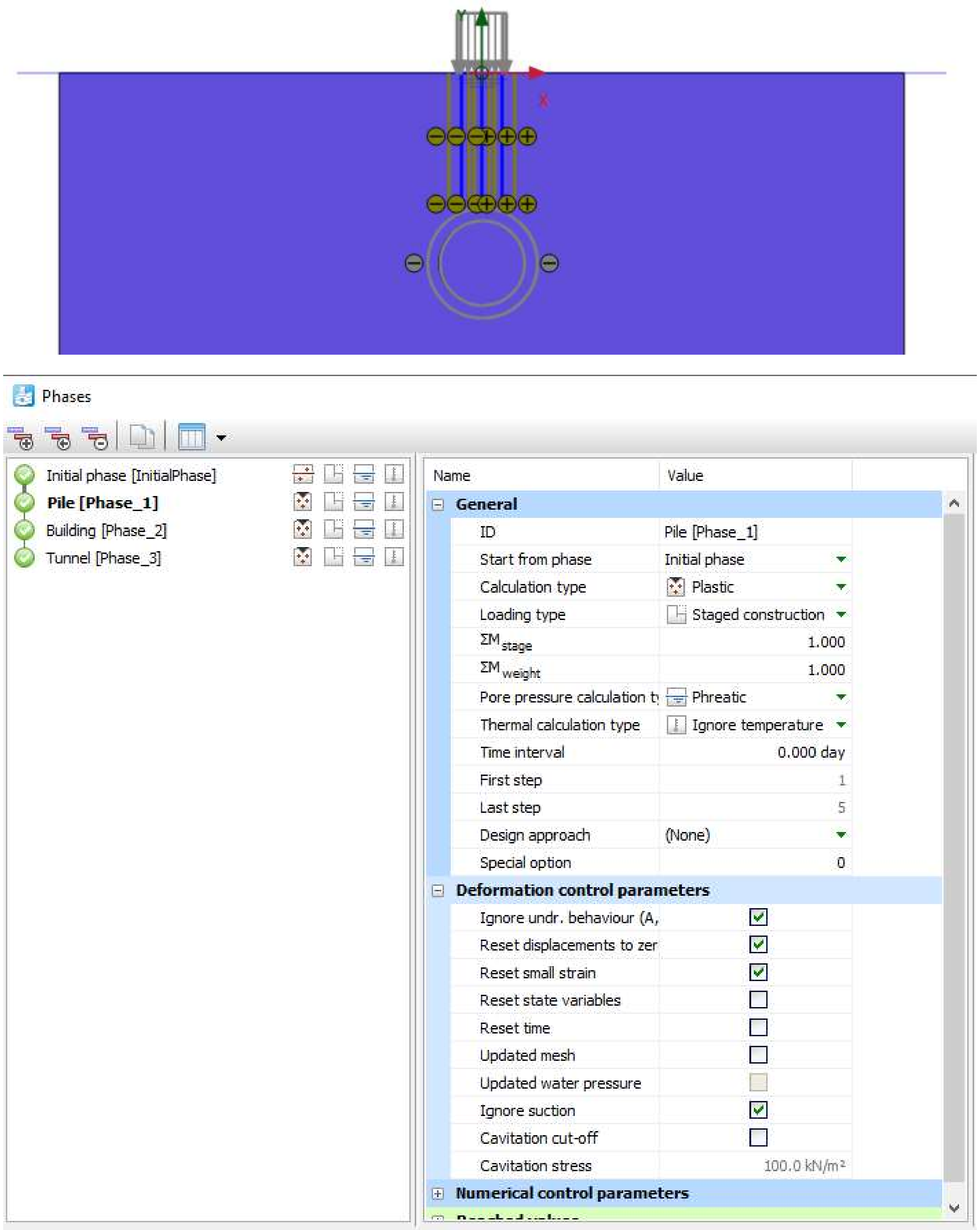


Figure 3-6: Phase_1 for Analysis No.1

- Phase_2 Pile Cap & Structural Loading: As the second construction step, pile cap is constructed and structural load is applied. To do that, structural load and pile cap is activated in this step. Note that structural load is taken as 150kPa.

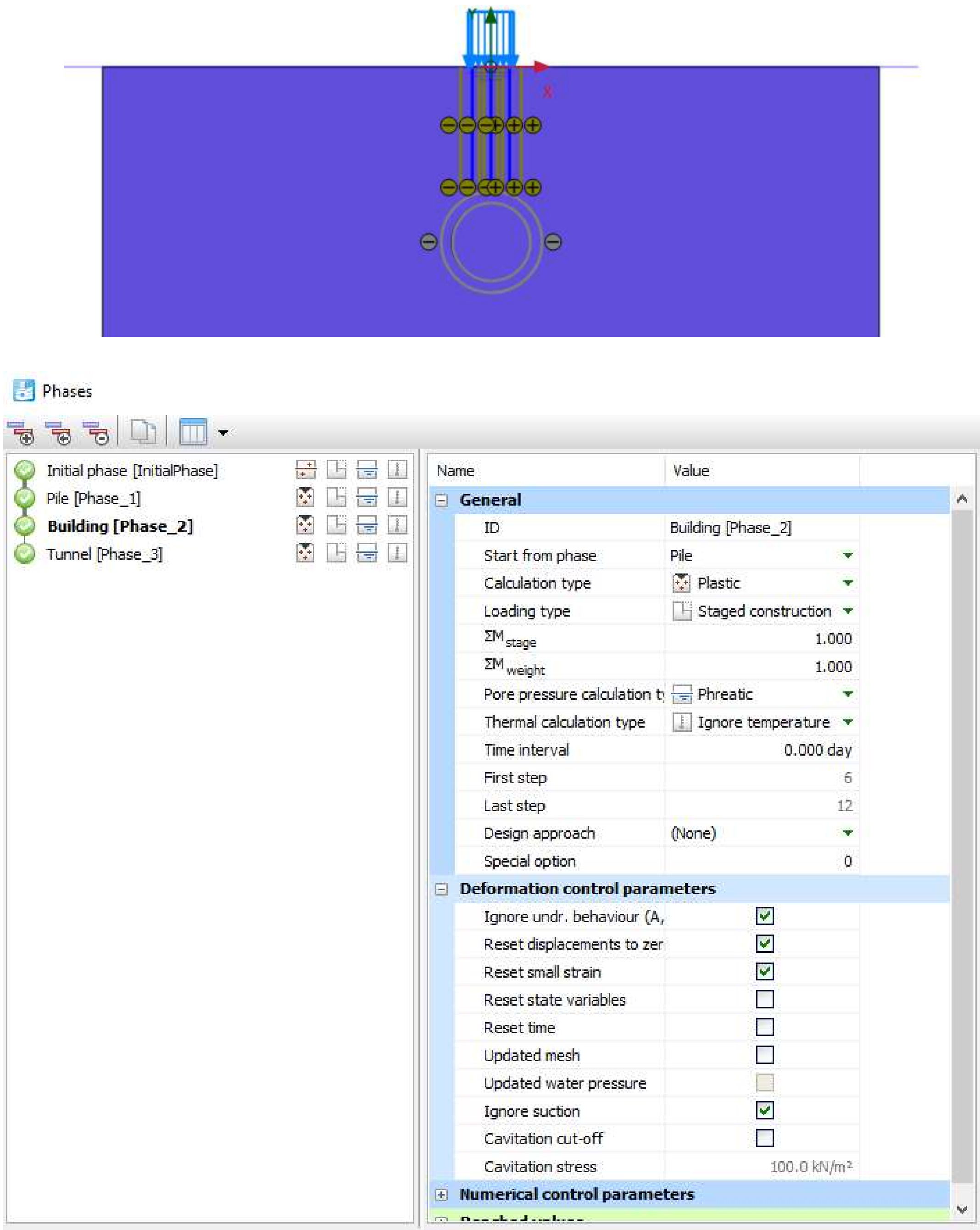


Figure 3-7: Phase_2 for Analysis No.1

- <u>Phase 3 Tunnel construction:</u> As the last but not the least construction step, tunnel is drilled. In order to do that, tunnel lining and its interface is activated, soil inside the tunnel is deactivated and for this inside area, cluster sets to dry in flow conditions tab.

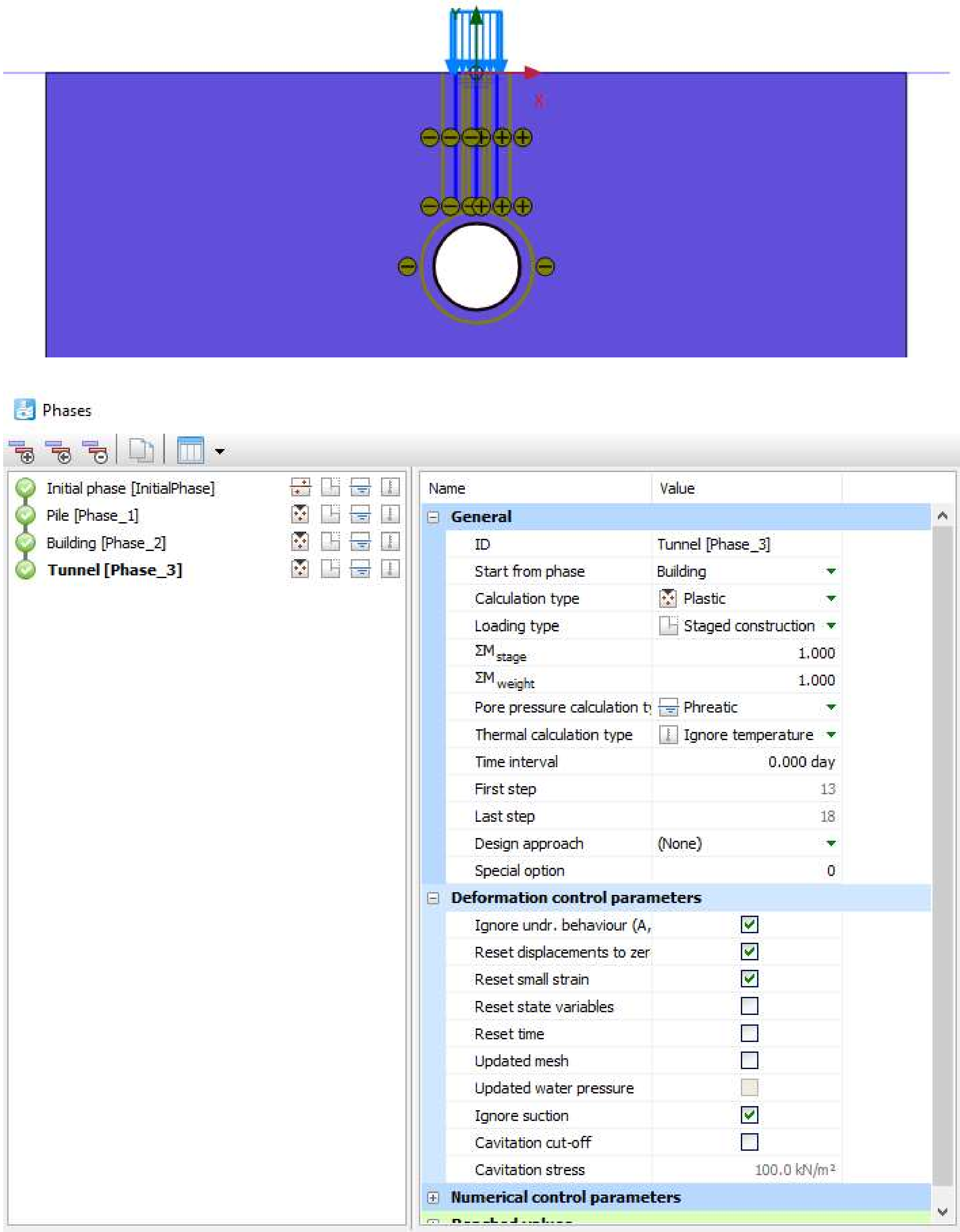


Figure 3-8: Phase_3 for Analysis No.1

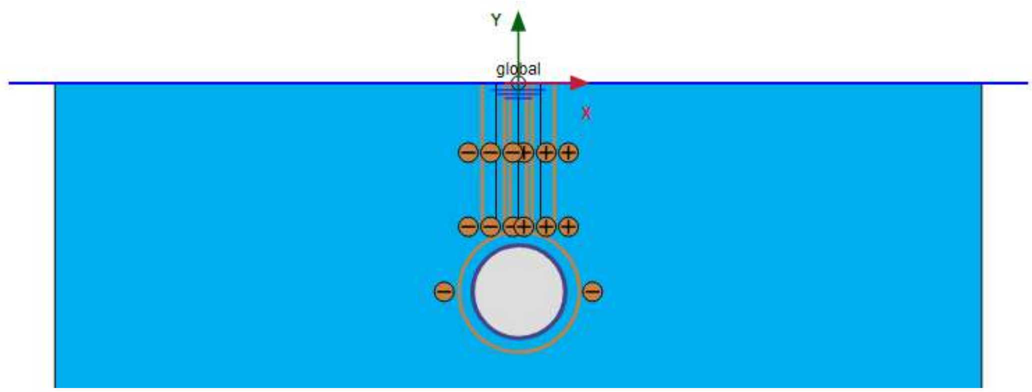


Figure 3-9: Phase_3 groundwater conditions for Analysis No.1 (Flow Conditions tab)

Typical design mesh of Analysis No.1 is given in Figure 3-10 as an example for all analyses. Five different displacement diagrams in which the location of the piles, thus the displacement pattern is different are presented in Figure 3-11, 3-12, 3-13, 3-14 and 3-15.

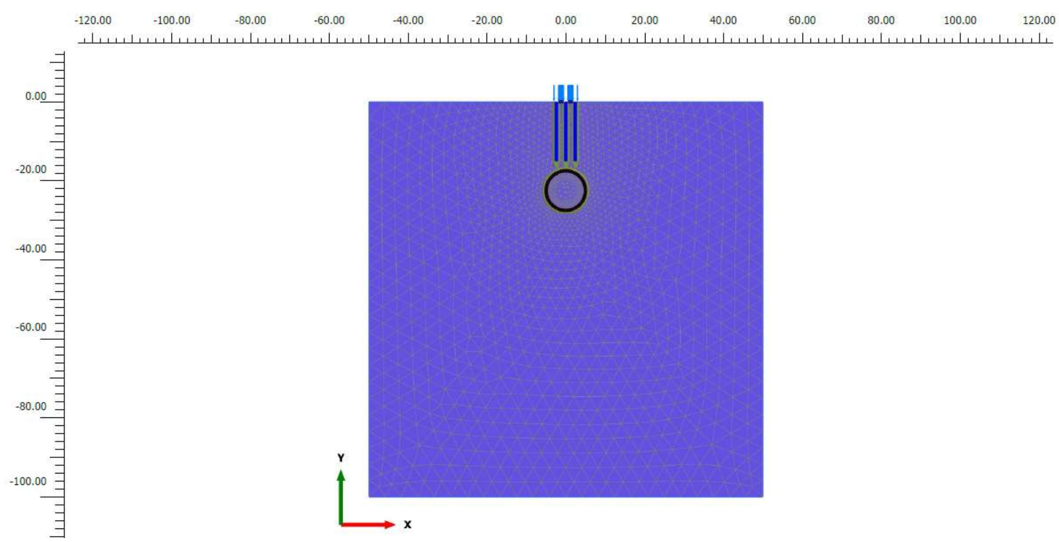


Figure 3-10: Design Mesh for Analysis No.1

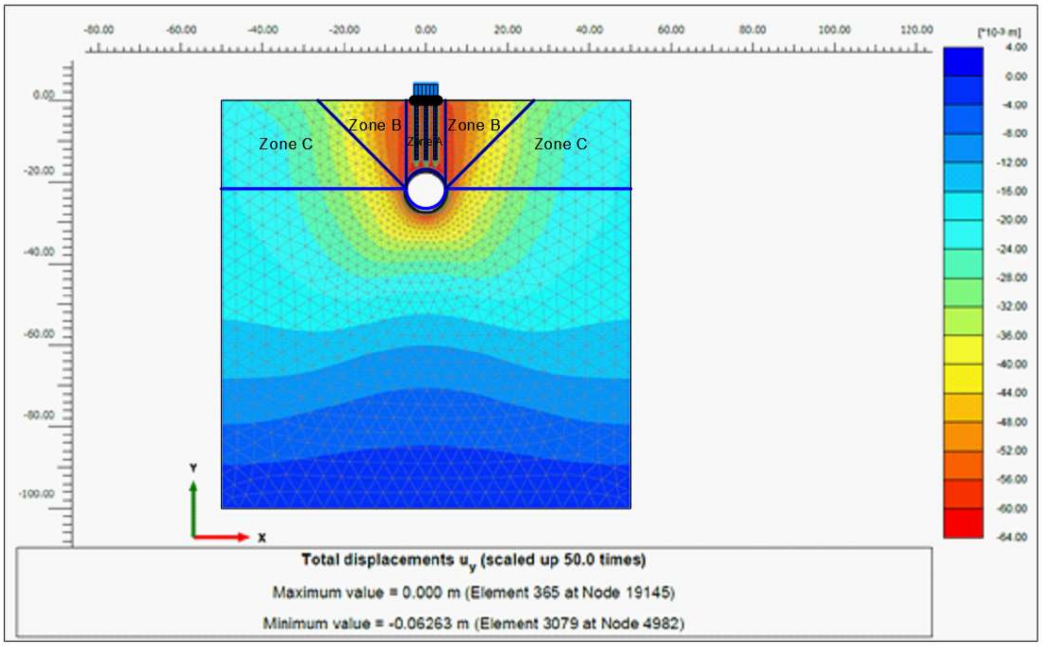


Figure 3-11: Displacement diagram for Analysis No.1

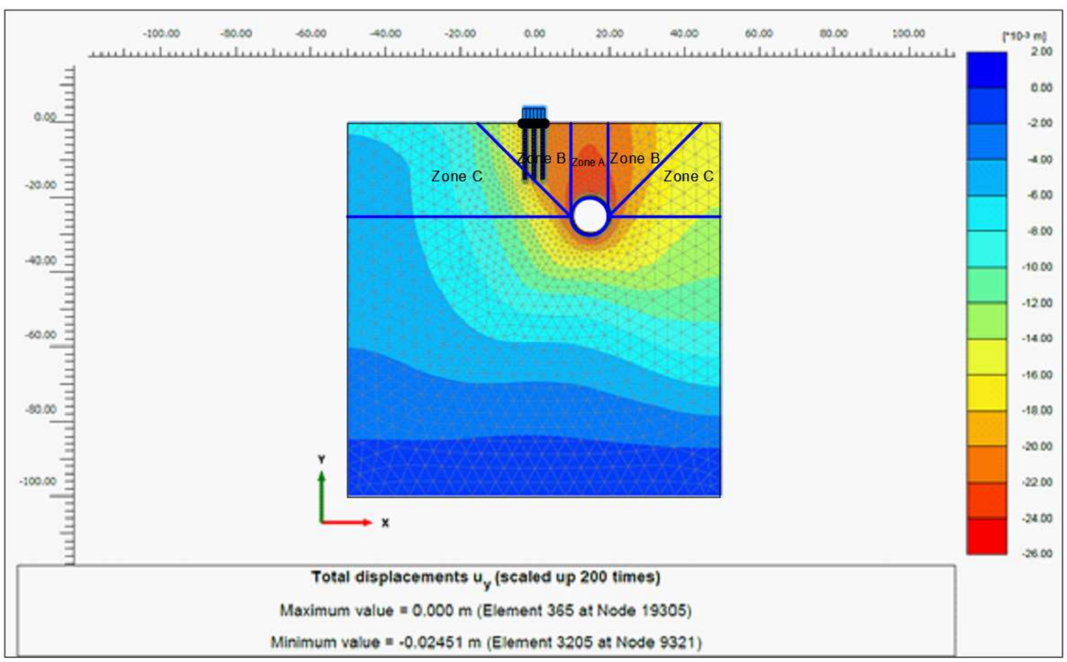


Figure 3-12: Displacement diagram for Analysis No.27

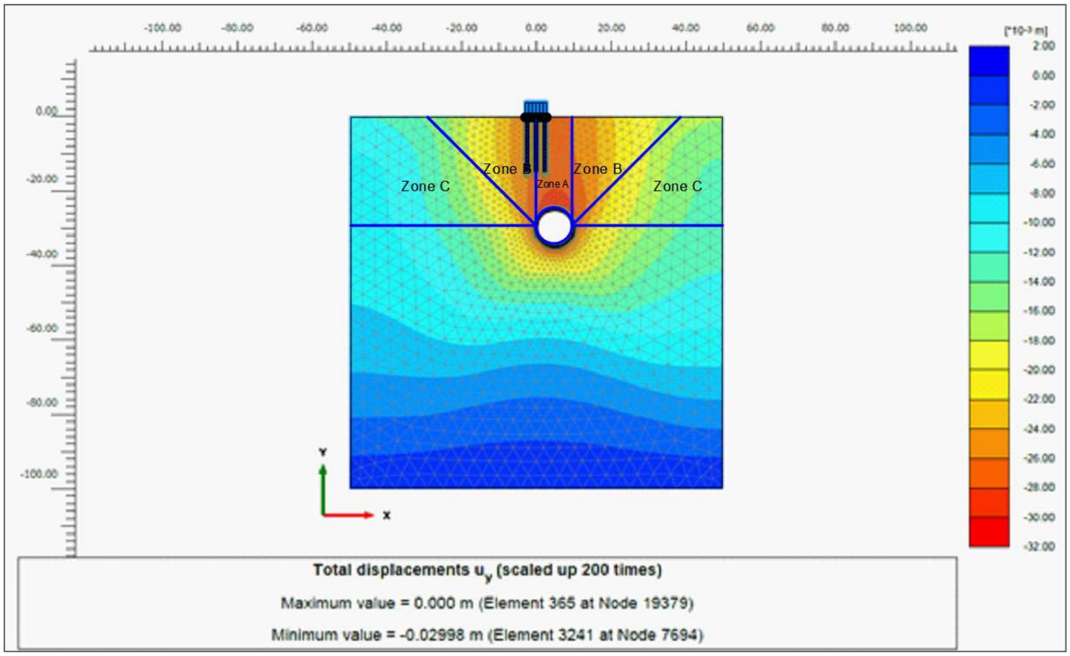


Figure 3-13: Displacement diagram for Analysis No.35

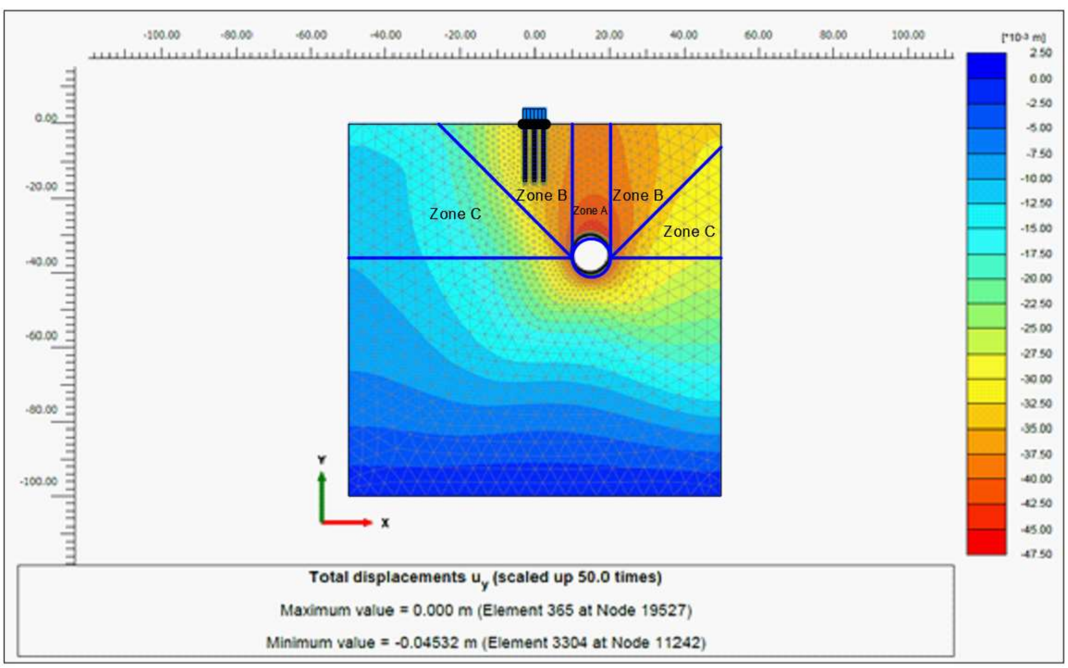


Figure 3-14: Displacement diagram for Analysis No.55

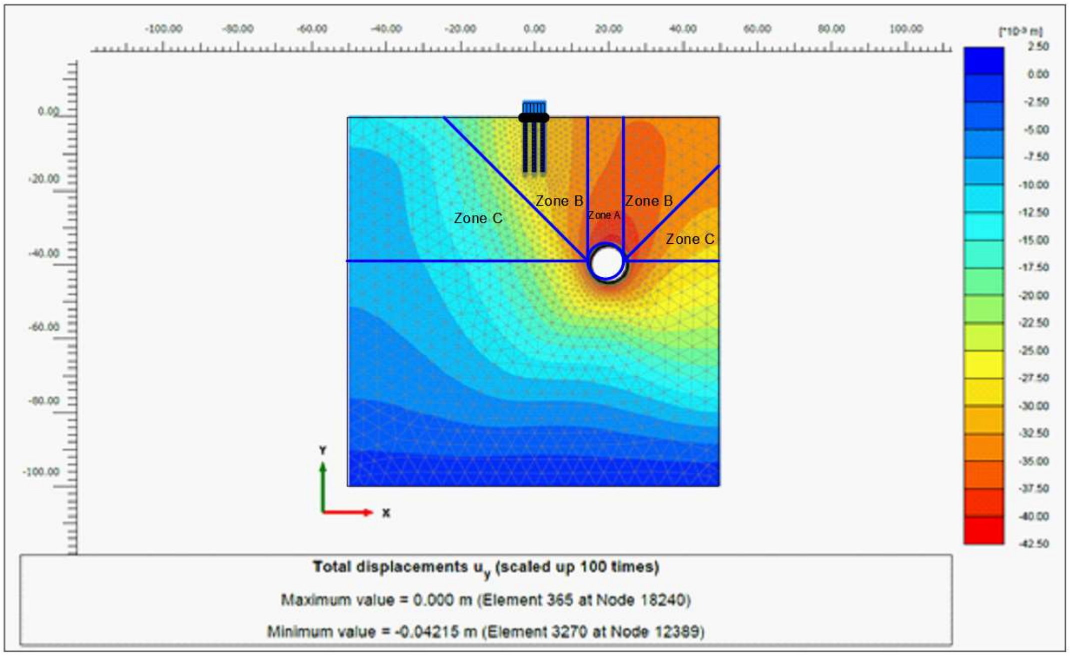


Figure 3-15: Displacement diagram for Analysis No.73

Displacement curves fit the influence zones proposed by Selemetas (2006) as it can be seen above figures. Detailed discussion about the analysis results are given in following chapter.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Analysis Results

75 analyses are conducted in the scope of this study. As it mentioned before, three main parameters are stated as z/D, x/D and deformation modulus of the soil. After the analyses conducted, additional settlement values of the foundation due to tunneling are noted. According to the analysis results, important conclusions are made. In this chapter, results are presented and discussed further. At the end of the completion of 75 analyses, foundation settlements are investigated and tabulated below Table 4-1.

Table 4-1: Analyses Results

Analysis Number	z/D	x/D	E (kPa)	Maximum Foundation Settlement (cm)	Maximum Tunnel Settlement (cm)
No.1	0.25	0.00	15000	6.020	6.282
No.2	0.25	0.00	25000	3.632	3.791
No.3	0.25	0.00	35000	2.583	2.696
No.4	0.25	0.50	15000	6.015	6.319
No.5	0.25	0.50	25000	3.637	3.814
No.6	0.25	0.50	35000	2.590	2.713
No.7	0.25	1.00	15000	5.614	6.203
No.8	0.25	1.00	25000	3.400	3.743
No.9	0.25	1.00	35000	2.426	2.663
No.10	0.25	1.50	15000	4.940	6.075
No.11	0.25	1.50	25000	2.992	3.665
No.12	0.25	1.50	35000	2.133	2.607
No.13	0.25	2.00	15000	4.236	6.080
No.14	0.25	2.00	25000	2.563	3.667
No.15	0.25	2.00	35000	1.826	2.608

Analysis Number	z/D	x/D	E (kPa)	Maximum Foundation Settlement (cm)	Maximum Tunnel Settlement (cm)
No.16	0.50	0.00	15000	5.381	5.750
No.17	0.50	0.00	25000	3.248	3.470
No.18	0.50	0.00	35000	2.310	2.469
No.19	0.50	0.50	15000	5.411	5.815
No.20	0.50	0.50	25000	3.271	3.508
No.21	0.50	0.50	35000	2.329	2.495
No.22	0.50	1.00	15000	5.178	5.817
No.23	0.50	1.00	25000	3.134	3.510
No.24	0.50	1.00	35000	2.216	2.484
No.25	0.50	1.50	15000	4.692	5.773
No.26	0.50	1.50	25000	2.841	3.483
No.27	0.50	1.50	35000	2.024	2.477
No.28	0.50	2.00	15000	4.112	5.794
No.29	0.50	2.00	25000	2.477	3.494
No.30	0.50	2.00	35000	1.764	2.485
No.31	1.00	0.00	15000	4.444	4.977
No.32	1.00	0.00	25000	2.683	3.003
No.33	1.00	0.00	35000	1.909	2.137
No.34	1.00	0.50	15000	4.479	5.026
No.35	1.00	0.50	25000	2.707	3.032
No.36	1.00	0.50	35000	1.927	2.157
No.37	1.00	1.00	15000	4.361	5.092
No.38	1.00	1.00	25000	2.637	3.072
No.39	1.00	1.00	35000	1.878	2.184
No.40	1.00	1.50	15000	4.098	5.156
No.41	1.00	1.50	25000	2.479	3.111
No.42	1.00	1.50	35000	1.766	2.212
No.43	1.00	2.00	15000	3.747	5.248
No.44	1.00	2.00	25000	2.266	3.165
No.45	1.00	2.00	35000	1.614	2.251
No.46	1.50	0.00	15000	3.751	4.391
No.47	1.50	0.00	25000	2.264	2.649
No.48	1.50	0.00	35000	1.611	1.885
No.49	1.50	0.50	15000	3.778	4.432
No.50	1.50	0.50	25000	2.282	2.674
No.51	1.50	0.50	35000	1.625	1.902
No.52	1.50	1.00	15000	3.719	4.512
No.53	1.50	1.00	25000	2.248	2.722
No.54	1.50	1.00	35000	1.600	1.935
No.55	1.50	1.50	15000	3.584	4.627
No.56	1.50	1.50	25000	2.167	2.790
No.57	1.50	1.50	35000	1.542	1.984

Analysis Number	z/D	x/D	E (kPa)	Maximum Foundation Settlement (cm)	Maximum Tunnel Settlement (cm)
No.58	1.50	2.00	15000	3.389	4.769
No.59	1.50	2.00	25000	2.048	2.876
No.60	1.50	2.00	35000	1.459	2.045
No.61	2.00	0.00	15000	3.205	3.914
No.62	2.00	0.00	25000	1.935	2.362
No.63	2.00	0.00	35000	1.377	1.680
No.64	2.00	0.50	15000	3.227	3.953
No.65	2.00	0.50	25000	1.949	2.384
No.66	2.00	0.50	35000	1.387	1.695
No.67	2.00	1.00	15000	3.197	4.032
No.68	2.00	1.00	25000	1.932	2.432
No.69	2.00	1.00	35000	1.375	1.729
No.70	2.00	1.50	15000	3.127	4.154
No.71	2.00	1.50	25000	1.890	2.506
No.72	2.00	1.50	35000	1.345	1.782
No.73	2.00	2.00	15000	3.024	4.316
No.74	2.00	2.00	25000	1.827	2.603
No.75	2.00	2.00	35000	1.301	1.851

Note that given settlements are the additional settlements due to tunneling, they do not contain settlements due to structure itself.

Moreover, moments due to tunneling are investigated and tabulated in Table 4-2.

Table 4-2: Pile Moments before and after Tunneling

				Left Pile	Middle Pile	Right Pile
Analysis Number	**z/D**	**x/D**	**E**	**Bending Moment (kN.m/m) (max.)**	**Bending Moment (kN.m/m) (max.)**	**Bending Moment (kN.m/m) (max.)**
Before Tunneling	-	-	15000	168.20	0.12	168.10
	-	-	25000	136.20	0.08	136.30
	-	-	35000	118.80	0.06	118.08
No.1	0.25	0.00	15000	234.80	0.10	235.00
No.2	0.25	0.00	25000	188.40	0.08	188.60
No.3	0.25	0.00	35000	162.70	0.07	162.90
No.4	0.25	0.50	15000	238.80	18.43	211.10
No.5	0.25	0.50	25000	189.40	12.49	171.70
No.6	0.25	0.50	35000	162.30	9.36	149.70

				Left Pile	Middle Pile	Right Pile
Analysis Number	z/D	x/D	E	Bending Moment (kN.m/m) (max.)	Bending Moment (kN.m/m) (max.)	Bending Moment (kN.m/m) (max.)
No.7	0.25	1.00	15000	224.40	24.21	188.80
No.8	0.25	1.00	25000	177.40	16.50	154.30
No.9	0.25	1.00	35000	151.80	12.41	135.30
No.10	0.25	1.50	15000	203.30	20.94	173.20
No.11	0.25	1.50	25000	161.20	14.36	141.50
No.12	0.25	1.50	35000	138.30	10.83	124.10
No.13	0.25	2.00	15000	182.10	14.00	162.40
No.14	0.25	2.00	25000	145.50	9.64	132.70
No.15	0.25	2.00	35000	125.60	7.26	116.30
No.16	0.50	0.00	15000	228.50	0.02	228.60
No.17	0.50	0.00	25000	183.30	0.02	183.30
No.18	0.50	0.00	35000	158.30	0.01	158.30
No.19	0.50	0.50	15000	230.80	13.64	209.80
No.20	0.50	0.50	25000	183.50	9.24	170.10
No.21	0.50	0.50	35000	157.70	6.94	148.10
No.22	0.50	1.00	15000	218.50	18.32	191.10
No.23	0.50	1.00	25000	173.20	12.29	155.60
No.24	0.50	1.00	35000	148.80	9.38	136.10
No.25	0.50	1.50	15000	201.30	17.25	176.20
No.26	0.50	1.50	25000	160.00	11.69	143.80
No.27	0.50	1.50	35000	137.50	8.75	125.90
No.28	0.50	2.00	15000	183.00	12.44	165.30
No.29	0.50	2.00	25000	146.40	8.68	134.80
No.30	0.50	2.00	35000	126.40	6.52	118.10
No.31	1.00	0.00	15000	215.80	0.02	215.80
No.32	1.00	0.00	25000	173.00	0.02	172.90
No.33	1.00	0.00	35000	149.50	0.02	149.50
No.34	1.00	0.50	15000	217.30	7.96	204.70
No.35	1.00	0.50	25000	173.30	5.37	165.20
No.36	1.00	0.50	35000	149.40	4.09	143.50
No.37	1.00	1.00	15000	208.90	11.21	191.90
No.38	1.00	1.00	25000	166.40	7.40	155.50
No.39	1.00	1.00	35000	143.40	5.52	135.60
No.40	1.00	1.50	15000	197.00	11.75	179.60
No.41	1.00	1.50	25000	157.20	7.82	146.10
No.42	1.00	1.50	35000	135.50	5.80	127.60

				Left Pile	Middle Pile	Right Pile
Analysis Number	**z/D**	**x/D**	**E**	**Bending Moment (kN.m/m) (max.)**	**Bending Moment (kN.m/m) (max.)**	**Bending Moment (kN.m/m) (max.)**
No.43	1.00	2.00	15000	183.70	9.64	169.90
No.44	1.00	2.00	25000	147.10	6.40	138.50
No.45	1.00	2.00	35000	127.20	4.72	121.20
No.46	1.50	0.00	15000	204.90	0.21	204.80
No.47	1.50	0.00	25000	164.30	0.19	164.20
No.48	1.50	0.00	35000	142.20	0.17	142.20
No.49	1.50	0.50	15000	206.70	5.54	197.90
No.50	1.50	0.50	25000	165.30	3.85	159.40
No.51	1.50	0.50	35000	142.80	2.98	138.30
No.52	1.50	1.00	15000	200.80	7.23	189.60
No.53	1.50	1.00	25000	160.60	4.81	153.40
No.54	1.50	1.00	35000	138.60	3.54	133.50
No.55	1.50	1.50	15000	192.20	7.49	180.90
No.56	1.50	1.50	25000	153.80	4.99	146.70
No.57	1.50	1.50	35000	132.90	3.74	128.00
No.58	1.50	2.00	15000	182.80	7.01	173.00
No.59	1.50	2.00	25000	146.80	4.69	140.70
No.60	1.50	2.00	35000	127.10	3.58	122.90
No.61	2.00	0.00	15000	196.20	0.07	196.20
No.62	2.00	0.00	25000	157.60	0.07	157.60
No.63	2.00	0.00	35000	136.60	0.07	136.50
No.64	2.00	0.50	15000	197.40	3.61	191.80
No.65	2.00	0.50	25000	158.30	2.50	154.50
No.66	2.00	0.50	35000	136.90	1.90	134.10
No.67	2.00	1.00	15000	194.00	5.06	186.10
No.68	2.00	1.00	25000	155.50	3.38	150.40
No.69	2.00	1.00	35000	134.50	2.54	130.90
No.70	2.00	1.50	15000	187.70	5.26	180.10
No.71	2.00	1.50	25000	150.70	3.59	146.00
No.72	2.00	1.50	35000	130.50	2.74	127.20
No.73	2.00	2.00	15000	181.10	5.36	174.50
No.74	2.00	2.00	25000	145.60	3.70	141.70
No.75	2.00	2.00	35000	126.20	2.85	123.70

For further discussion in the results of the analyses, variation of settlement with the aforementioned parameters are presented below.

4.1.1 Foundation Settlement vs z/D Graph

The first parameter found to be effective in foundation settlement is the ratio of z (the clear distance between the tunnel and the pile) to the pile diameter D. For different soil stiffnesses (E) and horizontal locations of the piles (x/D), comparative graphs are presented in Figures 4-1 to 4-3 below.

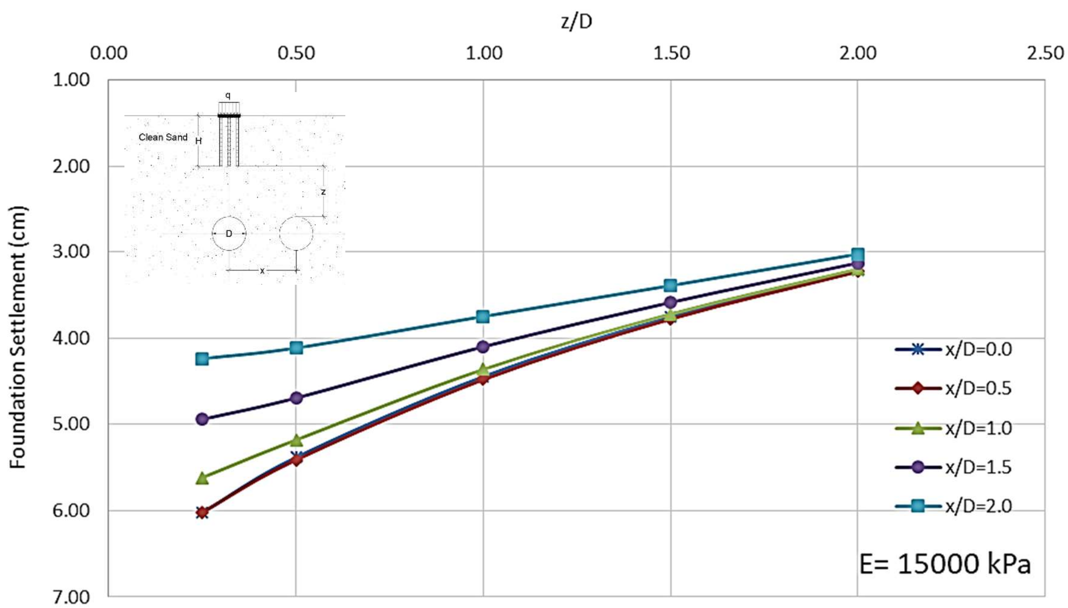


Figure 4-1: Foundation Settlement vs z/D Graph for E=15000kPa

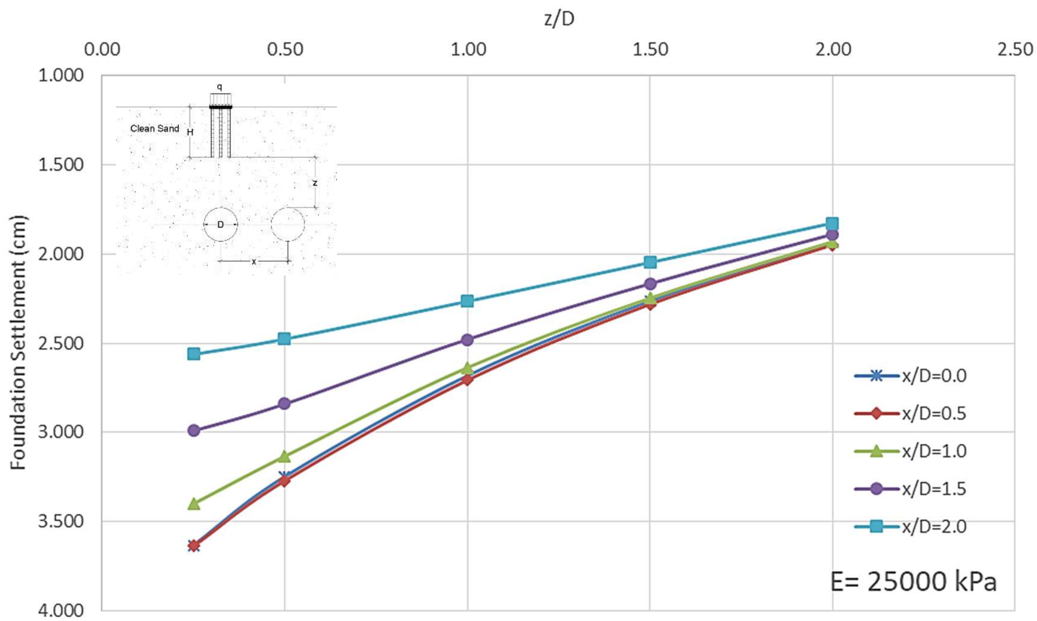


Figure 4-2: Foundation Settlement vs z/D Graph for E=25000kPa

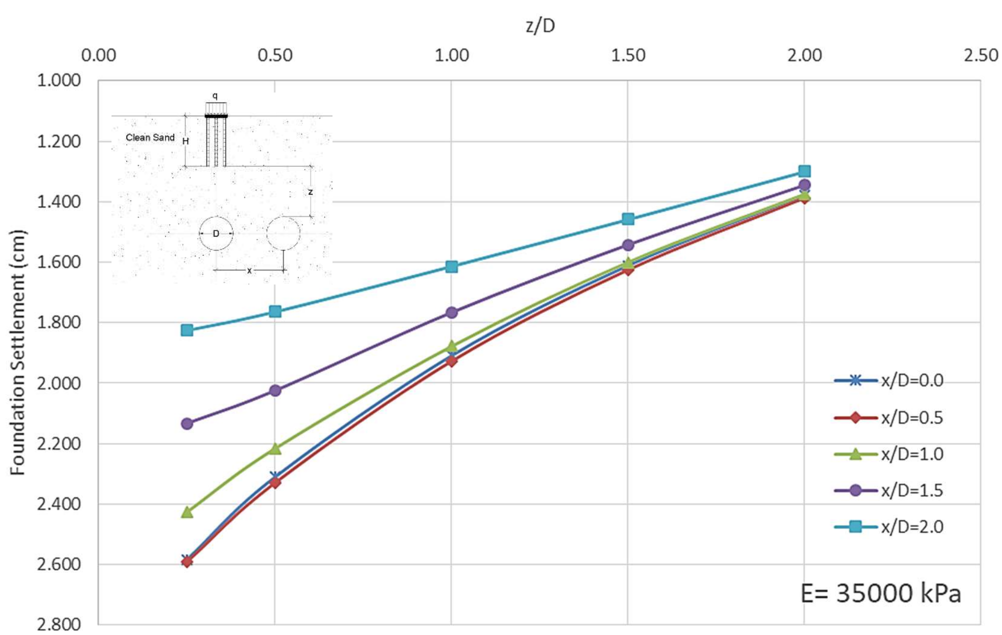


Figure 4-3: Foundation Settlement vs z/D Graph for E=35000kPa

In these figures, the foundation settlement is the settlement value resulted from the piles, i.e. the additional settlement of the foundation after the construction of the tunnel. According to Figures 4-7, 4-8 and 4-9, it could be stated that additional settlement of the foundation is decreased while tunnel is located further than the foundation center axis. With increasing ‘x/D’ and ‘z/D’, settlement is decreasing.

Figures indicate that deformation modulus also shows a similar trend on foundation settlement. With increasing deformation modulus, settlement is decreasing as expected. To understand this mentioned behavior, Figure 4-4 is drawn.

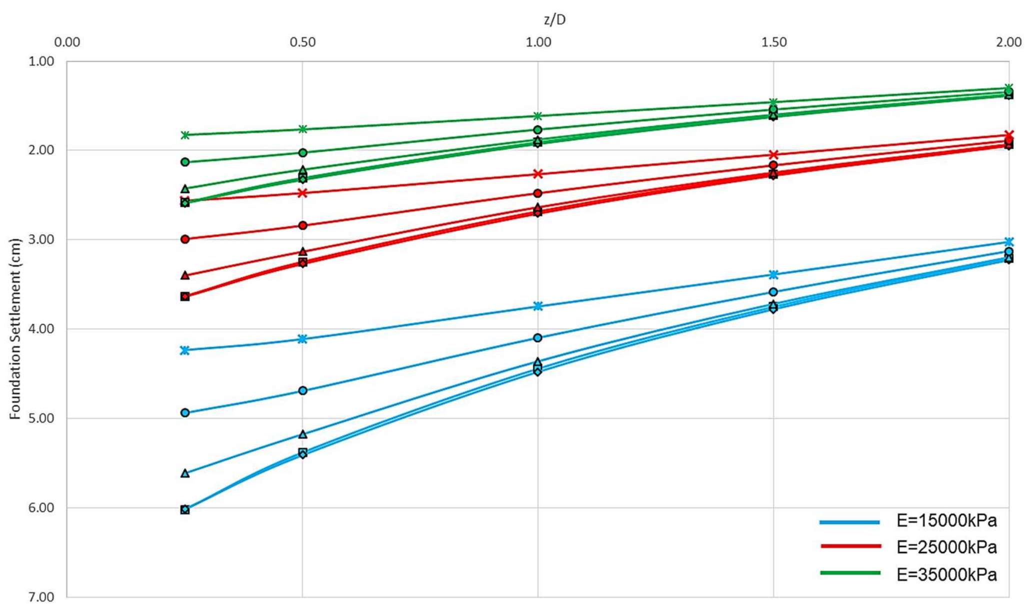


Figure 4-4: Foundation Settlement vs z/D Graph

Figure 4-4 indicates that deformation modulus, E, is a crucial parameter for the problem as well as the x/D and z/D parameters.

4.1.2 Surface Settlement Graphs

Another parameter that is thought to be important in the variation of surface settlements is the horizontal location of the tunnel with respect to the pile center which is shown by x/D. For this purpose, the graphs showing the variation of surface settlement with the change in the horizontal location is shown in Figures 4-5, 4-6, 4-7 for z/D=0.25, 1 and 2 for E=15000 kPa. The other graphs show similar trends so they are presented in Appendix1.

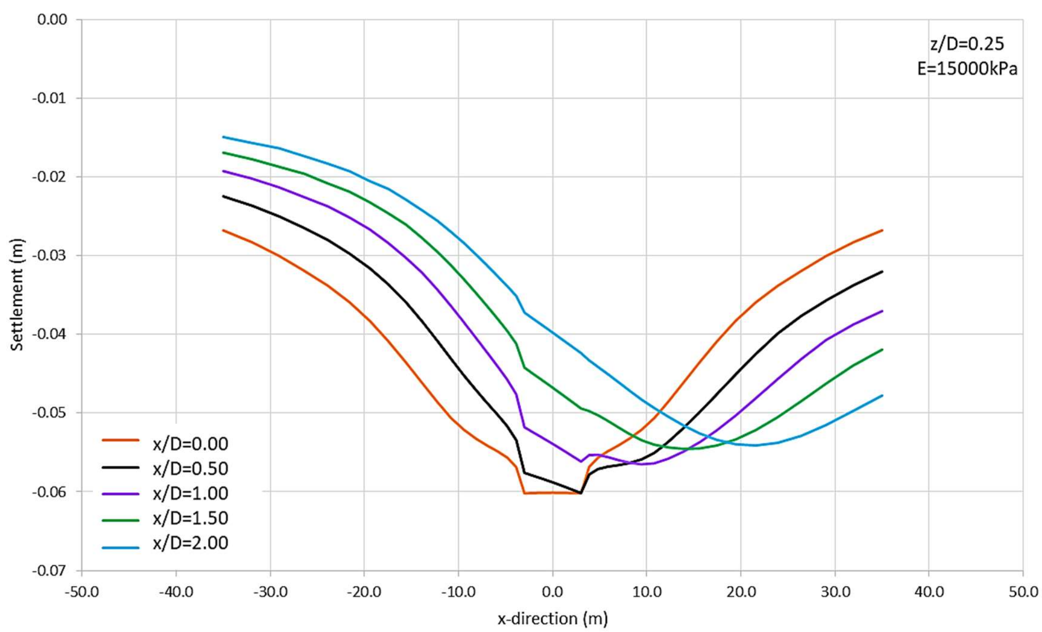


Figure 4-5: Surface Settlement Change with respect to x/D (z/D=0.25 & E=15000kPa)

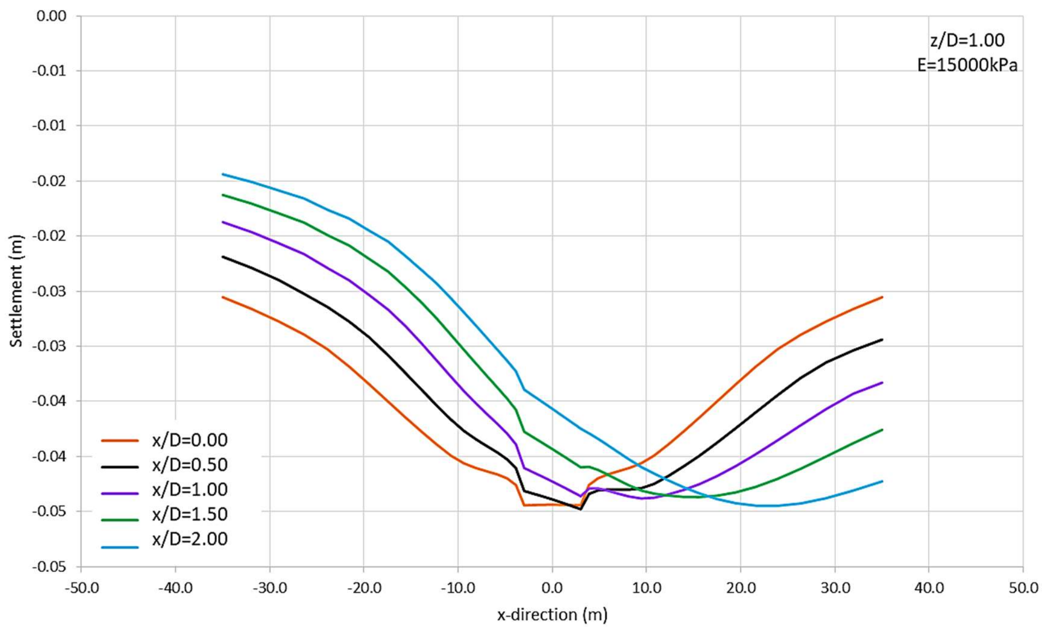


Figure 4-6: Surface Settlement Change with respect to x/D (z/D=1.00 & E=15000kPa)

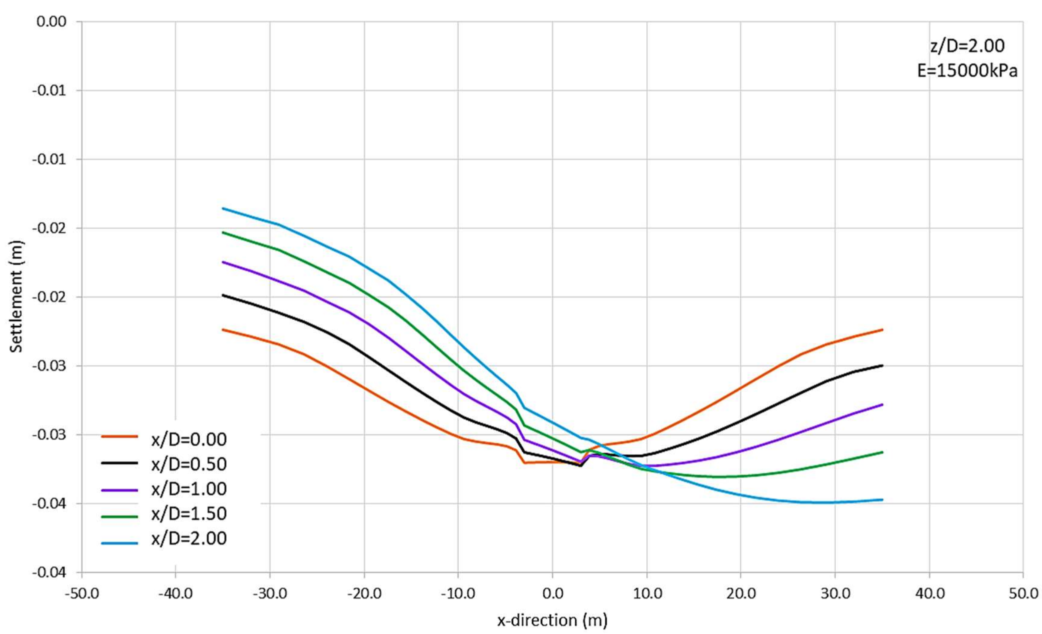


Figure 4-7: Surface Settlement Change with respect to x/D (z/D=2.00 & E=15000kPa)

Additionally, the change in settlement values at the surface are compared according to the change in the depth of the tunnel. For this purpose, the case with x/D=0 is chosen as the others show a similar trend with each other (Figure 4-8 to 4-10). The other graphs are presented in Appendix 1.

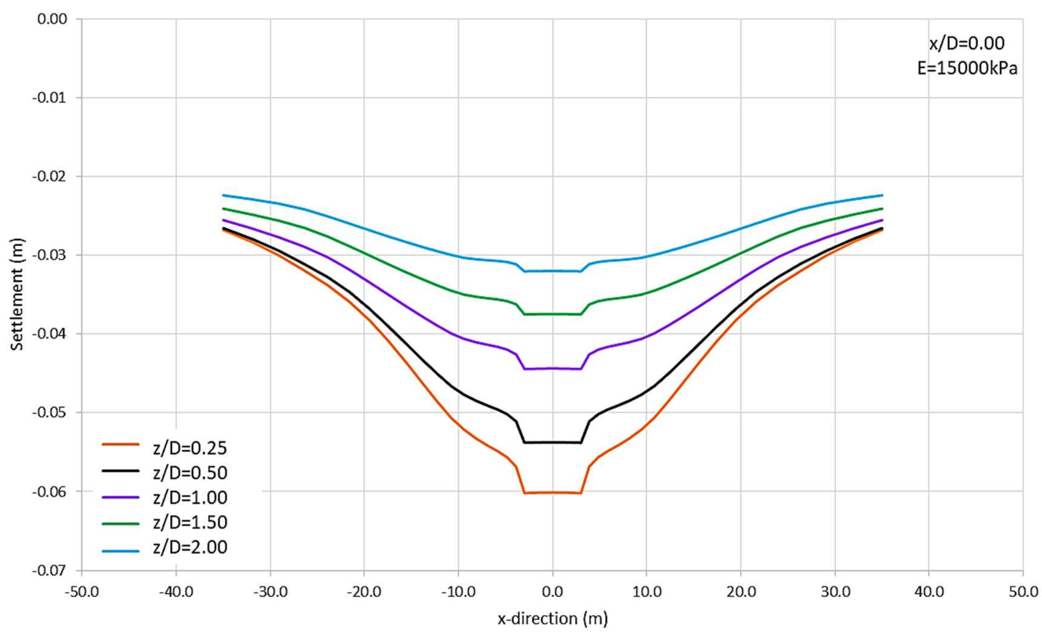


Figure 4-8: Surface Settlement Change with respect to z/D (x/D=0.0 & E=15000kPa)

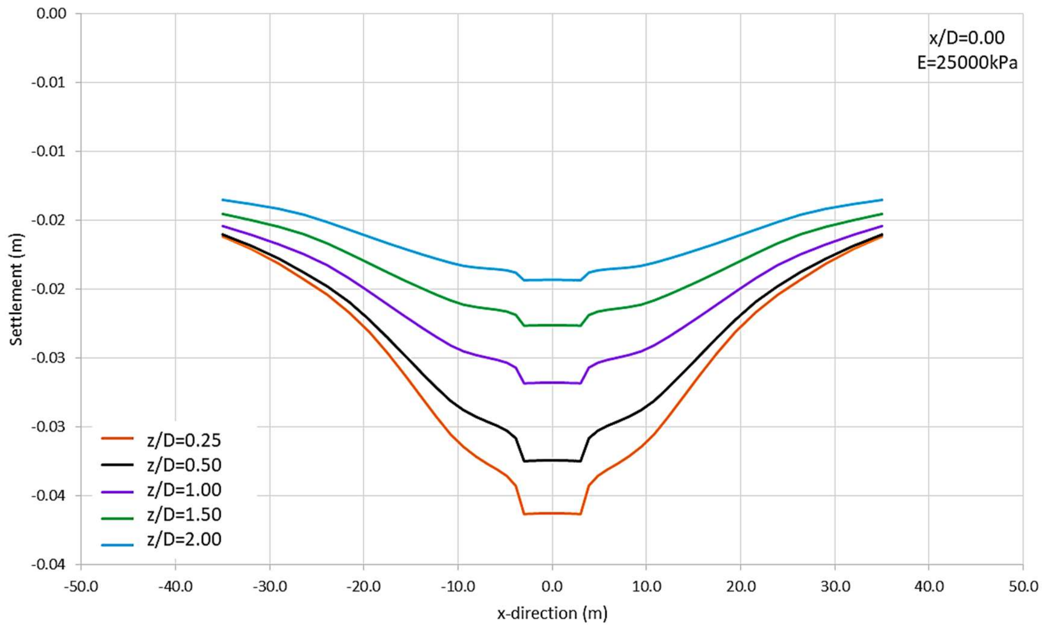


Figure 4-9: Surface Settlement Change with respect to z/D (x/D=0.0 & E=25000kPa)

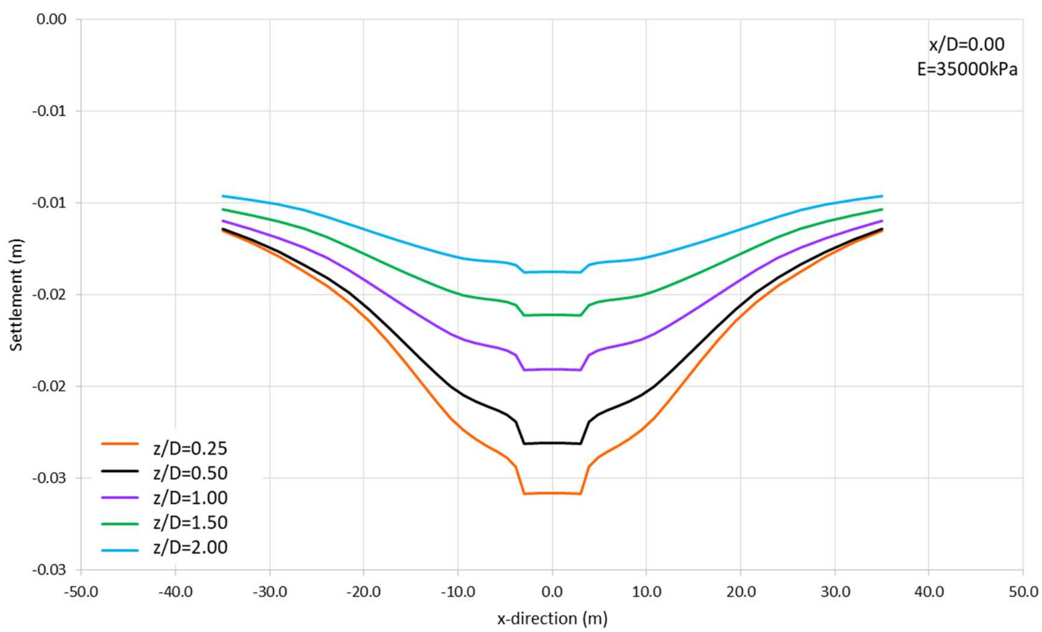


Figure 4-10: Surface Settlement Change with respect to z/D (x/D=0.0 & E=35000kPa)

As it can be seen from the graphs above, as the tunnel goes deeper, the value of the settlement decreases with increasing z/D value.

According to the presented graphs, it could be said that while the tunnel is located further than the foundation center axis in x-direction, total settlement values are decreased. However, differential settlement might become more important. As it could be seen in figures, the most critical cases for every analysis are the ones when x/D=2.0. Therefore, differential settlement values are calculated and given in below table for the x/D=2.0 cases (Table 4-3).

Table 4-3: Differential Settlement Values for x/D=2.0

Analysis Number	z/D	x/D	E (kPa)	Differential Settlement (‰)
No.13	0.25	2.00	15000	0.85
No.14	0.25	2.00	25000	0.54
No.15	0.25	2.00	35000	0.40
No.28	0.50	2.00	15000	0.74
No.29	0.50	2.00	25000	0.48
No.30	0.50	2.00	35000	0.35
No.43	1.00	2.00	15000	0.59
No.44	1.00	2.00	25000	0.38
No.45	1.00	2.00	35000	0.27
No.58	1.50	2.00	15000	0.46
No.59	1.50	2.00	25000	0.29
No.60	1.50	2.00	35000	0.21
No.73	2.00	2.00	15000	0.36
No.74	2.00	2.00	25000	0.23
No.75	2.00	2.00	35000	0.17

Differential settlement values in Table 4-3 indicate that they are all less than 1‰ which means additional differential settlements are not in critical state. However, in design, differential settlement criterion should always be checked.

4.1.3 Moment Graphs

Moments before and after tunneling are investigated in detail by the help of moment graphs through pile length which are presented in Figures 4-11 to 4-13. In these figures, moments are given for the left pile which is in more critical state than the other considering the moment susceptibility. The other graphs show similar trends so they are presented in Appendix1.

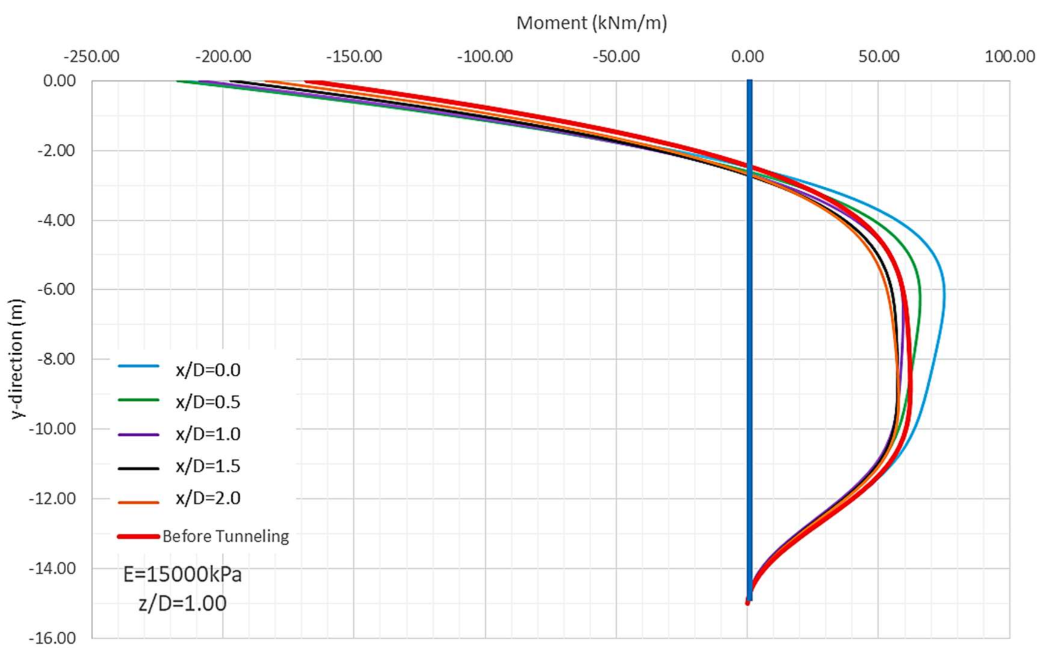


Figure 4-11: Bending Moments for E=15000kPa and z/D=1.0

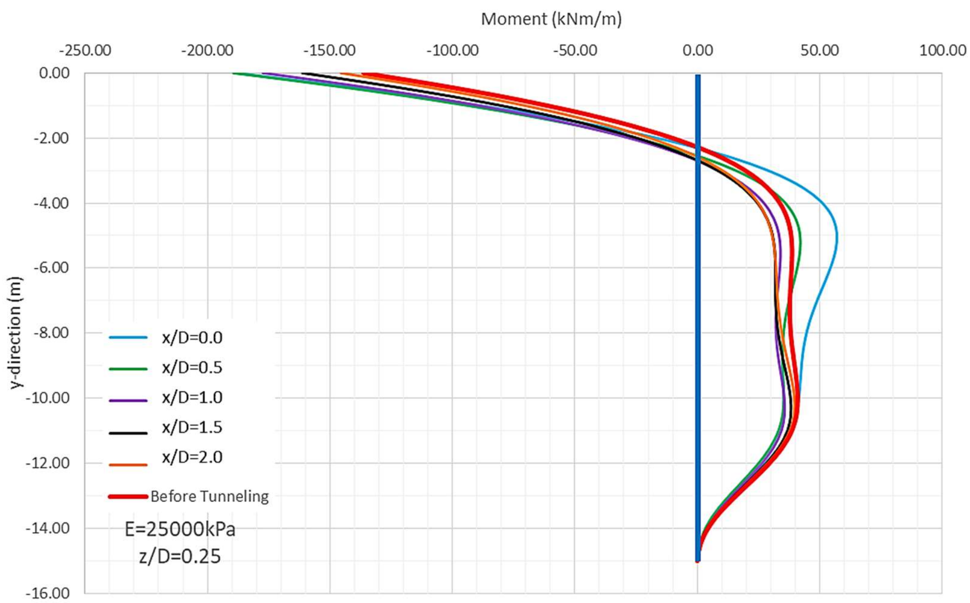


Figure 4-12: Bending Moments for E=25000kPa and z/D=0.25

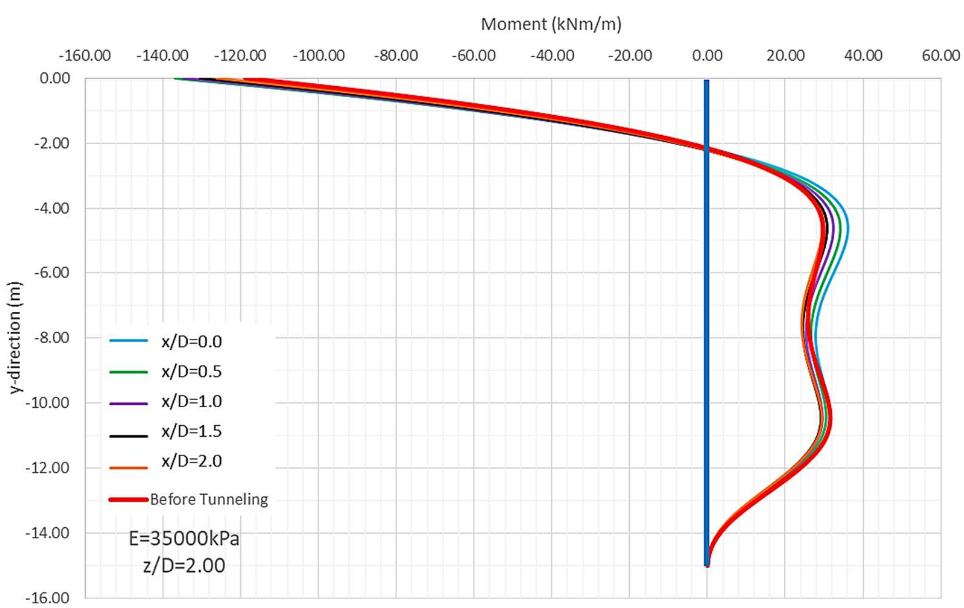


Figure 4-13: Bending Moments for E=35000kPa and z/D=2.0

From the graphs above, it can be said that, the tunnel increases the maximum bending moment which occurs at the pile cap regardless of its location or depth. However, if we look at the distribution of the moment along the length of the pile, the moment on the pile increases for tunnel located just beneath the piles.

However, when the horizontal location of the tunnel moves away from the pile (for cases x/D>1.0), there is a slight decrease in the bending moment. This is same for all soils with different stiffness values. The most affected pile is the left one since tunnel location is changed in +x-direction (to the right). This might be because of the influence zones. When the tunnel located in the influence zone, its effects becomes more crucial. Once tunnel is getting further in the influence zone, its effect becomes less effective.

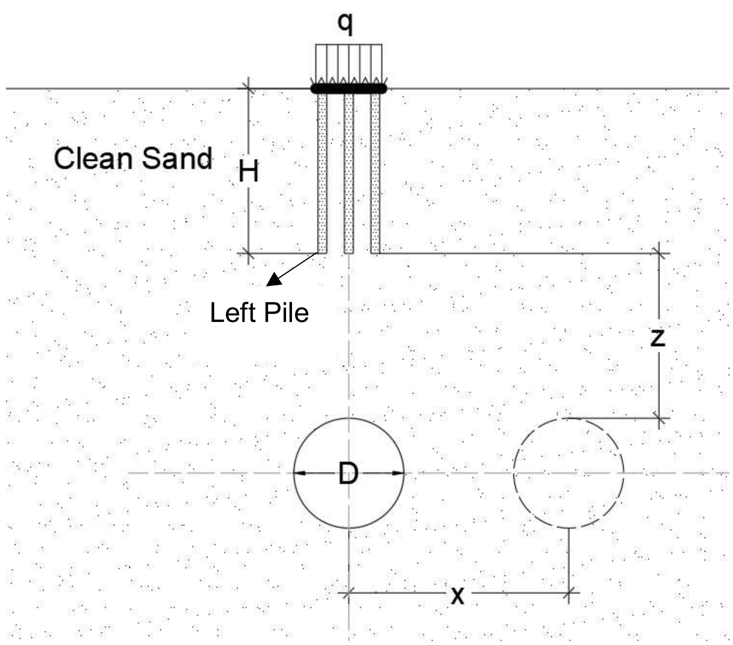


Figure 4-14: Location of the left pile

Increasing deformation modulus has a decreasing effect on extra pile moment because the displacement is decreasing with increasing deformation modulus of the soil. As expected, the moments on the piles decrease with increasing tunnel depth. This is because, when tunnel is located deeper in the influence zones, its effects on both settlement and pile forces are decreasing.

4.2 Simplified Procedure for Determining Additional Settlement due to Tunneling

In this section, the data from analysis results will be used for creating a simplified equation. It is expected with this simple equation that someone can find an estimated value for additional settlement of pile foundations due to tunneling with any E, x/D and z/D parameters. Note that, this equation is only valid for cohesionless soils.

First of all, depending on the results of the analysis, the most effective parameters in settlement is determined. According to the results presented in the sections above, the settlement values decrease with increasing stiffness of soil (E), lateral position of the tunnel and the depth of the tunnel. Therefore, the main form of the settlement equation is as follows;

$$S_x \approx \frac{1}{E+\frac{x}{D}+\frac{z}{D}} \qquad \text{Equation 4-1}$$

In order to have a representative function, some random constants are introduced and the equation becomes;

$$S_x = \frac{1}{(\alpha_1 * E)^{\alpha_4} + \left(\alpha_2 * \frac{x}{D}\right)^{\alpha_5} + \left(\alpha_3 * \frac{z}{D}\right)^{\alpha_6}} \qquad \text{Equation 4-2}$$

Then, $\alpha_1, \alpha_2, \alpha_3, \alpha_4, \alpha_5$ and α_6 become the random variables which all are assumed as identically distributed and independent and having a probability function ($f(X|\alpha)$) of normal distribution. If each random variable has the same probability distribution and are mutually independent, then they are identically distributed and independent in probability theory. This is the main assumption in order to use the Normal Maximum Likelihood Estimation.

The general form of the probability density function of normal distribution is given below.

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2} \qquad \text{Equation 4-3}$$

where

σ= standard deviation

μ= mean or expectation of the distribution (in this case it is the analysis results which are the expected values)

One of the many algorithms for estimating the parameters is the maximum likelihood estimation (MLE). The main idea behind MLE is to select that parameters that make the observed data the most likely. Mathematically speaking, the likelihood is,

$$L(\alpha) = \prod_{i=1}^{n} f(X_i|\alpha) \qquad \text{Equation 4-4}$$

Maximum likelihood estimation assumes that,

$$\hat{\alpha} = \underset{\alpha}{argmax}\, L(\alpha) \qquad \text{Equation 4-5}$$

$\hat{\alpha}$ represents the best choice of values for the parameters. Argmax is the short form of Arguments of the Maxima which are the points, or elements, of the domain of some function at which the function values are maximized. One of the properties of argmax is that argmax of a function is the same as the argmax of the log of function since log is a monotone function. Therefore, log of MLE becomes Log Likelihood function (LL);

$$LL(\alpha) = logL(\alpha) = log\prod_{i=1}^{n} f(X_i|\alpha) = \sum_{i=1}^{n} log\, f(X_i|\alpha) \qquad \text{Equation 4-6}$$

To find the values for $\alpha_1, \alpha_2, \alpha_3, \alpha_4, \alpha_5$ and α_6 which maximize the $\sum_{i=1}^{n} log\, f(X_i|\alpha)$ function, Solver add-in in Excel is used.

Note: Since probability function of normal distribution is a type of natural exponential function, by taking the logarithm, natural logarithm function, $lnL(\alpha)$, is used.

After the required calculations, $\alpha_1, \alpha_2, \alpha_3, \alpha_4, \alpha_5$ and α_6 are found as;

α_1=0.015 α_2=0.168 α_3=0.117 α_4=1.3 α_5=2.5 α_6=1.2

And the proposed equation becomes;

$$s_x = \frac{1}{(0.015*E)^{1.3} + \left(0.168*\frac{x}{D}\right)^{2.5} + \left(0.117*\frac{z}{D}\right)^{1.2}} \qquad \text{Equation 4-7}$$

where,

s_x: Additional settlement due to tunneling [cm]

E: Deformation modulus of the soil [MPa]

D: Tunnel diameter [m]

z: Clear distance between pile toe and tunnel cover [m]

x: Distance between tunnel center axis and the pile group center axis. [m]

Settlements obtained from Plaxis analyses vs settlements estimated from the proposed equation graph is drawn and it could clearly be seen that the results are very close to each other.

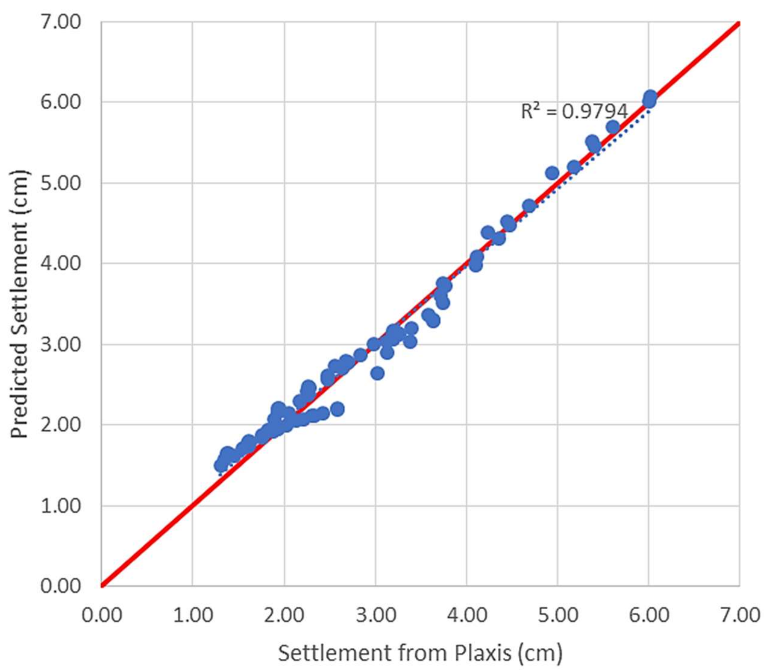


Figure 4-15: Settlements obtained from Plaxis analyses vs settlements estimated from the proposed equation graph

CHAPTER 5

CONCLUSION

Population on earth is increasing by the time passes. People prefer living in urban areas, like cities, for more work opportunities, social activities, health services and etc. This situation creates more need of buildings for multipurpose use. Therefore, much more space is occupied in urban areas. Considering the crowd of the people and the buildings, underground structures are getting more and more popular as a solution of many problems, such as transportation, piping, electricity and etc. Since the ground level is occupied by several buildings, infrastructures have to be constructed under them. When this is the case, investigation of the effects of tunneling becomes an interesting topic for civil engineers. How the soil layers and foundations behave during and after the construction of tunnels gains prominence in geotechnical point of view.

In this study, how tunneling affects piled foundations is investigated. First of all, background research has been completed. Previous studies show that generally three influence zones are existed under foundations. Tunnel location is very important when considering the influence zones. Its depth and distance to the foundation center are the main location parameters which are also the main parameters in this research. Soil type and its deformability is another important variable so one of the three main parameters for this study is the deformation modulus of the soil. Soil type is not chosen as a variable parameter for this particular work.

Several finite element analyses have been conducted to collect data. Plaxis software has been used for the analysis. This software performs two-dimensional analysis of deformation and stability in geotechnical engineering and it works with finite element method. 75 analyses are conducted within the scope of this study and they are tabulated in below table.

Table 5-1: Analyses and their parameters

z/D=0.25	x/D=0.0	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=0.5	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=1.0	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=1.5	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=2.0	E=15000kPa, E=25000kPa, E=35000kPa
z/D=0.5	x/D=0.0	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=0.5	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=1.0	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=1.5	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=2.0	E=15000kPa, E=25000kPa, E=35000kPa
z/D=1.0	x/D=0.0	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=0.5	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=1.0	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=1.5	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=2.0	E=15000kPa, E=25000kPa, E=35000kPa
z/D=1.5	x/D=0.0	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=0.5	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=1.0	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=1.5	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=2.0	E=15000kPa, E=25000kPa, E=35000kPa
z/D=2.0	x/D=0.0	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=0.5	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=1.0	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=1.5	E=15000kPa, E=25000kPa, E=35000kPa
	x/D=2.0	E=15000kPa, E=25000kPa, E=35000kPa

According to the results given in the previous chapters, the followings can be concluded:

- Additional settlement of the foundation is decreased while tunnel is located further than the foundation center axis, i.e. with increasing 'x/D' and 'z/D', settlement is decreasing.
- With increasing deformation modulus (E), settlement is decreasing as expected.
- While the tunnel is located further than the foundation center axis in x-direction, total settlement values are also decreased. However, differential settlement might become more important. Therefore, in design, differential settlement criterion should always be checked.
- Tunneling creates additional moments on piles, especially on the further piles where tunnel is located. In this study, the most affected pile is the left one since tunnel location is changed in +x-direction (to the right).
- Increasing deformation modulus has a decreasing effect on additional pile moment because the displacement is decreasing with increasing deformation modulus of the soil.

Besides from the conclusions above, a simplified equation is proposed to estimate the additional settlement due to tunneling using the easily obtainable parameters; E, x/D and z/D and presented in Equation 5-1.

$$s_x = \frac{1}{(0.015 * E)^{1.3} + \left(0.168 * \frac{x}{D}\right)^{2.5} + \left(0.117 * \frac{z}{D}\right)^{1.2}} \qquad \text{Equation 5-1}$$

where,

s_x: Additional settlement due to tunneling [cm]

E: Deformation modulus of the soil [MPa]

D: Tunnel diameter [m]

z: Clear distance between pile toe and tunnel cover [m]

x: Distance between tunnel center axis and the pile group center axis. [m]

It should be kept in mind that this equation is only valid for cohesionless soils and tunnels excavated at once (TBM tunnels). These results and the equation can be used in preliminary design stages as it depends on a limited number of numerical analysis results. The scope of this study is limited with respect to the soil type, a rigid tunnel and certain rigidity ratios. This study only covers cohesionless soils with limited deformation moduli.

For increasing the reliability of this equation

- Additional studies should be performed for cohesive soils.
- The range of parameters (x/D, z/D and etc.) can be expanded.
- Different tunnel construction methodology (NATM) can be used in the analyses.
- Case study data can be used.

in the future studies.

REFERENCES

Al-Omari, R. R., Al-Soud, M. S., Al-Zuhairi, O. I., Effect of Tunnel Progress on the Settlement of Existing Piled Foundation, Studia Geotechnica et Mechanica, 41(2); 102–113, 2019.

Basile, F., Effects of tunneling on pile foundations, Soils and Foundations, The Japanese Geotechnical Society, 54(3), 280-295, 2014.

Cham W.M., Singapore Case Histories on Performance of Piles Subjected to Tunnelling-Induced Soil Movement, International Journal of Geoengineering Case histories, http://casehistories.geoengineer.org, Vol.3, Issue 3, p.128-148, 2016.

Chen, L. T., Poulos, H.G., Loganathan, N., Pile Responses Caused by Tunneling, Journal of Geotechnical and Geoenvironmental Engineering, 126(6), 580-581, 1999.

Cheng, C .Y., Dasari, G. R., Leung C. F., Chow, Y. K., & Rosser, H. B., 3D Numerical Study of Tunnel-Soil-Pile Interaction, Tunneling and Underground Space Technology, 19(4-5), 381-382, 2004.

Clayton, C.R.I., The standard penetration test (SPT): methods and use Construction Industry Research and Information Association, 129pp, 1995

Dias, T. G. S., Bezuijen, A., Data Analysis of Pile Tunnel Interaction, Journal of Geotechnical and Geoenvironmental Engineering, 141(12), 2015

Franza, A., Tunneling and its effects on piles and piled structures, Ph.D. Thesis, University of Nottingham, 2016

Grant, R. J., Taylor, R. N., Tunneling-induced ground movements in clay, Proceedings of the Institution of Civil Engineers: Geotechnical Engineering, 143(1), 43-55, 2000

Huang, M., Zhang, C., Li, Z., A simplified analysis method for the influence of tunneling on grouped piles, Tunneling and Underground Space Technology, 24(4), 410-422, 2009

Jacobsz, S. W., Standing, J. S., Mair, R. J., The influence of tunneling on piled foundations, 16th International Conference on Soil Mechanics and Geotechnical Engineering, Osaka, 2015

Kaalberg, F. J., Teunissen, E. A. H., Van Tol, A. F., Bosch, J. W., Dutch research on the impact of shield tunneling on pile foundation, Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering: Geotechnology in Harmony with the Global Environment, 2005

Kitiyodom, P., Matsumoto, T., Kawaguchi, K., A simplified analysis method for piled raft foundations subjected to ground movements induced by tunneling, International Journal for Numerical and Analytical Methods in Geomechanics, 29(15), 2005

Kulhawy, F. H., Mayne, P. W., Manual on Estimating Soil Properties for Foundation Design, 1990

Loganathan, N. P., Ground and Pile-Group Responses due to Tunneling, Soils and Foundations, Japanese Geotechnical Society, 41(1), 57-67, 2001

Mair, R. J., Williamson, M. G., The influence of tunneling and deep excavation on piled foundations, Geotechnical Aspects of Underground Construction in Soft Ground, Korean Geotechnical Society, Korea, 2014

Marshall, A. M., Tunnel-Pile Interaction Analysis Using Cavity Expansion Methods, Journal of Geotechnical and Geoenvironmental Engineering, 138(10), 1237-1246, 2012

Marshall, A. M., Haji, T., An analytical study of tunnel-pile interaction, Tunneling and Underground Space Technology, 43-51, 2015

Mroueh, H., Shahrour, T., Three-dimensional finite element analysis of the interaction between tunneling and pile foundation, International Journal for Numerical and Analytical Methods in Geomechanics, 217-230, 2002

Ng, C. W. W., Lu, H., Peng, S. Y., Three-dimensional centrifuge modelling of the effects of twin tunneling on an existing pile, Tunneling and Underground Space Technology, 2013

Pramthawee, P., Jongpradist, P., Kongkitkul, W., Evaluation of hardening soil model on numerical simulation of behaviors of high rockfill dams, Songklanakarin Journal of Science and Technology, 2011

Piech, C., Sahami, M., 11. Parameter Estimation, https://web.stanford.edu/class/cs109/reader/11%20Parameter%20Estimation.pdf, May, 2017 (date accessed: December 1, 2020)

PLAXIS CONNECT Edition 20.04, PLAXIS 2D-Reference Manual, 2020 https://communities.bentley.com/products/geotech-analysis/w/plaxis-soilvision-wiki/46137/manuals---plaxis (date accessed: November, 2020)

PLAXIS CONNECT Edition 20.04, General Information Manual, 2020 https://communities.bentley.com/products/geotech-analysis/w/plaxis-soilvision-wiki/46137/manuals---plaxis (date accessed: November, 2020)

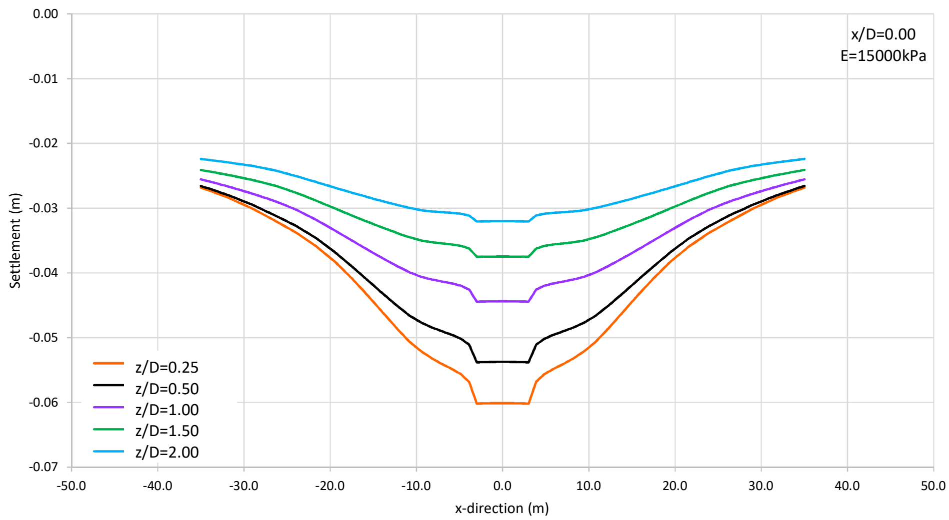
Selemetas, D., Standing, J. R., Mair, R. J., The response of full-scale piles to tunneling, International Society for Soil Mechanics and Geotechnical Engineering, 2006

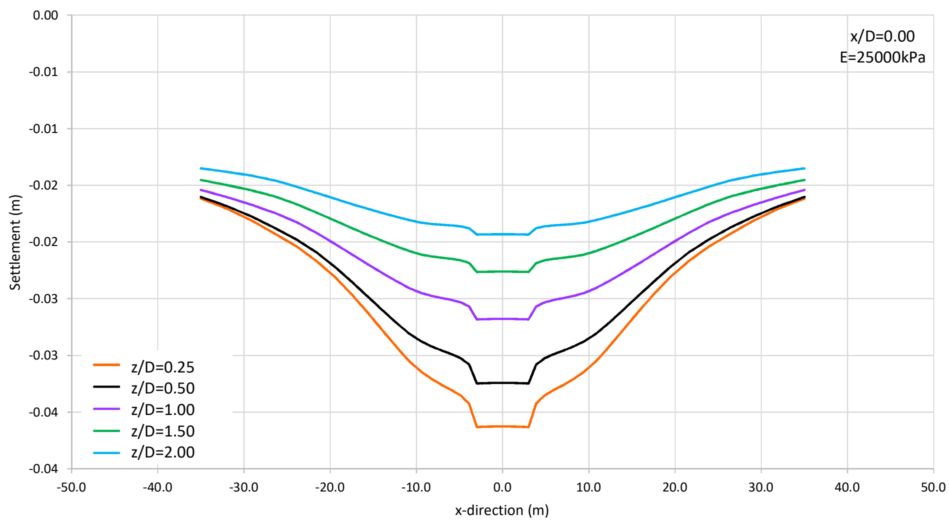
Son, M., Cording, E. J., Estimation of Building Damage Due to Excavation-Induced Ground Movements, Journal of Geotechnical and Geoenvironmental Engineering, 131(2), 162-177, 2005

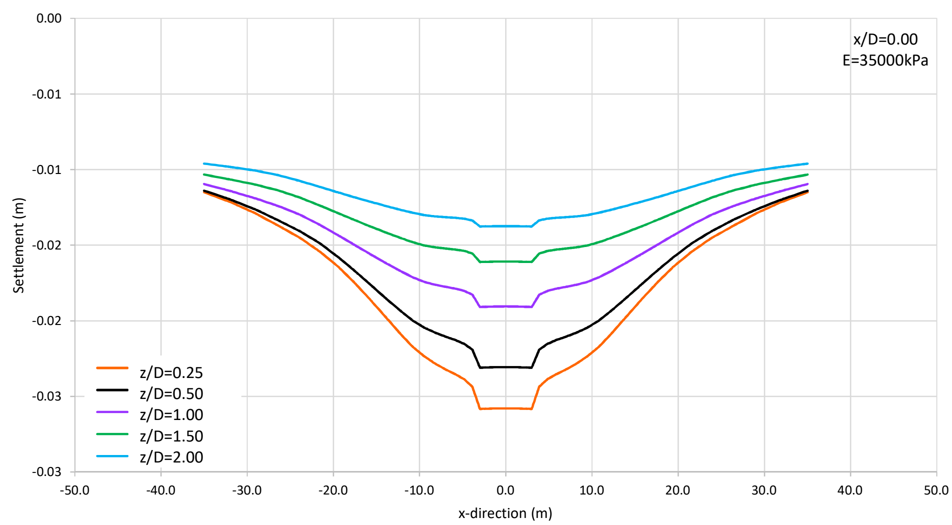
APPENDIX-1. ADDITIONAL GRAPHS

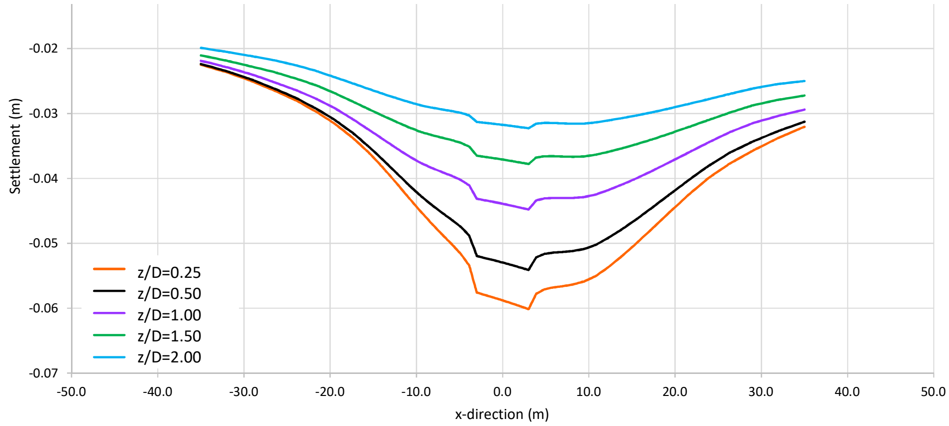
APPENDIX-1.1

Surface Settlement Change with respect to z/D

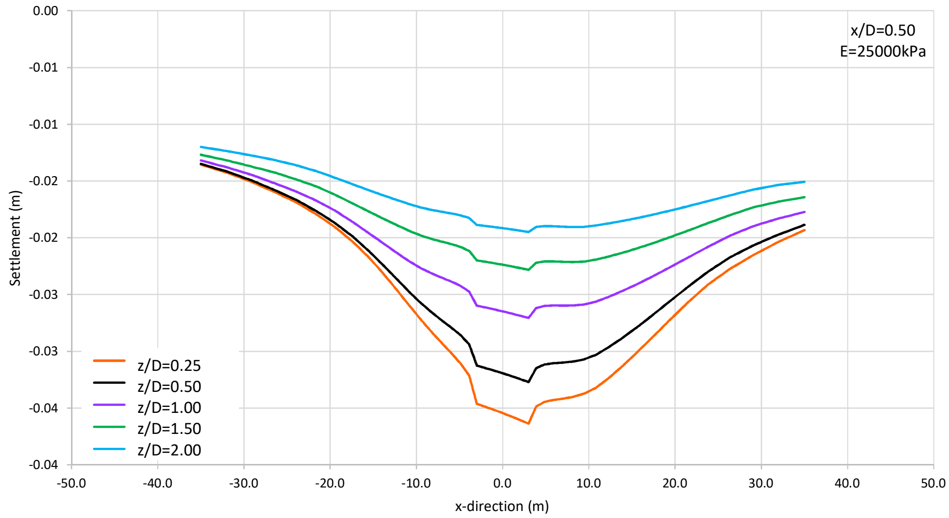

x/D=0.00
E=15000kPa
Settlement (m)
x-direction (m)
z/D=0.25
z/D=0.50
z/D=1.00
z/D=1.50
z/D=2.00


x/D=0.00
E=25000kPa
Settlement (m)
x-direction (m)
z/D=0.25
z/D=0.50
z/D=1.00
z/D=1.50
z/D=2.00

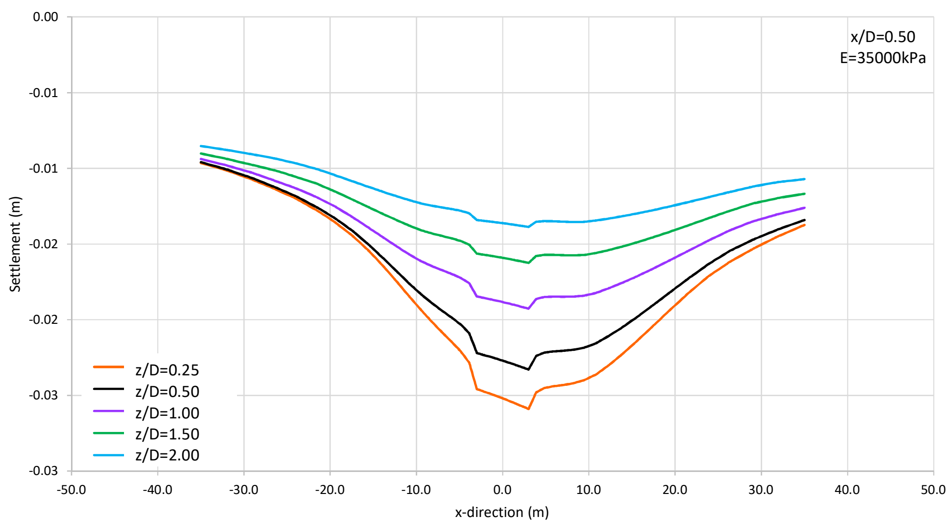

x/D=0.00
E=35000kPa
Settlement (m)
x-direction (m)
z/D=0.25
z/D=0.50
z/D=1.00
z/D=1.50
z/D=2.00



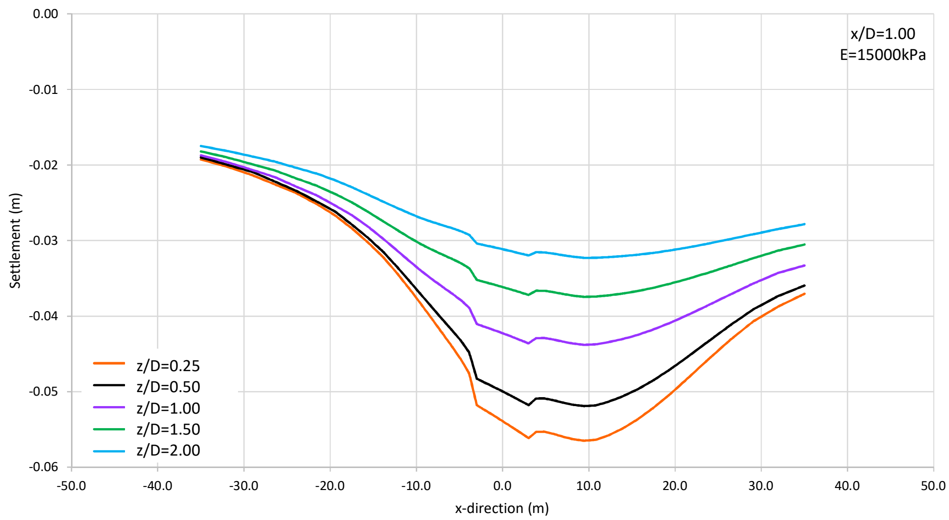
Settlement (m)
x-direction (m)
z/D=0.25
z/D=0.50
z/D=1.00
z/D=1.50
z/D=2.00

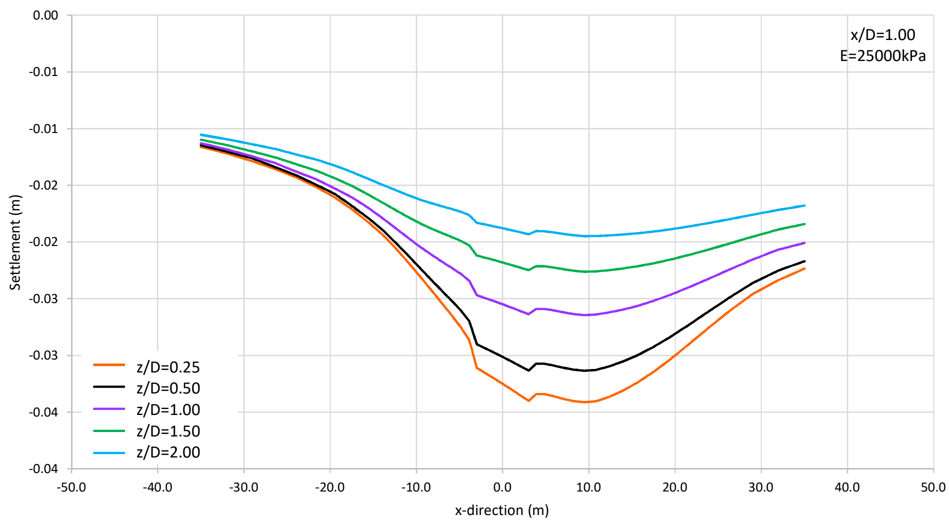


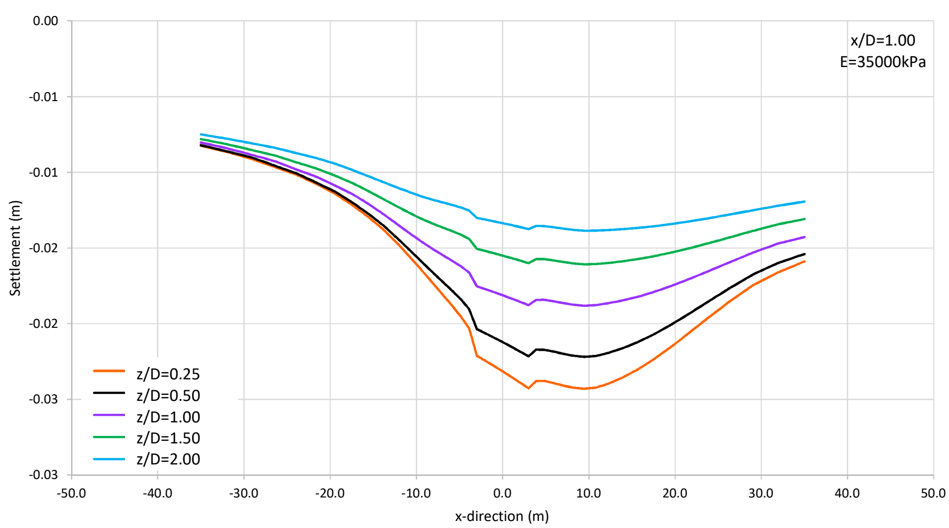
x/D=0.50
E=25000kPa
Settlement (m)
x-direction (m)
z/D=0.25
z/D=0.50
z/D=1.00
z/D=1.50
z/D=2.00

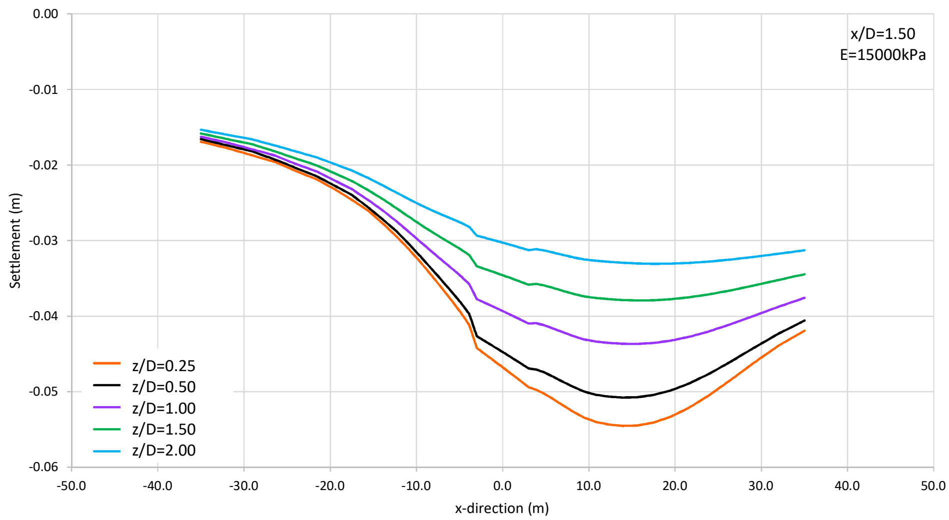


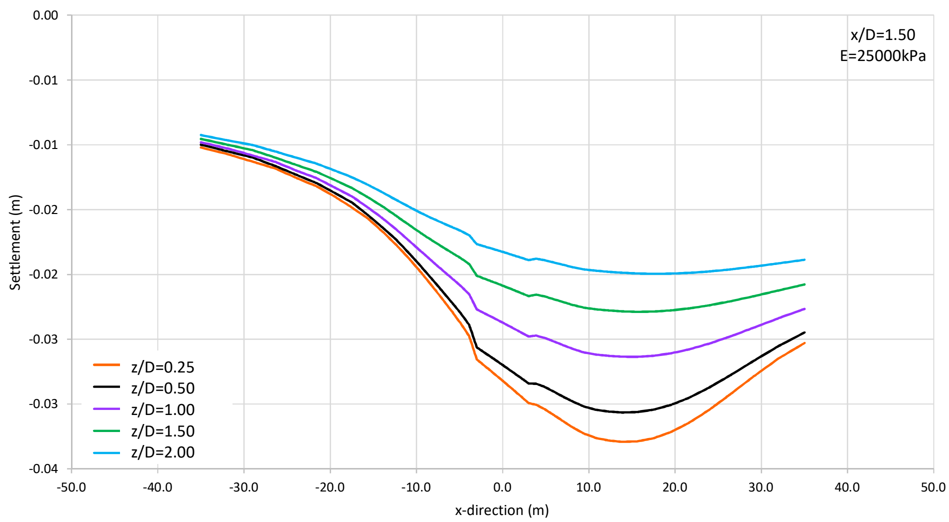
x/D=0.50
E=35000kPa
Settlement (m)
x-direction (m)
z/D=0.25
z/D=0.50
z/D=1.00
z/D=1.50
z/D=2.00

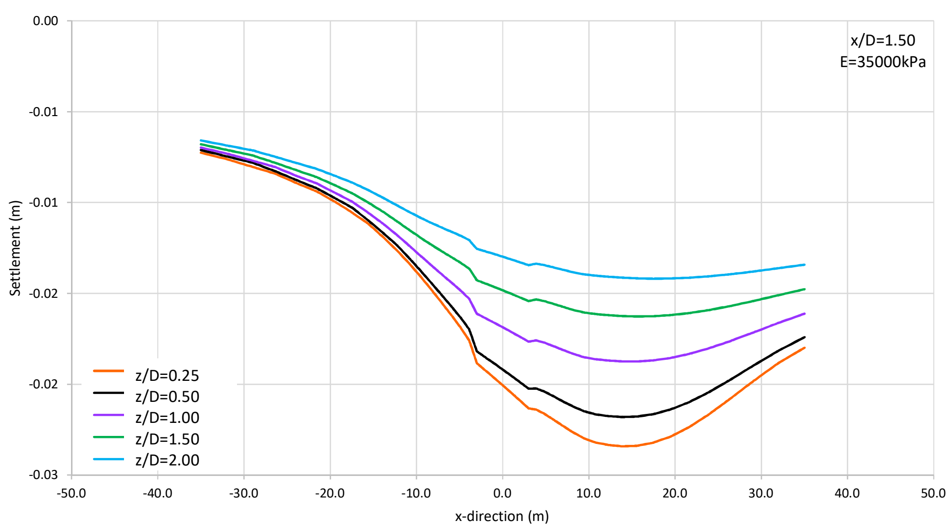

x/D=1.00
E=15000kPa
Settlement (m)
x-direction (m)
z/D=0.25
z/D=0.50
z/D=1.00
z/D=1.50
z/D=2.00

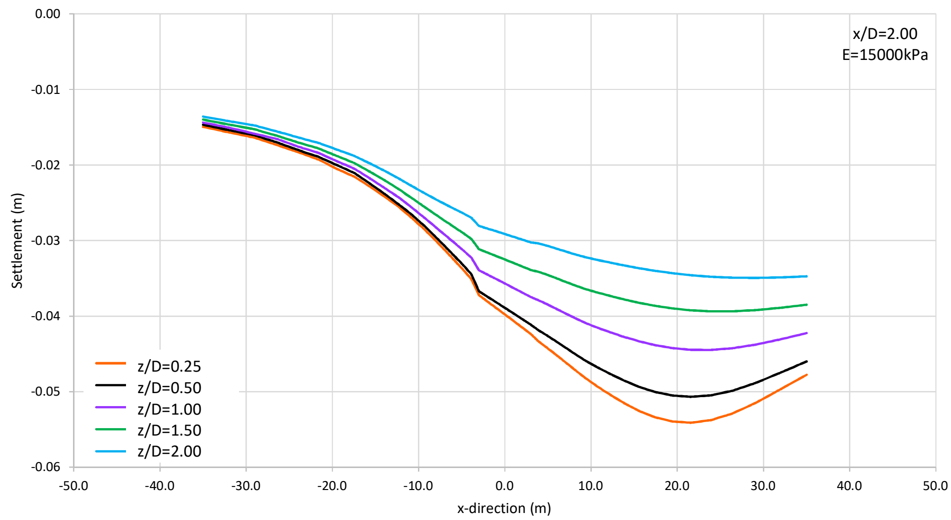

x/D=1.00
E=25000kPa
Settlement (m)
x-direction (m)
z/D=0.25
z/D=0.50
z/D=1.00
z/D=1.50
z/D=2.00


x/D=1.00
E=35000kPa
Settlement (m)
x-direction (m)
z/D=0.25
z/D=0.50
z/D=1.00
z/D=1.50
z/D=2.00

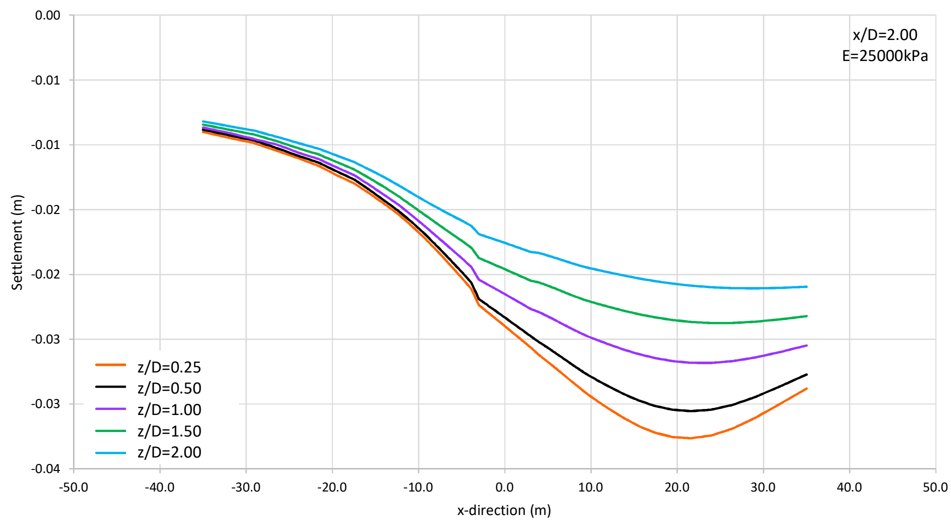

x/D=1.50
E=15000kPa
Settlement (m)
x-direction (m)
z/D=0.25
z/D=0.50
z/D=1.00
z/D=1.50
z/D=2.00


x/D=1.50
E=25000kPa
Settlement (m)
x-direction (m)
z/D=0.25
z/D=0.50
z/D=1.00
z/D=1.50
z/D=2.00

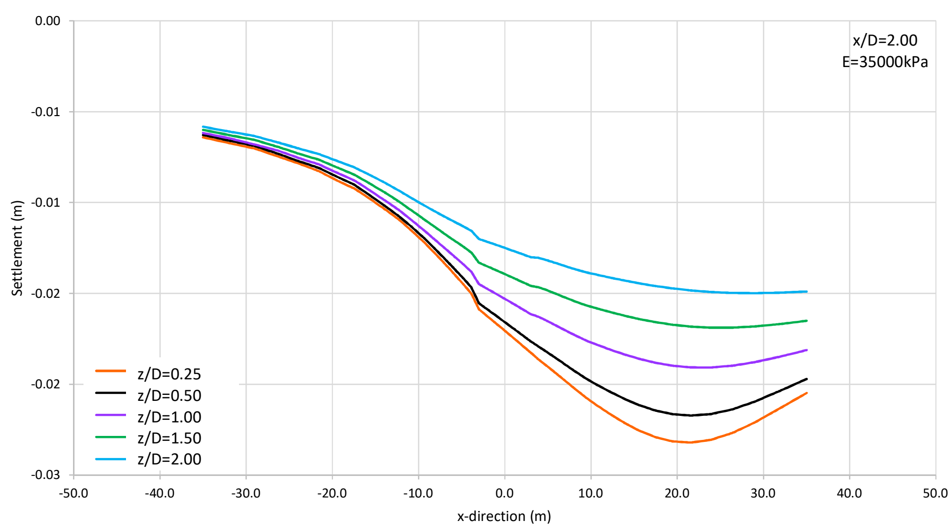

x/D=1.50
E=35000kPa
Settlement (m)
x-direction (m)
z/D=0.25
z/D=0.50
z/D=1.00
z/D=1.50
z/D=2.00



x/D=2.00
E=15000kPa
Settlement (m)
x-direction (m)
z/D=0.25
z/D=0.50
z/D=1.00
z/D=1.50
z/D=2.00
0.00
-0.01
-0.02
-0.03
-0.04
-0.05
-0.06
-50.0
-40.0
-30.0
-20.0
-10.0
0.0
10.0
20.0
30.0
40.0
50.0



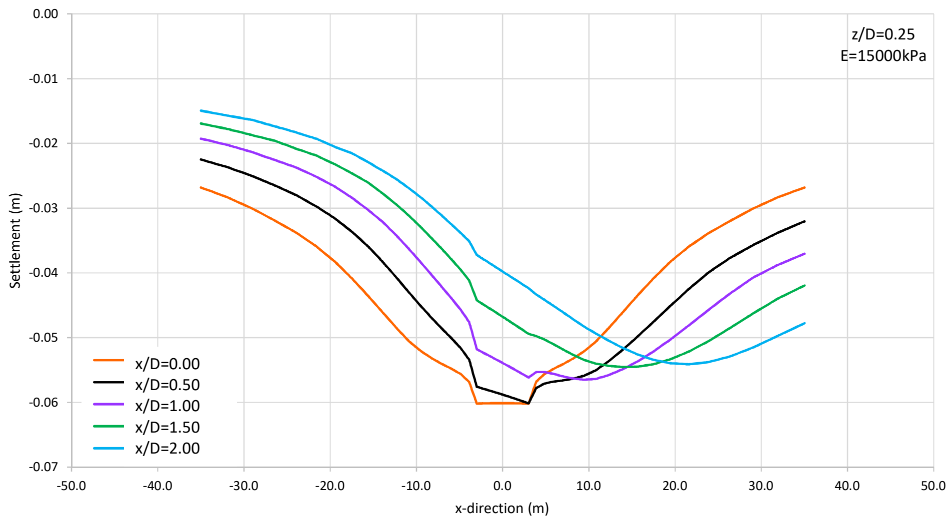
x/D=2.00
E=25000kPa
Settlement (m)
x-direction (m)
z/D=0.25
z/D=0.50
z/D=1.00
z/D=1.50
z/D=2.00
0.00
-0.01
-0.01
-0.02
-0.02
-0.03
-0.03
-0.04
-50.0
-40.0
-30.0
-20.0
-10.0
0.0
10.0
20.0
30.0
40.0
50.0

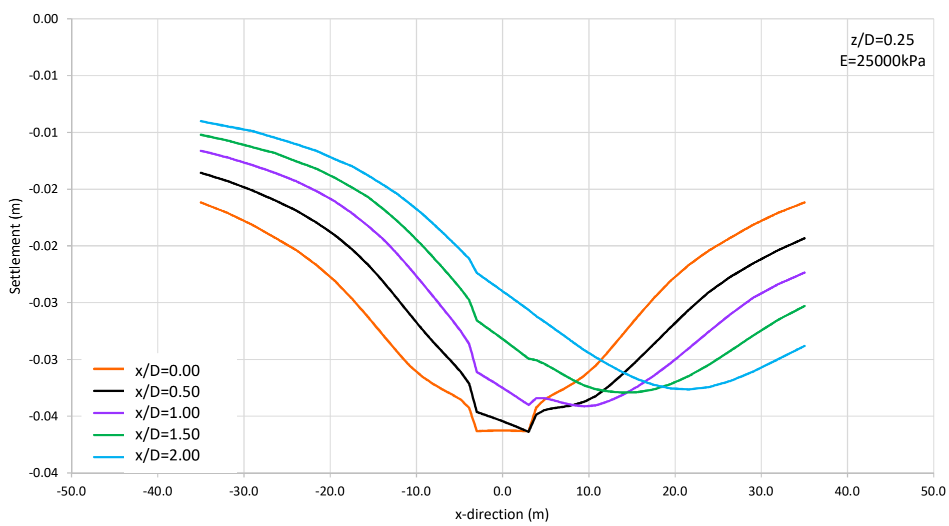


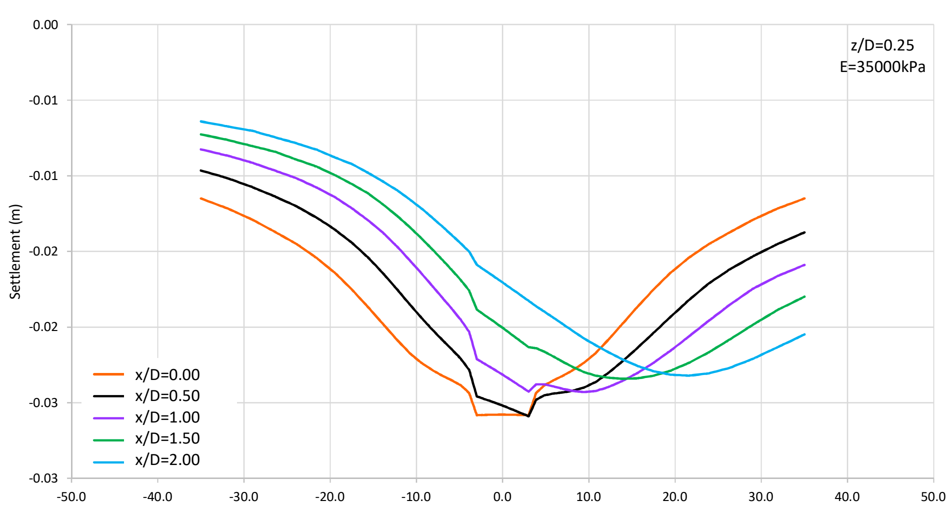
x/D=2.00
E=35000kPa
Settlement (m)
x-direction (m)
z/D=0.25
z/D=0.50
z/D=1.00
z/D=1.50
z/D=2.00
0.00
-0.01
-0.01
-0.02
-0.02
-0.03
-50.0
-40.0
-30.0
-20.0
-10.0
0.0
10.0
20.0
30.0
40.0
50.0

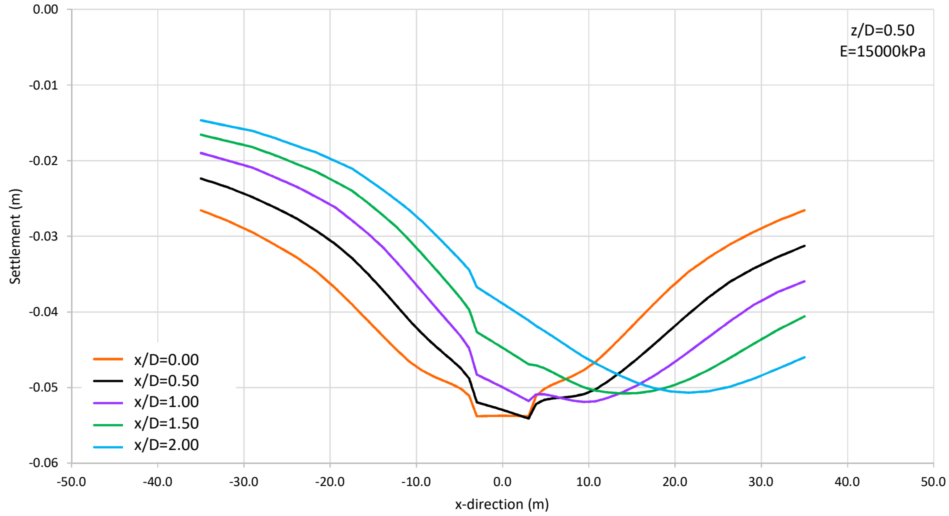
APPENDIX-1.2

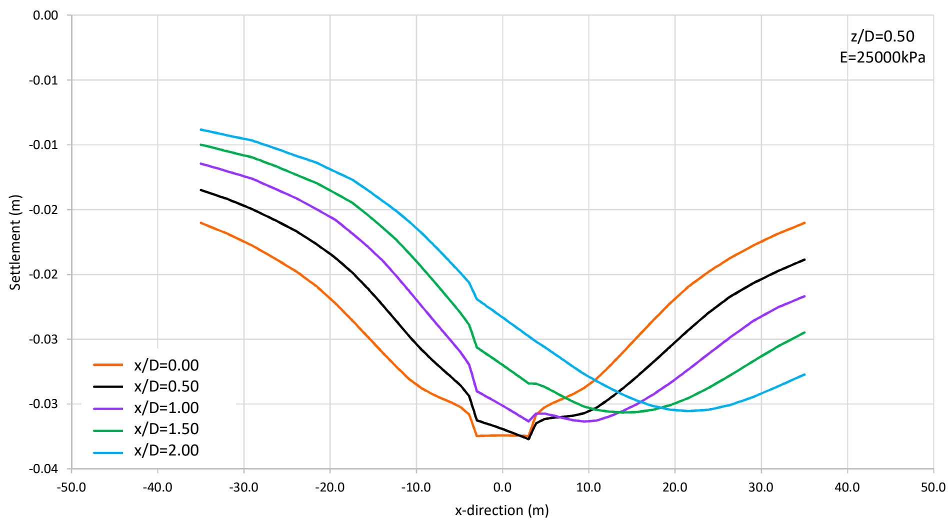
Surface Settlement Change with respect to x/D

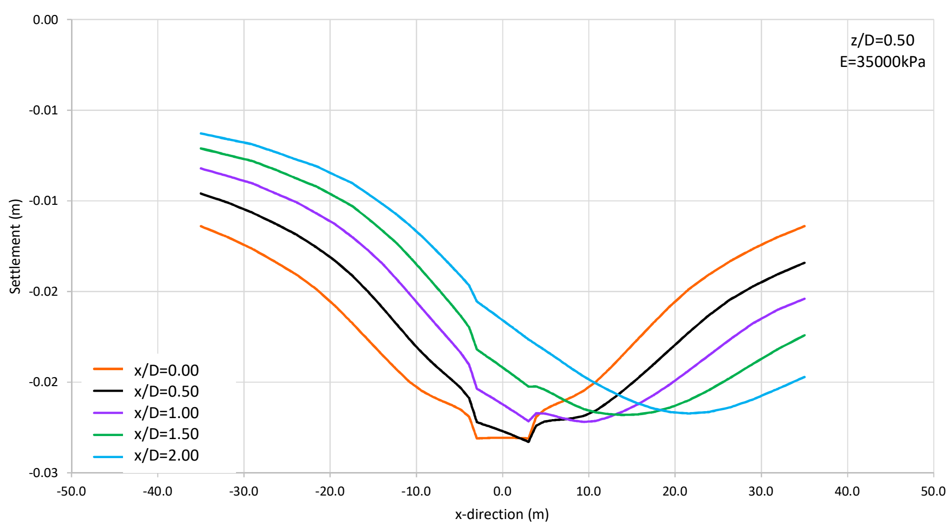

z/D=0.25
E=15000kPa
Settlement (m)
x-direction (m)
x/D=0.00
x/D=0.50
x/D=1.00
x/D=1.50
x/D=2.00
0.00
-0.01
-0.02
-0.03
-0.04
-0.05
-0.06
-0.07
-50.0
-40.0
-30.0
-20.0
-10.0
0.0
10.0
20.0
30.0
40.0
50.0

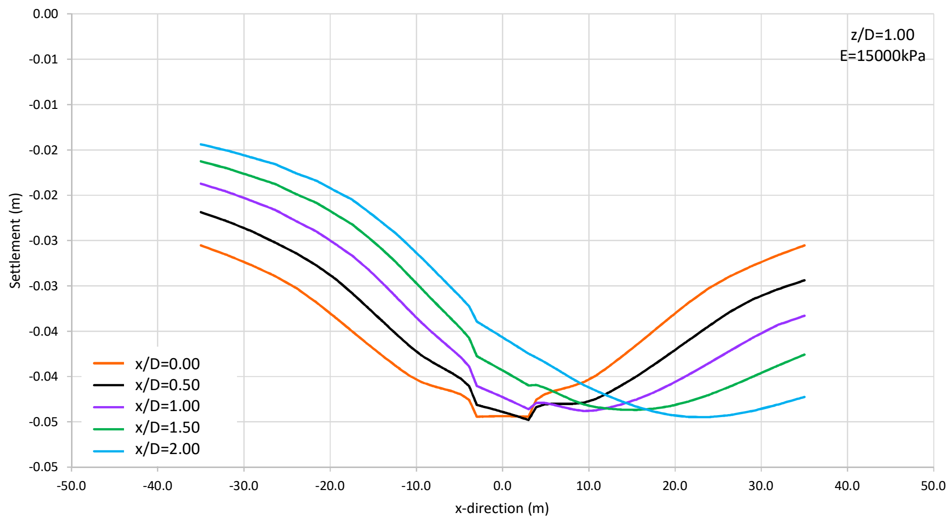

z/D=0.25
E=25000kPa
Settlement (m)
x-direction (m)
x/D=0.00
x/D=0.50
x/D=1.00
x/D=1.50
x/D=2.00
0.00
-0.01
-0.01
-0.02
-0.02
-0.03
-0.03
-0.04
-0.04
-50.0
-40.0
-30.0
-20.0
-10.0
0.0
10.0
20.0
30.0
40.0
50.0

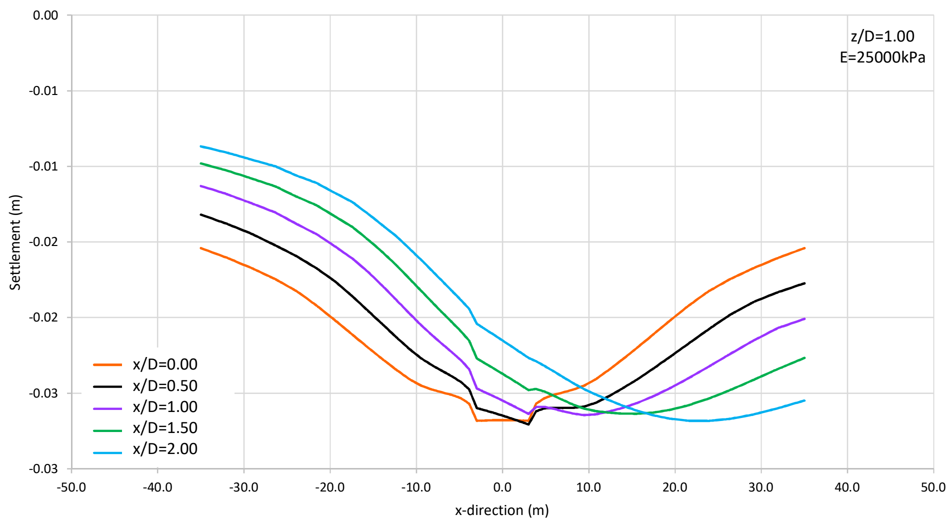

z/D=0.25
E=35000kPa
Settlement (m)
x-direction (m)
x/D=0.00
x/D=0.50
x/D=1.00
x/D=1.50
x/D=2.00
0.00
-0.01
-0.01
-0.02
-0.02
-0.03
-0.03
-50.0
-40.0
-30.0
-20.0
-10.0
0.0
10.0
20.0
30.0
40.0
50.0

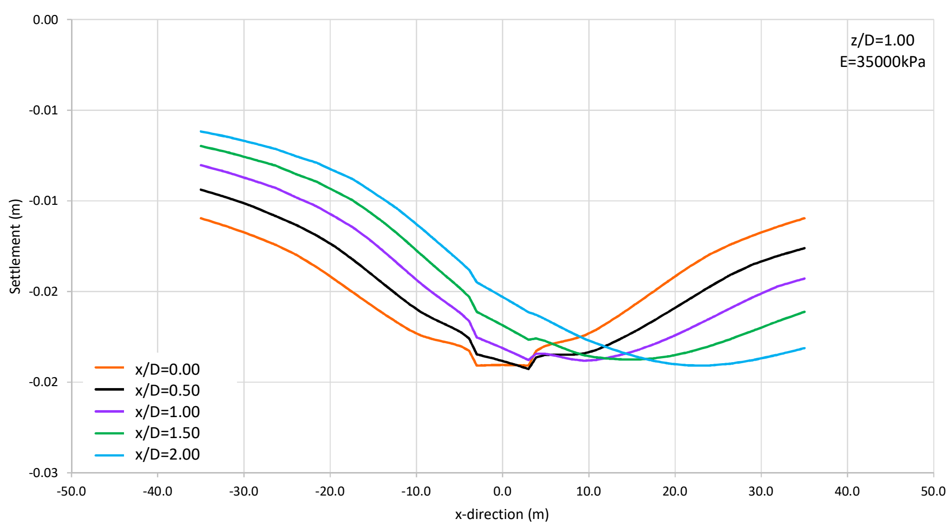

z/D=0.50
E=15000kPa
Settlement (m)
x-direction (m)
x/D=0.00
x/D=0.50
x/D=1.00
x/D=1.50
x/D=2.00

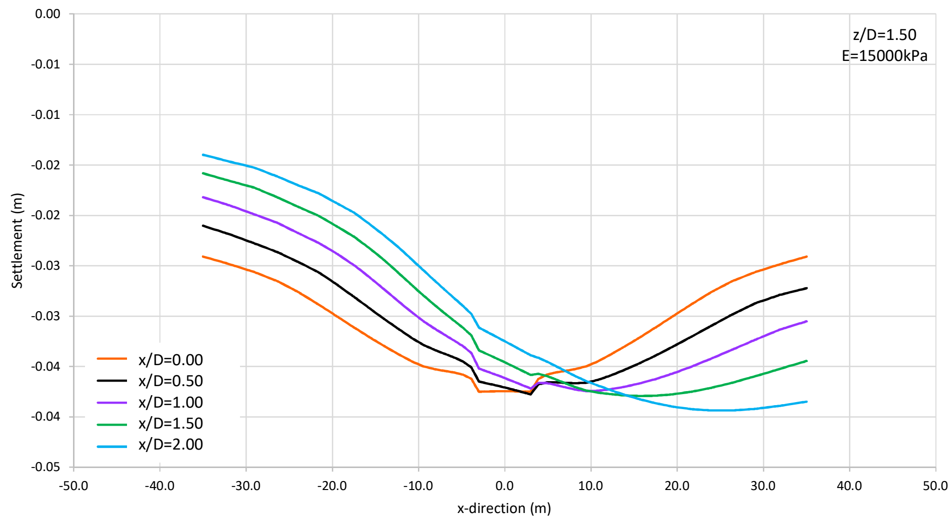

z/D=0.50
E=25000kPa
Settlement (m)
x-direction (m)
x/D=0.00
x/D=0.50
x/D=1.00
x/D=1.50
x/D=2.00


z/D=0.50
E=35000kPa
Settlement (m)
x-direction (m)
x/D=0.00
x/D=0.50
x/D=1.00
x/D=1.50
x/D=2.00

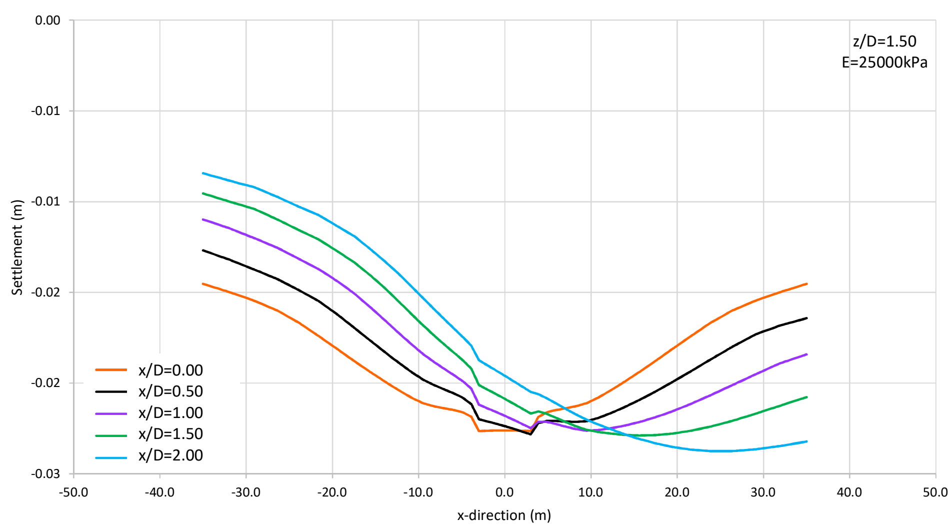

z/D=1.00
E=15000kPa
Settlement (m)
x-direction (m)
x/D=0.00
x/D=0.50
x/D=1.00
x/D=1.50
x/D=2.00
0.00
-0.01
-0.01
-0.02
-0.02
-0.03
-0.03
-0.04
-0.04
-0.05
-0.05
-50.0
-40.0
-30.0
-20.0
-10.0
0.0
10.0
20.0
30.0
40.0
50.0


z/D=1.00
E=25000kPa
Settlement (m)
x-direction (m)
x/D=0.00
x/D=0.50
x/D=1.00
x/D=1.50
x/D=2.00
0.00
-0.01
-0.01
-0.02
-0.02
-0.03
-0.03
-50.0
-40.0
-30.0
-20.0
-10.0
0.0
10.0
20.0
30.0
40.0
50.0

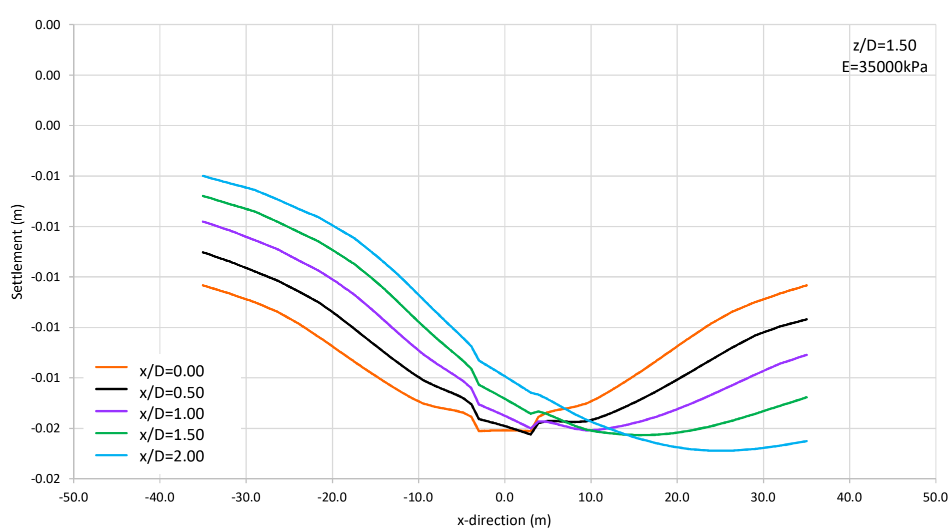

z/D=1.00
E=35000kPa
Settlement (m)
x-direction (m)
x/D=0.00
x/D=0.50
x/D=1.00
x/D=1.50
x/D=2.00
0.00
-0.01
-0.01
-0.02
-0.02
-0.03
-50.0
-40.0
-30.0
-20.0
-10.0
0.0
10.0
20.0
30.0
40.0
50.0



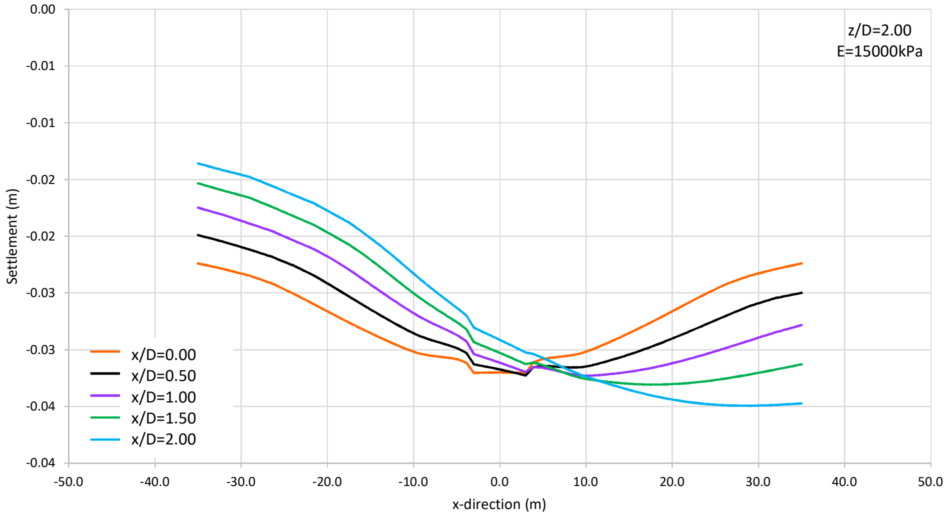
z/D=1.50
E=15000kPa
Settlement (m)
x-direction (m)
x/D=0.00
x/D=0.50
x/D=1.00
x/D=1.50
x/D=2.00
0.00
-0.01
-0.01
-0.02
-0.02
-0.03
-0.03
-0.04
-0.04
-0.05
-50.0
-40.0
-30.0
-20.0
-10.0
0.0
10.0
20.0
30.0
40.0
50.0

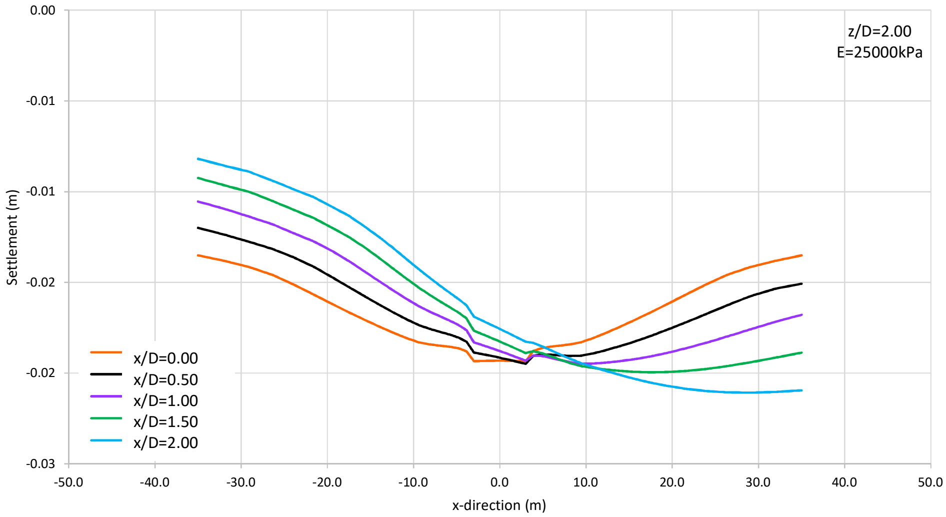


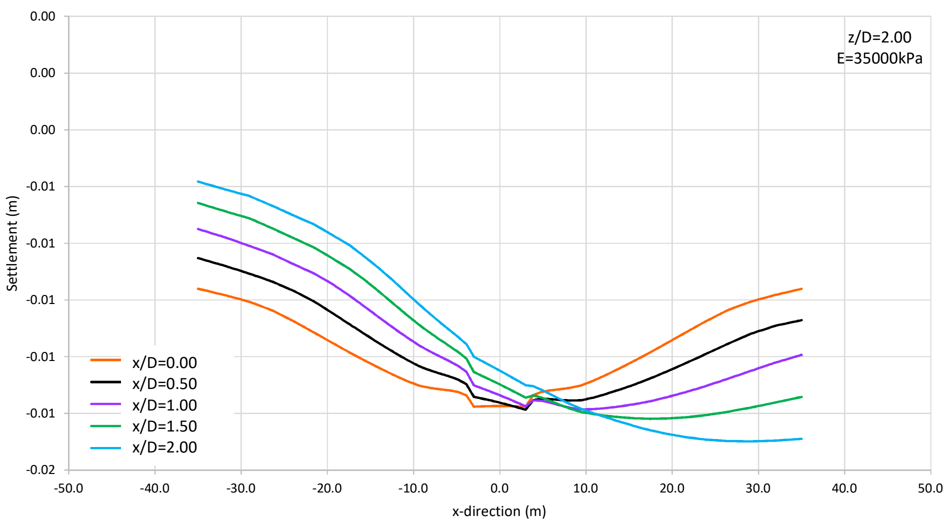
z/D=1.50
E=25000kPa
Settlement (m)
x-direction (m)
x/D=0.00
x/D=0.50
x/D=1.00
x/D=1.50
x/D=2.00
0.00
-0.01
-0.01
-0.02
-0.02
-0.03
-50.0
-40.0
-30.0
-20.0
-10.0
0.0
10.0
20.0
30.0
40.0
50.0



z/D=1.50
E=35000kPa
Settlement (m)
x-direction (m)
x/D=0.00
x/D=0.50
x/D=1.00
x/D=1.50
x/D=2.00
0.00
0.00
0.00
-0.01
-0.01
-0.01
-0.01
-0.01
-0.02
-0.02
-50.0
-40.0
-30.0
-20.0
-10.0
0.0
10.0
20.0
30.0
40.0
50.0

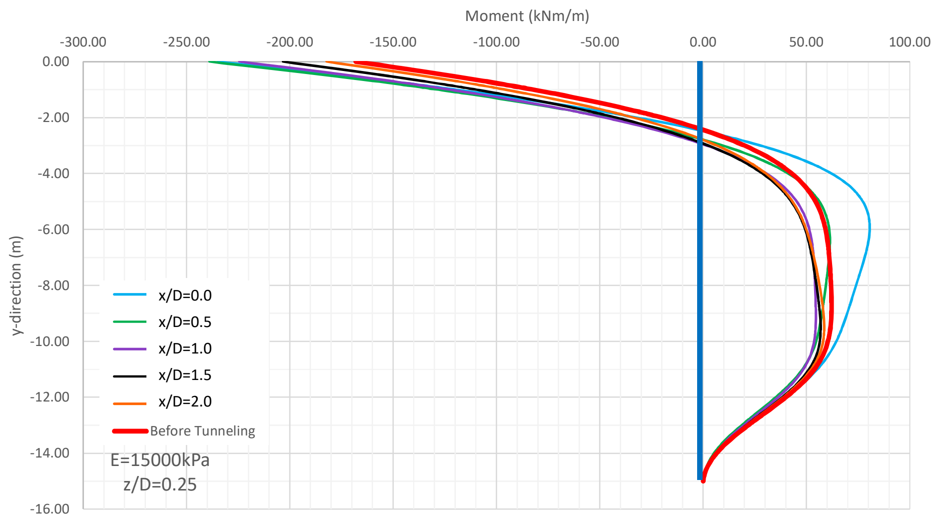

z/D=2.00
E=15000kPa
Settlement (m)
x-direction (m)
x/D=0.00
x/D=0.50
x/D=1.00
x/D=1.50
x/D=2.00

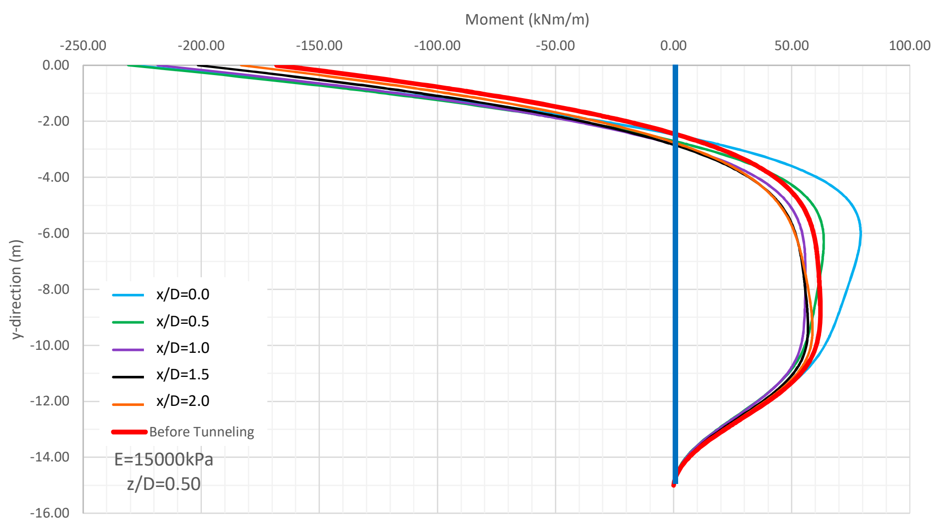

z/D=2.00
E=25000kPa
Settlement (m)
x-direction (m)
x/D=0.00
x/D=0.50
x/D=1.00
x/D=1.50
x/D=2.00

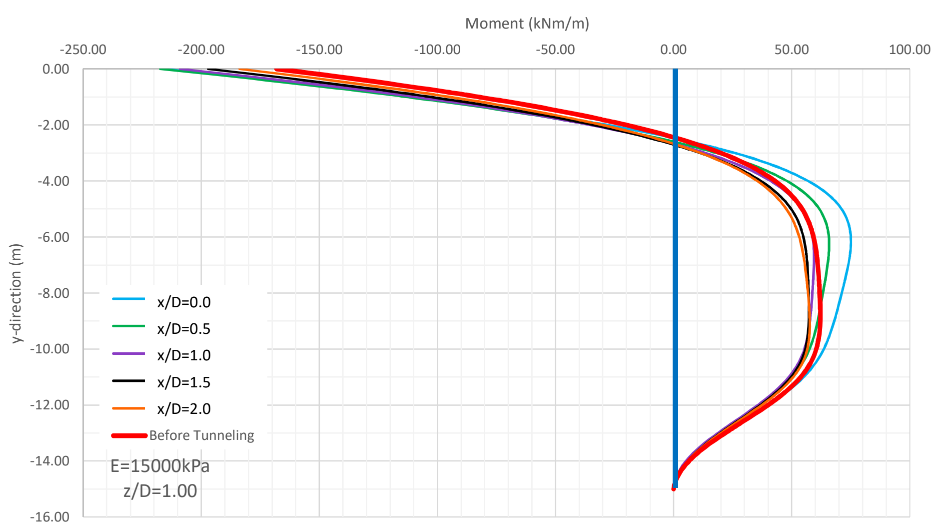

z/D=2.00
E=35000kPa
Settlement (m)
x-direction (m)
x/D=0.00
x/D=0.50
x/D=1.00
x/D=1.50
x/D=2.00

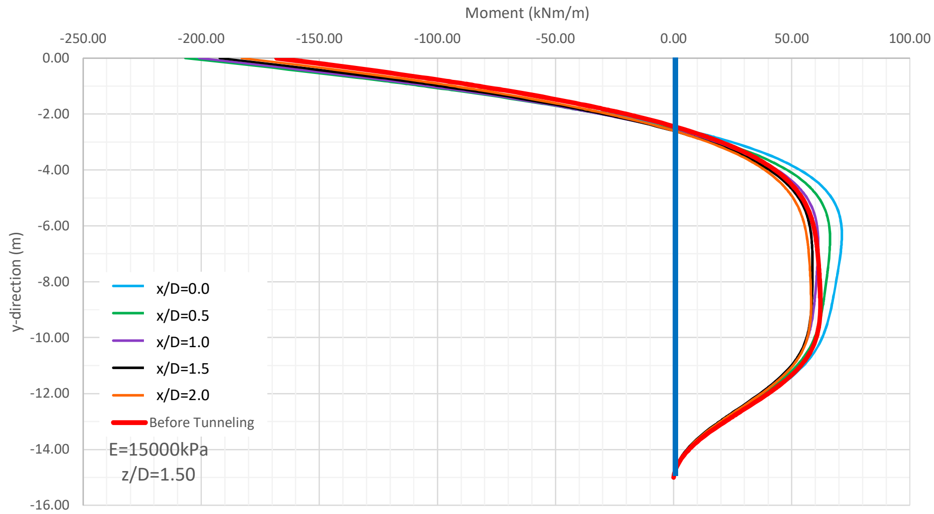
APPENDIX-1.3

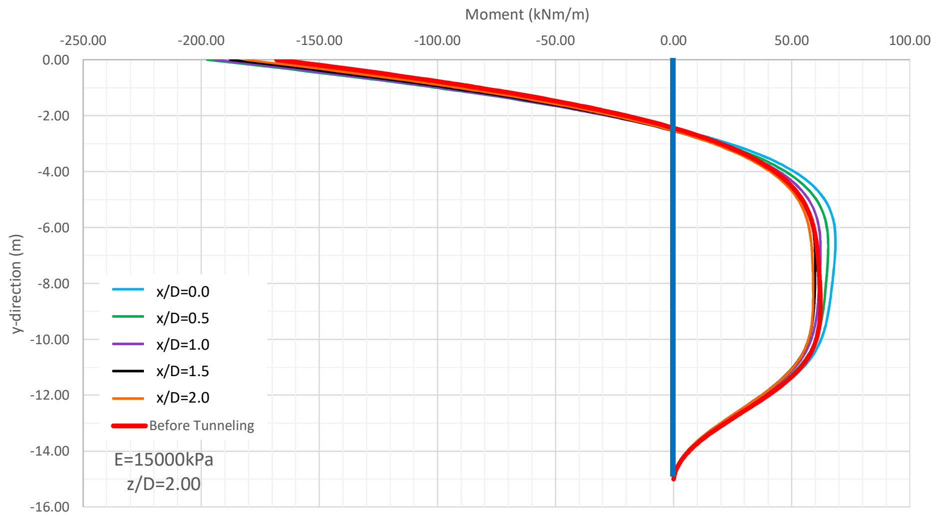
Moment Graphs

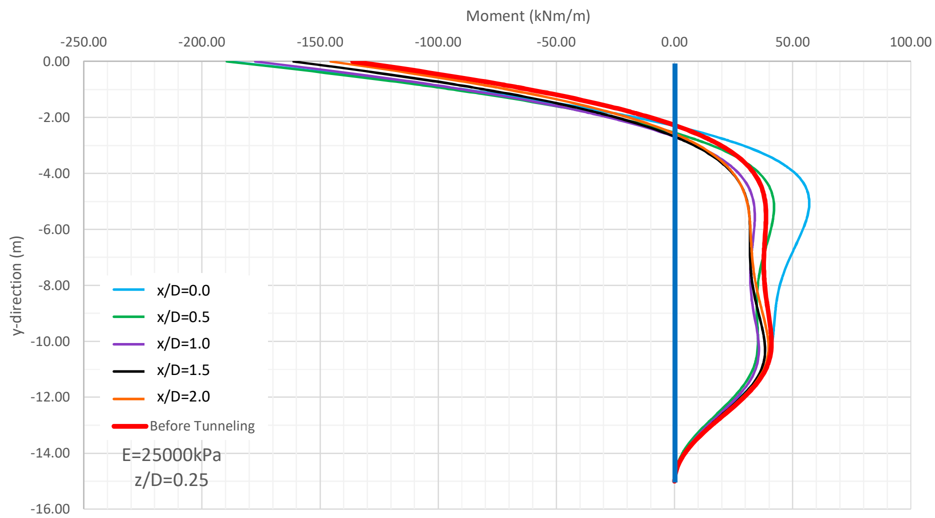

Moment (kNm/m)
-300.00
-250.00
-200.00
-150.00
-100.00
-50.00
0.00
50.00
100.00
0.00
-2.00
-4.00
-6.00
-8.00
-10.00
-12.00
-14.00
-16.00
y-direction (m)
x/D=0.0
x/D=0.5
x/D=1.0
x/D=1.5
x/D=2.0
Before Tunneling
E=15000kPa
z/D=0.25

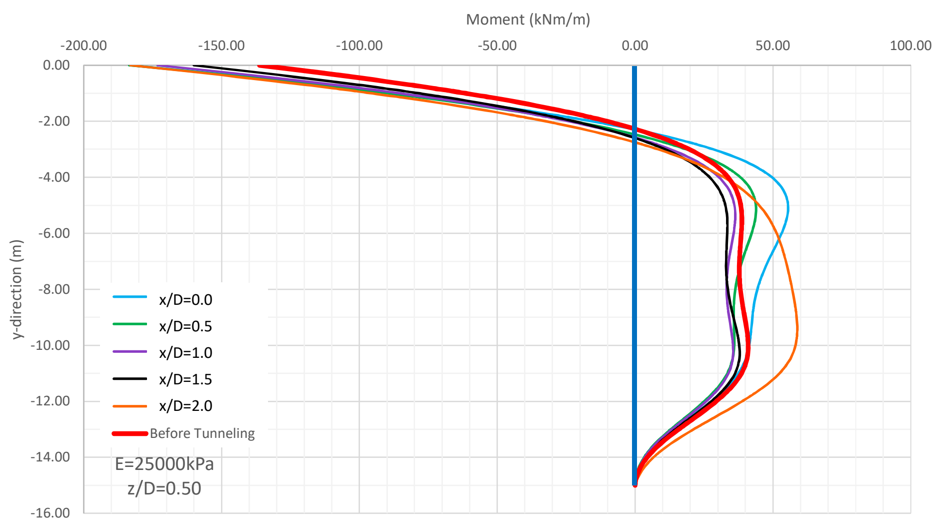

Moment (kNm/m)
-250.00
-200.00
-150.00
-100.00
-50.00
0.00
50.00
100.00
0.00
-2.00
-4.00
-6.00
-8.00
-10.00
-12.00
-14.00
-16.00
y-direction (m)
x/D=0.0
x/D=0.5
x/D=1.0
x/D=1.5
x/D=2.0
Before Tunneling
E=15000kPa
z/D=0.50

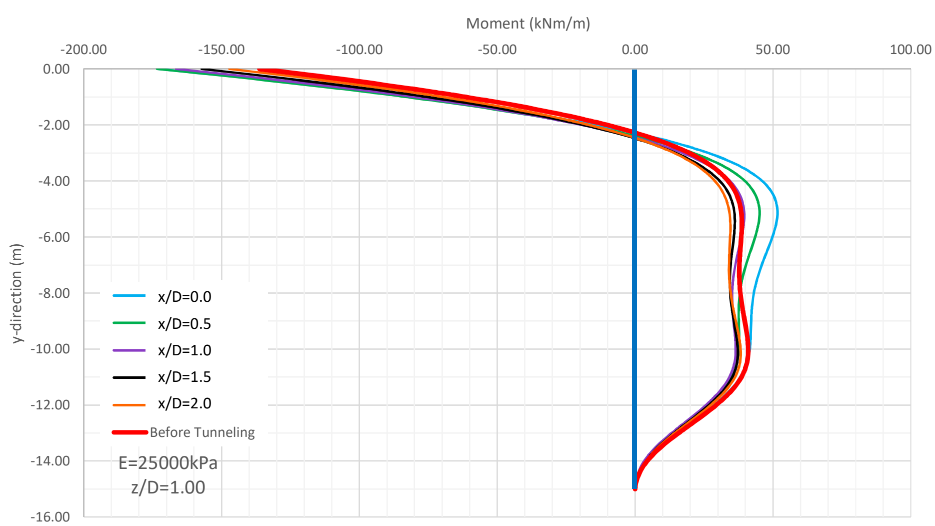

Moment (kNm/m)
-250.00
-200.00
-150.00
-100.00
-50.00
0.00
50.00
100.00
0.00
-2.00
-4.00
-6.00
-8.00
-10.00
-12.00
-14.00
-16.00
y-direction (m)
x/D=0.0
x/D=0.5
x/D=1.0
x/D=1.5
x/D=2.0
Before Tunneling
E=15000kPa
z/D=1.00

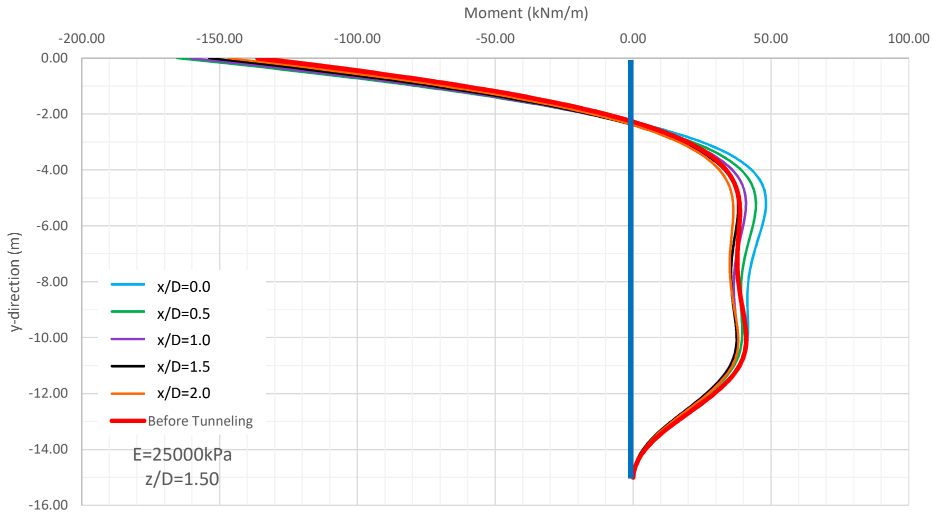

Moment (kNm/m)
-250.00
-200.00
-150.00
-100.00
-50.00
0.00
50.00
100.00
y-direction (m)
0.00
-2.00
-4.00
-6.00
-8.00
-10.00
-12.00
-14.00
-16.00
x/D=0.0
x/D=0.5
x/D=1.0
x/D=1.5
x/D=2.0
Before Tunneling
E=15000kPa
z/D=1.50

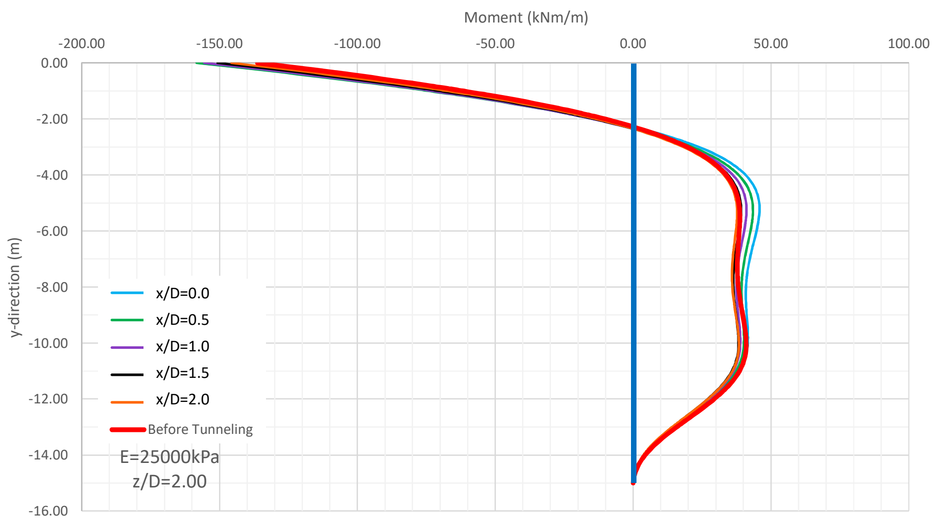

Moment (kNm/m)
-250.00
-200.00
-150.00
-100.00
-50.00
0.00
50.00
100.00
y-direction (m)
0.00
-2.00
-4.00
-6.00
-8.00
-10.00
-12.00
-14.00
-16.00
x/D=0.0
x/D=0.5
x/D=1.0
x/D=1.5
x/D=2.0
Before Tunneling
E=15000kPa
z/D=2.00

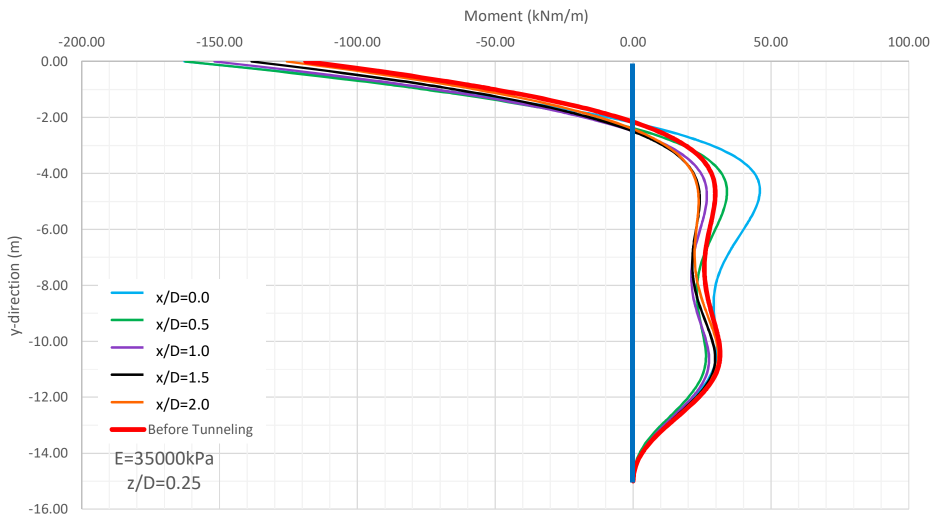

Moment (kNm/m)
-250.00
-200.00
-150.00
-100.00
-50.00
0.00
50.00
100.00
0.00
-2.00
-4.00
-6.00
-8.00
-10.00
-12.00
-14.00
-16.00
y-direction (m)
x/D=0.0
x/D=0.5
x/D=1.0
x/D=1.5
x/D=2.0
Before Tunneling
E=25000kPa
z/D=0.25

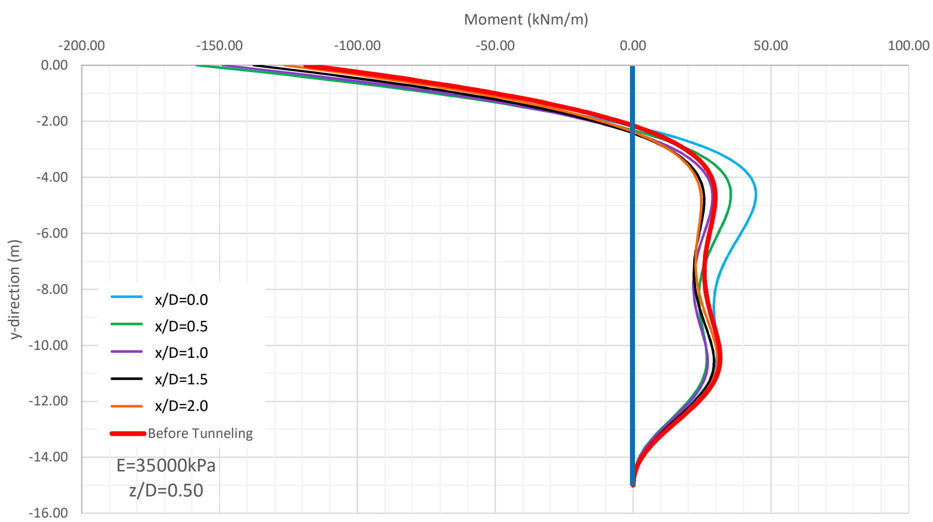

Moment (kNm/m)
-200.00
-150.00
-100.00
-50.00
0.00
50.00
100.00
0.00
-2.00
-4.00
-6.00
-8.00
-10.00
-12.00
-14.00
-16.00
y-direction (m)
x/D=0.0
x/D=0.5
x/D=1.0
x/D=1.5
x/D=2.0
Before Tunneling
E=25000kPa
z/D=0.50

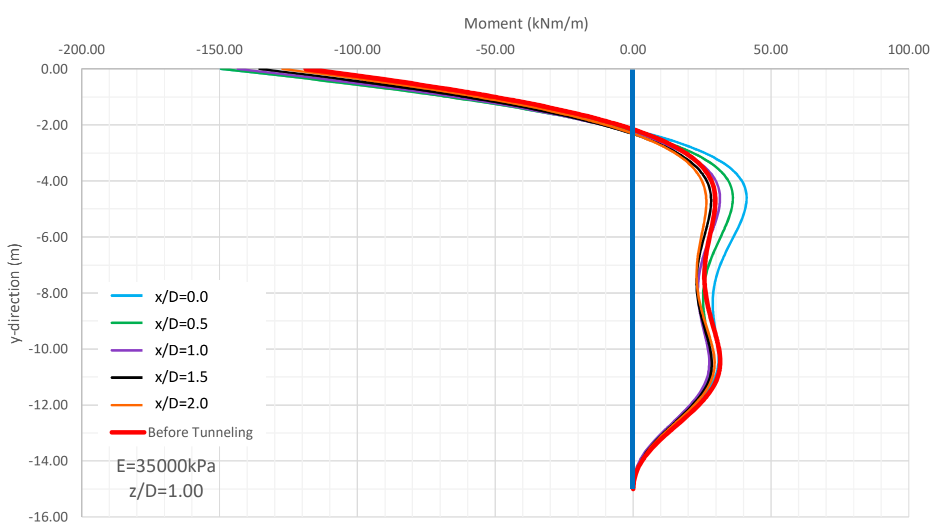

Moment (kNm/m)
-200.00
-150.00
-100.00
-50.00
0.00
50.00
100.00
0.00
-2.00
-4.00
-6.00
-8.00
-10.00
-12.00
-14.00
-16.00
y-direction (m)
x/D=0.0
x/D=0.5
x/D=1.0
x/D=1.5
x/D=2.0
Before Tunneling
E=25000kPa
z/D=1.00

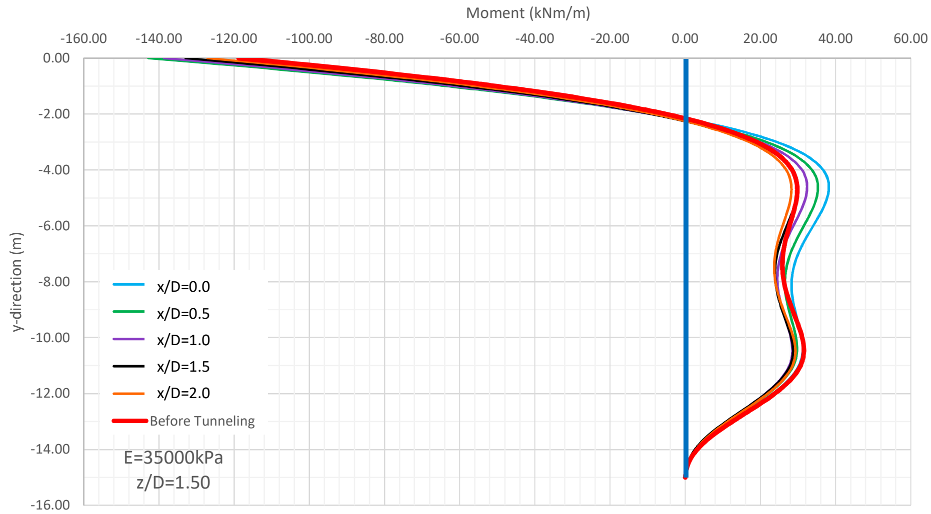

Moment (kNm/m)
-200.00
-150.00
-100.00
-50.00
0.00
50.00
100.00
0.00
-2.00
-4.00
-6.00
-8.00
-10.00
-12.00
-14.00
-16.00
y-direction (m)
x/D=0.0
x/D=0.5
x/D=1.0
x/D=1.5
x/D=2.0
Before Tunneling
E=25000kPa
z/D=1.50

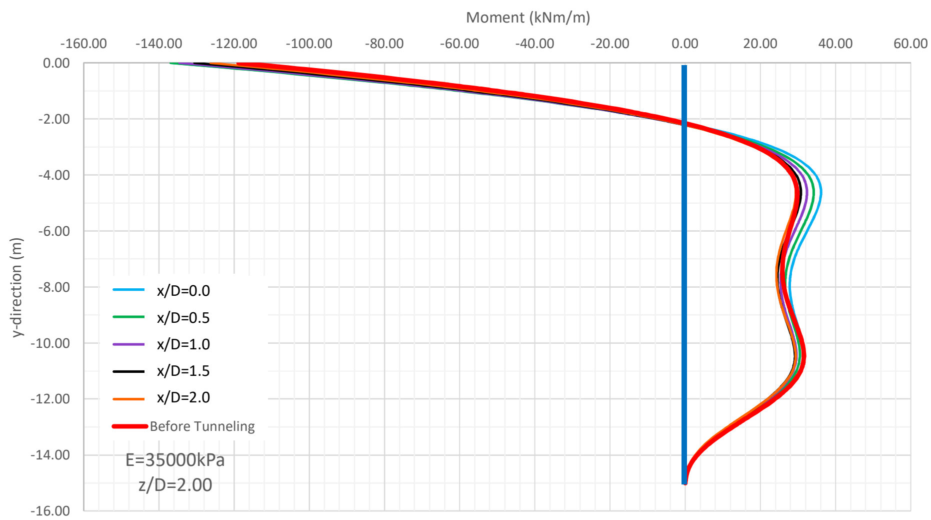

Moment (kNm/m)
-200.00
-150.00
-100.00
-50.00
0.00
50.00
100.00
0.00
-2.00
-4.00
-6.00
-8.00
-10.00
-12.00
-14.00
-16.00
y-direction (m)
x/D=0.0
x/D=0.5
x/D=1.0
x/D=1.5
x/D=2.0
Before Tunneling
E=25000kPa
z/D=2.00


Moment (kNm/m)
y-direction (m)
x/D=0.0
x/D=0.5
x/D=1.0
x/D=1.5
x/D=2.0
Before Tunneling
E=35000kPa
z/D=0.25


Moment (kNm/m)
y-direction (m)
x/D=0.0
x/D=0.5
x/D=1.0
x/D=1.5
x/D=2.0
Before Tunneling
E=35000kPa
z/D=0.50


Moment (kNm/m)
y-direction (m)
x/D=0.0
x/D=0.5
x/D=1.0
x/D=1.5
x/D=2.0
Before Tunneling
E=35000kPa
z/D=1.00


Moment (kNm/m)
-160.00
-140.00
-120.00
-100.00
-80.00
-60.00
-40.00
-20.00
0.00
20.00
40.00
60.00
0.00
-2.00
-4.00
-6.00
-8.00
-10.00
-12.00
-14.00
-16.00
y-direction (m)
x/D=0.0
x/D=0.5
x/D=1.0
x/D=1.5
x/D=2.0
Before Tunneling
E=35000kPa
z/D=1.50


Moment (kNm/m)
-160.00
-140.00
-120.00
-100.00
-80.00
-60.00
-40.00
-20.00
0.00
20.00
40.00
60.00
0.00
-2.00
-4.00
-6.00
-8.00
-10.00
-12.00
-14.00
-16.00
y-direction (m)
x/D=0.0
x/D=0.5
x/D=1.0
x/D=1.5
x/D=2.0
Before Tunneling
E=35000kPa
z/D=2.00

HACETTEPE UNIVERSITY
GRADUATE SCHOOL OF SCIENCE AND ENGINEERING
THESIS/DISSERTATION ORIGINALITY REPORT

HACETTEPE UNIVERSITY
GRADUATE SCHOOL OF SCIENCE AND ENGINEERING
TO THE DEPARTMENT OF CIVIL ENGINEERING

Date: …/…./………

Thesis Title / Topic: **EFFECTS OF TUNNELING ON PILED FOUNDATIONS IN CLEAN SANDS USING FINITE ELEMENT METHOD AND SENSITIVITY ANALYSES**

According to the originality report obtained by ~~myself~~/**my thesis advisor** by using the *Turnitin* plagiarism detection software and by applying the filtering options stated below on **28/01/2021** for the total of **84** pages including the a) Title Page, b) Introduction, c) Main Chapters, d) Conclusion sections of my thesis entitled as above, the similarity index of my thesis is **5 %**.

Filtering options applied:

1. Bibliography/Works Cited excluded
2. Quotes excluded / included
3. Match size up to 5 words excluded

I declare that I have carefully read Hacettepe University Graduate School of Sciene and Engineering Guidelines for Obtaining and Using Thesis Originality Reports; that according to the maximum similarity index values specified in the Guidelines, my thesis does not include any form of plagiarism; that in any future detection of possible infringement of the regulations I accept all legal responsibility; and that all the information I have provided is correct to the best of my knowledge.

I respectfully submit this for approval.

Date and Signature

Name Surname:	Cansu GÜNDAY URAS
Student No:	18135709
Department:	Civil Engineering
Program:	
Status:	☒ Masters ☐ Ph.D. ☐ Integrated Ph.D.

ADVISOR APPROVAL

APPROVED.

Prof. Dr. Berna UNUTMAZ

(Title, Name Surname, Signature)