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M.SC. THESIS

Tunneling Effect on Foundation Settlement

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إقــرار

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Tunneling Effect on Foundation Settlement

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بناءً على موافقة شئون البحث العلمي والدراسات العليا بالجامعة الإسلامية بغزة على تشكيل لجنة الحكم على أطروحة الباحث/ **منيب عمران محمد جاد الله** لنيل درجة الماجستير في **كلية *الهندسة*** **قسم الهندسة المدنية- تصميم وتأهيل المنشآت** وموضوعها:

تأثير الحفريات على هبوط المباني

Tunneling Effect on Foundation Settlement

وبعد المناقشة العلنية التي تمت اليوم السبت 26 ربيع الأول 1436هـ، الموافق 2014/01/17م الساعة الحادية عشرة صباحاً بمبنى القدس، اجتمعت لجنة الحكم على الأطروحة والمكونة من:

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وبعد المداولة أوصت اللجنة بمنح الباحث درجة الماجستير في **كلية *الهندسة*/ قسم الهندسة المدنية- تصميم وتأهيل المنشآت.**

واللجنة إذ تمنحه هذه الدرجة فإنها توصيه بتقوى الله ولزوم طاعته وأن يسخر علمه في خدمة دينه ووطنه.

والله ولي التوفيق،،،

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ABSTRACT

One of the problems of tunneling in urban spaces is ground settlements to surface structures. Ground movement prediction is closely related to structural performance and the interaction between the ground and the tunnel. This complexity means that normally conservative assumptions may not be appropriate and in some instances could even cover the most significant issues with particular tunnel crossings.

Current design approaches are conservative and lead to predict of the settlement of foundation building specially when tunnel crossing under the foundation of structure and guide engineers to design tunnels to safe these building from damages or danger cracks

Recently, a new approach, based on applying numerical methods using the PLAXIS finite element software code to provide direct equations to calculate settlement due to tunneling in urban area. Different loads and different type of soils were investigated.

Results show that tunnel diameter is a major geometrical parameter which increase the effect of settlement. And loads on foundation must be considered in simulation to assure reliable results were with more loads the settlement will increase. Also soil type is another important factor which has significant effects on the tunneling–building interaction behavior. And increasing of tunnel depth, surface distance of foundation from the upper face of tunnels decrease the effect of settlement.

Six equations was developed for predicting the maximum settlements of foundation to use in preliminary design stage.

Results compare very with measured available data (case study: - Shiraz metro line1).The results for medium clay show maximum settlement of 18.5 mm while the measured settlement by Shiraz meter case was 19 mm. This show a good agreement between calculation and measured values. And result shows that for sand settlement of foundation range from 0.60mm to 5.12 mm in Greenfield, but in clay settlement of foundation range from 11.4mm to 42.3mm.

ملخص باللغة العربية

حفر الأنفاق في المناطق السكنية يؤدي حتما إلي هبوط المباني بالتالي يجب حساب مقدار هذا الهبوط و دراسة أثره علي هذه المباني قبل الشروع في حفر الأنفاق أسفل هذه المباني حتى نحافظ علي المباني من الانهيار.

الدراسة الحالية تركز علي حساب قيمة الهبوط الناتج عن حفر الأنفاق أسفل المباني و بالتالي تجنب إلحاق الضرر بهذه المباني.

ركزت هذه الدراسة علي حساب قيمة الهبوط باستخدام برامج الحاسوب و التي تطورت بشكل كبير حيث ثم استخدام برنامج PLAXIS في هذه الدراسة لحساب قيمة الهبوط الناشئ عن حفر الأنفاق أسفل المباني لحالات مختلفة من التربة و كذلك لأقطار و أعماق متغيرة للنفق.

من خلال النتائج المتعلقة بحساب قيمة الهبوط تم استنتاج ست معادلات رئيسية لحساب قيمة الهبوط للمباني نتيجة حفر الأنفاق و ذلك بمعرفة قيمة الحمل من المبني و كذلك معرفة نوع التربة.

أظهرت النتائج من الدراسة أن قيمة الهبوط تزداد مع زيادة قطر النفق الذي يمر أسفل المبني و كذلك أظهرت النتائج أن قيمة الهبوط تقل كلما زاد عمق النفق من أسفل المبني إلي السطح العلوي للنفق.

كذلك أظهرت النتائج أن قيمة الهبوط تزداد في التربة الطينية عنها في التربة الرملية حيت تراوحت قيمة الهبوط في التربة الرملية من 0.60 ملم إلي 5.12 ملم و في التربة الطينية من 11.4 ملم إلي 42.3 ملم و ذلك في حالة عدم وجود أحمال من المبني و هي حالة ما يسمي (Greenfield).

تم التحقق من دقة المعادلات التي تم استنتاجها من خلال الدراسة و ذلك بعمل مقارنة مع نتائج مخبريه تم من خلالها حساب قيمة الهبوط في الموقع في مشروع مترو أنفاق في إيران حيث أظهرت هذه الدراسة التطابق بشكل كبير مع نتائج الموقع و هو ما يؤكد مدي دقة هذه المعادلات.

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SYMPOLS

B	Width of footing
Cu	Cohesion strength of soil
H	Depth of sand layer
E	Young's modulus of elasticity of soil
S	Vertical settlement
S_{max}	Maximum vertical settlement
y	Transverse distance from the tunnel axis
i	Represents the distance of the inflection point from the axis
z	Vertical level of the tunnel axis
k	Depending on the geotechnical characteristics of the ground
V_L	Volume loss
Vo	volume required for tunnel
P	concentrated load
$\varepsilon_x, \varepsilon_y, \varepsilon_z$	Normal strain components
$\sigma_x, \sigma_y, \sigma_z$	Normal stress components
$\gamma_{xy}, \gamma_{yz}, \gamma_{zx}$	Shear strain components
$\tau_{xy}, \tau_{yz}, \tau_{zx}$	Shear stress components
ϕ	Angle of internal friction of soil
γ	Unit weight of soil
γ_{av}	Average unit weight of soil
ν	Poisson's ratio of soil
ψ	Dilatancy angle of soil
K	Hydraulic Conductivity

CHAPTER 1

INTRODUCTION

1.1 Background

Due to the increase of traffic congestion in Gaza Strip, construction of underground transportations paces (e.g. underground Roads) is inevitable. Tunneling will be vital solution to crowded traffics in Gaza city since the city one of the crowded place on earth. Gaza strip as a whole is about 360 square kilometer and it's about 40 Km long. The current population density of Gaza strip is about 3500 people per square kilometer. Eventually a tunneling system will be necessary to deal with the congestions on traffic signals in Gaza city. One of the problems of tunneling in urban spaces is ground settlements to surface structures. Therefore, the prediction of tunnel effect on building deformation is very important for planning process. Current design approaches are conservative and lead to predict of the settlement of foundation building specially when tunnel crossing under the foundation of structure and guide engineers to design tunnels to safe these building from damages or danger cracks

This research project focus on the settlement of shallow foundation caused by tunneling. Settlement prediction of shallow foundation with different variables such as depth, diameter of tunnel and type of soil where investigated. There are three methods used to estimate tunneling caused ground movements: 1) empirical, 2) analytical and 3) numerical methods. Numerical analyses are the only method which model the complexities of soil-structure interactions settlement calculations of shallow foundations where performed applying numerical methods using the PLAXIS finite element software code. Therefore, a two-dimensional numerical modeling using finite element method will be considered.

1.2 Problem Statement:

The increase of tunneling in Gaza strip resulted in many structural and infrastructural problems to the existing structures. As urban space becomes more limited, where the population density in built up areas is very high per meter square where subsurface structures such as tunnels are becoming more efficient in providing the required infrastructure. So settlement value must considered by direct equations to avoid damage of building.

1.3 Aim and Objectives:

The main objective of this research is to:-

- Evaluate settlement of foundations due to tunneling.
- Study the effects of different variables which will be considered such as type of soils (sand and clay), depth and diameter of tunnels on foundation settlement.
- Settlement calculations will be calculate by applying numerical methods using the PLAXIS finite element software code
- Provide direct equations to calculate settlement due to tunneling in urban area where different loads and different type of soil were investigated.
- Compared developed equations with measured data available for (case study: - Shiraz metro line1) to verification the results.

1.4 Methodology

The methodology of works in this research will be in four steps as explained below:

Step I: literature review from books, papers and researches, which was talked about this object “Tunneling Effect on Foundation Settlement"

Step II: Making numerical analysis for many cases to obtained the relationship between different variables to obtained tunneling effect on foundation settlement

Step III: Validate the present numerical method, a comparison between the results obtained by finite element program “PLAXIS” and empirical analysis the problem was investigated theoretically via a parametric study performed by using the well-known finite element program “PLAXIS”.

Step IV: Conclusion and Recommendations.

1.5 Thesis layout

Chapter 1: An introductory chapter and provides general overview of the importance of prediction of settlement of shallow foundation over underground tunnels in highly dense populated and crowded area

Chapter 2: Literature review of all previous works related to the subject of "Settlement of Shallow Foundation Due to Tunneling ".A universally accepted principal of settlements pattern is the Gaussian function established by Schmidt (1969) and Peck (1969) for tunnels. In this thesis, a generalization of the expression proposed by Cording (1991) is used.

Chapter 3: Methodology of work will defined at this chapter where Basic Definitions, Sensitivity analyses, model geometry, finite element mesh, and boundary condition and material properties of sand will defined at this chapter.

Chapter 4: Settlement analysis using numerical method, Calculation of foundation settlement due to tunnel excavation is done by the PLAXIS finite element software

Chapter 5: Conclusion and Recommendations

CHAPTER 2

Literature Review

This chapter is a brief review of the previous studies dealing with settlement of foundation over tunnels. When structures are built over tunnels, it may be damaged due to excessive settlements under these building. Tunnels with different diameter and different depth causes different effect on building especially with large variables in soil properties. Therefore, a brief review of previous studies has been conducted the review covered a range of experimental, analytical and numerical work for better understanding of the subject matter.

The review was divided into two parts; the first was dealing with prediction of settlements by empirical analysis, and the second was dealing with prediction of settlements by analytical analysis.

2.1 Tunnel Type

The ancient people of Babylonia About 2180 to 2160 BC were the first to construct tunnels underneath the Euphrates River. These tunnels were used extensively for irrigation; and it was used as lines with length not exceeding 900m, which connect the royal palace with the temple. Ancient Egyptians was excavating temple rooms inside rock cliffs as Abu Simbel Temple on the Nile. A lot of temples were excavated in Ethiopia and India in the past. Design and excavation of tunnel in the past was depend on experience. Nowadays the design of tunnels developed by the development of geotechnical engineering where field data collected and computer programs developed to aid engineers. Also tunnel excavations has been developed where different machines have been used to excavate tunnel in different type of soil and rocks. In fact, difficult challenges faced the designer of tunnel with different geotechnical conditions underneath urban areas.

Scale used for the National Bridge Inventory is similar to tunnel were length of tunnel is based upon a condition assessment scale that varies from “0” to “9,” with 0 being the worst condition and 9 being the best condition. The length of a tunnel segment for which these ratings will be applied will vary with each tunnel.

Based on AASHTO Code 2001 the minimum roadway width between curbs, as shown in Figure 2.1, should be at least 0.6 m [2 ft.] greater than the approach traveled way, but not less than 7.2 m [24 ft]. The curb or sidewalk on either side should be a minimum of 0.5 m [1.5 ft.]. The total clearance between walls of a two-lane tunnel should be a minimum of 9 m [30 ft.]. The total width and the curb or sidewalk width can be varied as needed within the 9-m [30-ft] minimum wall clearance; however, each width should not be less than the minimum value stated above.

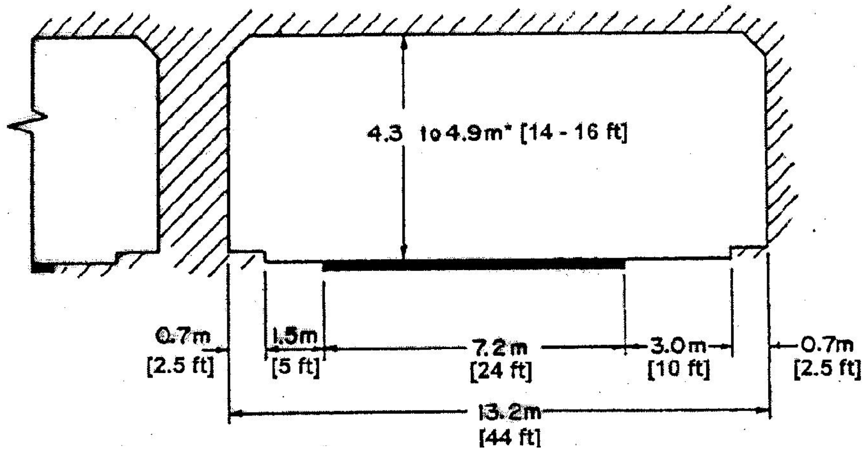


Figure 2.1: Typical Two-lane Tunnel Sections (Source AASHTO Code 2001)

Tunnel types are classified by their shape, liner type, invert type, and construction method. As a general guideline, a minimum length of 100 meters was used in defining a tunnel for inventory purposes. This length is primarily to exclude long underpasses; however, other reasons for using the tunnel classification may exist such as the presence of lighting or a ventilation system, which could override the length limitation.

2.1.1 Tunnel Shapes

There are four main shapes of highway tunnels as shown in Figures 2.2 to 2.5: circular, rectangular, horseshoe, and oval/egg. The different shapes depend on method of construction and the ground conditions. Some tunnels may be constructed using combinations of these types due to different soil conditions along the length of the tunnel.

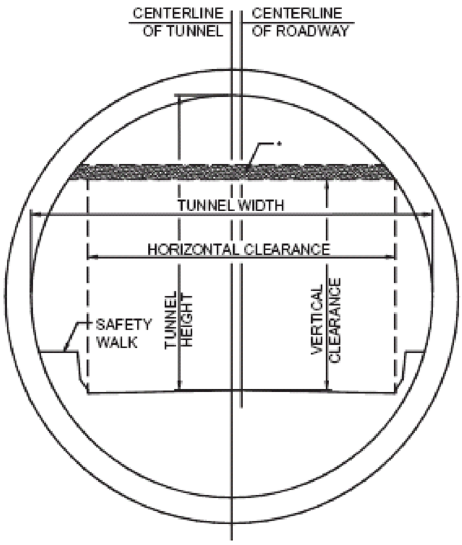


Figure 2.2: Circular tunnel with two traffic lanes and one safety walk (Source: Highway and Rail Transit Tunnel Inspection manual, 2005)

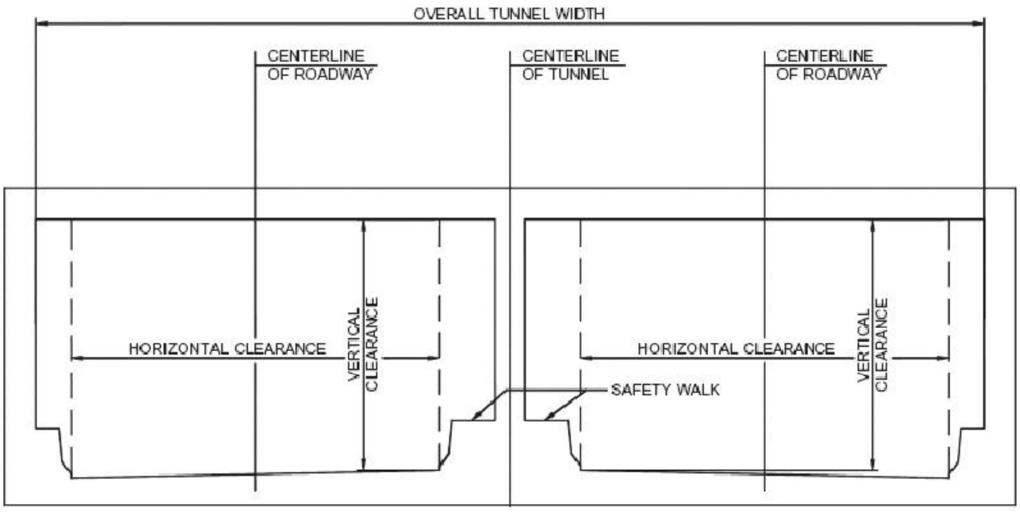


Figure 2.3: Double box tunnel with two traffic lanes and one safety walk in each box (Source: Highway and Rail Transit Tunnel Inspection manual, 2005)

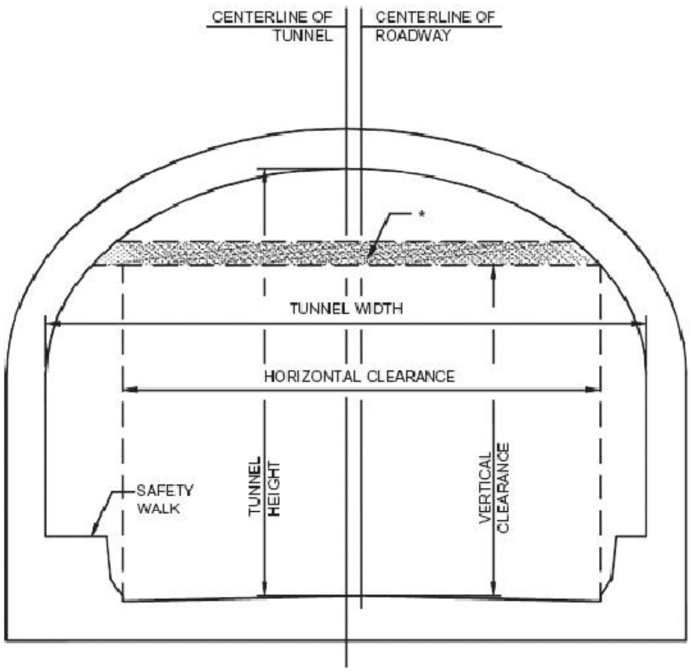


Figure 2.4: Horseshoe tunnel with two traffic lanes and one safety walk (Source: Highway and Rail Transit Tunnel Inspection manual, 2005)

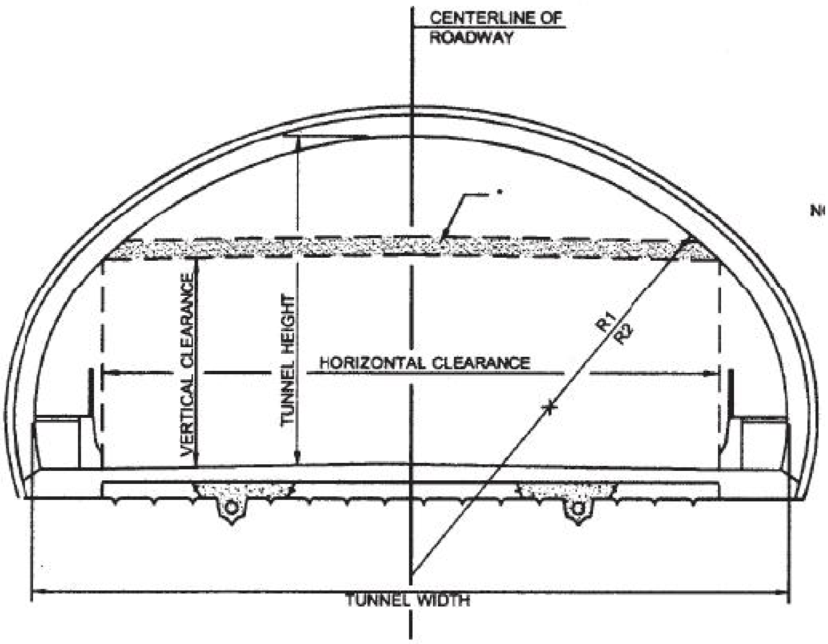


Figure 2.5: Oval/egg tunnel with three traffic lanes and two safety walks (Source: Highway and Rail Transit Tunnel Inspection manual, 2005)

2.1.2 Liner Types

Tunnel liner types can be classified (Ref. Tunnel Inspection manual, 2005)as following:

Unlined Rock

Rock Reinforcement Systems

Ribbed Systems

Segmental Linings

Poured Concrete

2.1.2.1 Unlined Rock

Unlined rock tunnel were no lining exists. Lining may be exists where zones of weak rock. This type of liner was common in older railroad tunnels.

2.1.2.2 Rock Reinforcement Systems

Rock reinforcement systems are used in rocks where tunnel is crossing to add additional stability to rock. Reinforcement systems include the use of metal straps and mine ties with short bolts to unify the rock pieces to produce a composite resistance to the outside forces.

2.1.2.3 Ribbed Systems

Ribbed systems are usually consist of a two-pass system for lining a drill-and-blast rock tunnel. The first pass consists of timber, steel, or precast concrete ribs usually with blocking between them to provide stability to the tunnel. The second pass typically consists of poured concrete that is placed inside of the ribs.

2.1.2.4 Segmental Linings

Segmental linings are primarily used in union with a tunnel boring machine (TBM) in soft ground conditions. The precast lining segments are constructed within the cylindrical tail shield of the TBM. These precast concrete segments are usually bolted together to compress gaskets for preventing water penetration.

2.1.2.5 Placed Concrete

Placed concrete linings are usually the final linings that are installed over any of the previous initial stabilization methods. They can be reinforced or unreinforced. They can be designed as a non-structural finish element or as the main structural support for the tunnel.

2.1.3 Construction Methods

As mentioned previously, the shape of the tunnel is dependent on the method used to construct the tunnel. Table 2.1 lists the six main methods used for tunnel construction with different shapes.

Table 2.1: Construction Methods of Tunnels
(Source: Highway and Rail Transit Tunnel Inspection manual, 2005)

Construction Methods	Circular	Horseshoe	Rectangular
Cut and Cover			X
Shield Driven	X		
Bored	X		
Drill and Blast	X	X	
Immersed Tube	X		X
Sequential Excavation		X	
Jacked Tunnels	X		X

2.1.3.1 Cut and Cover

Where trench is excavated in which the tunnel is constructed to the design finish elevation and then covered with various compacted soils. Supporting the soil is very important in this method during the excavation where sheet piles are used to construct the walls of a cut and cover tunnel.

2.1.3.2 Shield Driven

In shield driven method, a shield will be pushed into the soft soil ahead. Soil inside the shield is removed and a lining system is constructed around the tunnel before the shield is continue in pushing.

2.1.3.3 Bored

Bored method by using a mechanical (Tunnel boring machine) TBM in which the machine is excavated the tunnel with full diameter by a different cutting tools which depend on ground conditions (soft ground or rock). The TBM is designed to excavate and support tunneling until linings are finished.

2.1.3.4 Drill and Blast

In difficult ground conditions like rock where manually drill and blast the rock is used then rocks are removed using a conventional machine. Drilling and blasting method in generality was used for older tunnels and is still used when it need to reduce the cost where the laborer is available.

2.1.3.5 Immersed Tube

When the tunnel cross a channel, river, etc. immersed tube method is used. A trench is excavated under the water and precast tunnel segments are made then these segments are connected to produce the tunnel under water. After constructed the tunnel is covered and then protect the tunnel from the water.

2.1.3.6 Sequential Excavation Method (SEM)

Excavation of tunnel in cohesion soil like stiff clay or rock which have the strength to support the tunnel without direct support. This excavation method is called the sequential excavation method. The cohesion of soil or rock can be increased by injecting grouts into the ground before excavation of that segment.

2.1.3.7 Jacked Tunnels

Using cut and cover method in soft ground is impossible because of the existence of obstructions (highways, buildings, rail lines, etc.). This method is considered when the obstruction cannot be moved or temporarily disturbed. First specialized jacking equipment are constructed. Then tunnel sections are constructed and compulsory by hydraulic jacks into the soft ground, where the tunnel will encroaching through the soft soil.

2.2 Prediction of Settlements by Empirical Method

Excavation of tunnels in soft ground leads to ground movement. In an urban area, this movement can affect existing surface. While a semi-empirical methods are used is deal with ground movement due to tunneling under Greenfield area (i.e. there is no structures). These empirical methods is not suitable to predict settlement of structures due to tunnel construction.

Many research projects discussed the surface settlements caused by the construction of shallow tunnel at a Greenfield site. In rural area prediction of Greenfield settlement profiles can be estimated with high accuracy. But surface settlements that develop in urban areas where tunnel cross under buildings are less well understood. Field measurements of buildings subjected to tunnel induced settlements are available Lee van Kessel 2012 and Mohammad Ghafoori 2013. Field measurements show that surface settlement profiles are different from Greenfield site settlement. When designing of tunnel in urban area, surface settlement must be predicted due to tunneling to avoid any damage for surface structure.

The geometry and coordinate system shown in Figure 2.6, which will be adopted throughout the thesis. The coordinate system is defined as x represent the distance from the tunnel center in the transverse direction, y is the coordinate in the longitudinal direction and z is the depth under the surface.

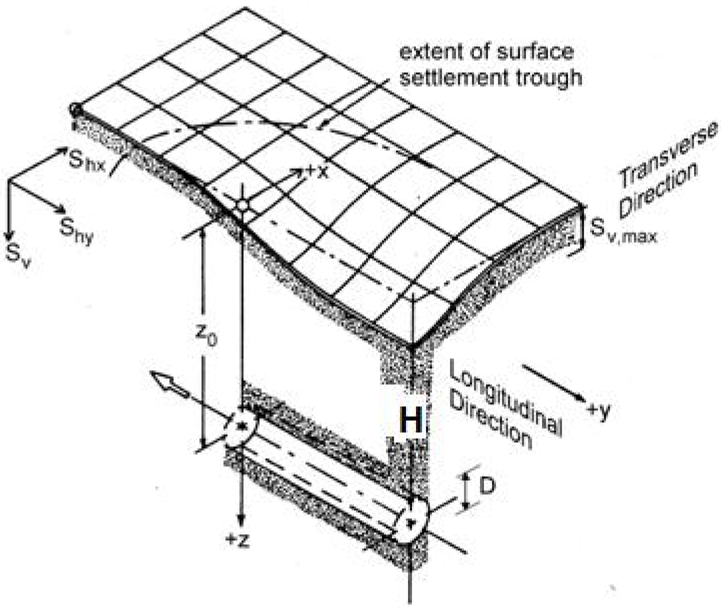


Figure 2.6: Geometry of the tunnel causes settlement by Burland et al. (2001)

It is accepted that the surface settlements can be represented by a Gaussian curve, shown in Figure 2.7 and represented by the formula;

$$\mathbf{s = s_{max}.\,e^{\frac{-y^2}{2i^2}}} \text{ .... } \mathbf{2\text{-}1}$$

Where S is the vertical settlement, S_{max} is the maximum vertical settlement, y is the transverse distance from the tunnel axis and (i) represents the distance of the inflection point from the axis. This description was first put forward by Martos (1958) and subsequently shown to be a valid approximation for the shape of the settlement trough above a tunnel in soft ground (Peck, 1969).

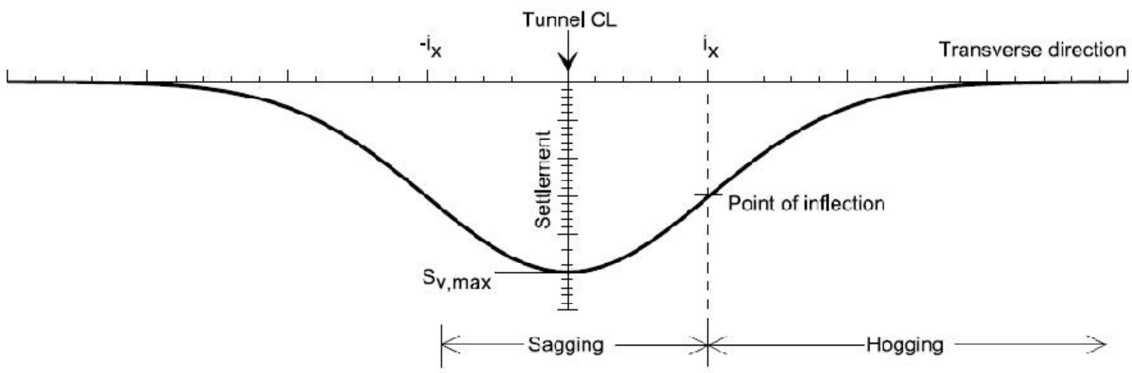


Figure 2.7: Transverse Gaussian settlement profile (sours J. Franzius 2003)

$$\mathbf{i = k.\,z} \text{ .... } \mathbf{2\text{-}2}$$

(i) is a linear function of the depth of the tunnel axis, z is the vertical level of the tunnel axis and 'k' is depending on the geotechnical characteristics of the ground where k is a trough width parameter which depends on the soil type and condition. Values of trough width parameter K vary in the range of 0.2 to 0.3 for granular materials above the water table and from 0.4 for stiff clays to approximately 0.7 for soft silty clay (O'Reillyand New, 1982; Rankin, 1988; and Mair et al., 1993).

The volume of the subsidence curve Vs is equal to (Eq.2.3) (Attewell et al., 1982):

$$\mathbf{V_s = \sqrt{2\pi}.\,i.\,s_{max} = 2.5.\,i.\,s_{max}} \text{ .... } \mathbf{2\text{-}3}$$

So the maximum settlement is:

$$\mathbf{s_{max} = \frac{V_s}{2.5.i}} \text{ .... } \mathbf{2\text{-}4}$$

The volume loss, V_L is the volume of the settlement trough per unit length expressed as a percentage of the total excavated volume of the tunnel,

$$\mathbf{V_L} = \frac{\mathbf{V_s}}{\mathbf{V_0}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \mathbf{2\text{-}5}$$

Where Vo is the volume required for tunnel. This is based on the assumption that soil movements occur under constant volume.

Volume loss is caused by the loss in the volume of soil excavated that need for construct of tunnel and the volume of the actual lined tunnel taking its place. Movement of soil around the tunnel fill this volume loss, it is dependent on the tunneling method of excavation and soil type (Potts and Zdravkovic, 2001). Sources of volume loss as shown in Figure 2.8.The volume loss V_L is related to (Guglielmetti et al., 2008):

Loss at the face where displacement of the ground at the face toward the machine.

Gap between the ground and the ring, i.e. the thickness of the shield.

Experience of contractor.

Alignment: In the curve with low radius, the driving operation of the machine can cause additional settlements.

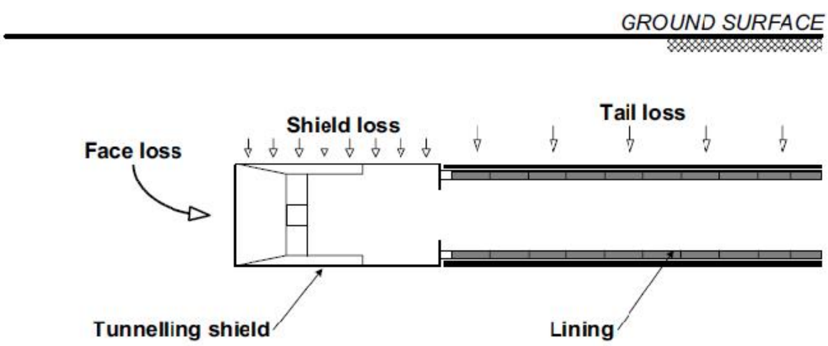
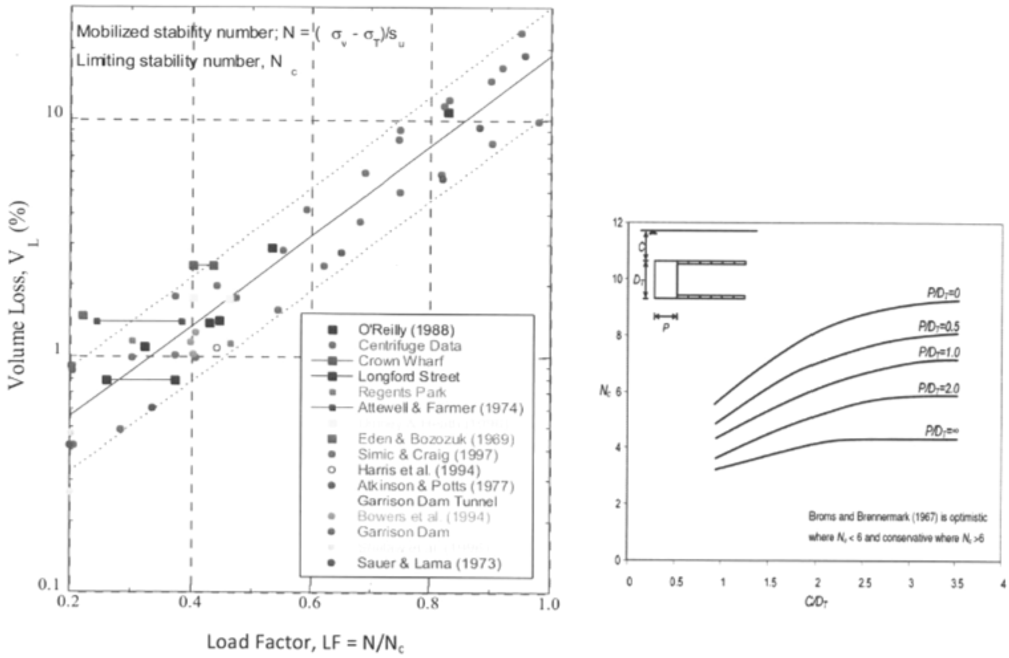


Figure 2.8: Sources of ground loss during soft ground tunnelling (sours J. Franzius 2003)

Macklin(1999)provided a relation between the volume loss ΔV_L for shallow tunnels in clay and the load factor (Figure 2.9) where LF $=N/N_c$, and N_c is the critical stability number derived by Kitamura and Mair (1981) and N is equal to:

$$N = (\sigma_v - \sigma_T)/Su \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2\text{-}6$$

Where σ_v is the overburden stress at the spring line elevation, σ_T the face pressure at the loading and Su, the undrained shear strength of the clay.



(a) Empirical estimate of ground loss for shallow tunnel in Clays (Macklin, 1999)

(b) Critical advanced stability of unsupported headings of tunnels in clay (Kitamura & Mair, 1981)

Figure 2.9 Empirical estimate of ground loss at the tunnel heading and correlation with stability number(sours Hoi. R- Law. C2012)

Recent experiences have shown that in sands and gravels, a high degree of settlement control can be achieved and small volume losses are recorded (i.e. often $V_L < 0.5\%$), while in soft clays, V_L ranges between 1% and 2%, excluding the long-term settlements.

The vertical settlement at any surface position can thus be found by combining equations2.1, 2.2 and 2.4 to give,

$$\mathbf{s} = \frac{\mathbf{V_s}}{\sqrt{2\pi}\mathbf{K z_0}} \cdot \mathbf{e}^{\frac{-\mathbf{y}^2}{2\mathbf{K}^2\mathbf{z_0}^2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{2-7}$$

Empirical method depends on past field observations in Greenfield conditions. In fact, settlement depends on various factors such as tunnel geometry, radius and depth, tunnel construction method, workmanship, soil type and volume loss. So empirical method is not valid in case of urban area where structures are exist above the tunnel.

2.3 Prediction of Tunnel Settlements by Analytical Methods

Useful and quick method of settlement prediction can be achieved using analytical methods. Many analytical solutions are described by Poulos and Davies (1980), where settlement prediction due to a point load in elastic half space. Settlement evaluated by integrating the solution for a line load equal to the magnitude of the weight of material excavated. Volume loss is neglected at this method. Chow's(1994)method considered volume loss and is based on in compressible irrational fluid. Chow derives the solution for vertical settlement as,

$$S = \frac{\gamma D^2 z_o^2}{4G(y^2+z_o^2)} \quad \text{2-8}$$

Where S is the vertical settlement, D is the tunnel diameter, γ is the soil density, and G is the shear modulus and z_o depth, y is the transverse distance from the tunnel axis.

A comparison between the analytical methods with Gaussian profile and field measurements from the Caracas Metro and M-40 Motorway in Madrid (Oteo and Sagaseta,1996) for settlement predictions it is noted that analytical methods produce a wider settlement more than the Gaussian profile and case study data with similar maximum settlement.

Celma and Izquierdo (1999) developed Sagaseta method and include the factors ϵ and δ which considered the ground loss of circular tunnels respectively and introduce equation for settlement for a is tunnel radius:

$$S = 2 \in a^2 \frac{z_o}{(y^2+z_o^2)} - 2\delta a^2 \frac{(y^2-z_o^2)}{(y^2+z_o^2)^2} \quad \text{2-9}$$

Settlement predictions according to Celma and Izquierdo method are found to be similar to the semi-empirical Gaussian profile.

Pinto and Whittle (2011) have also shown how the results are influenced by soil plasticity(close to the tunnel) and have developed closed-form solutions for uniform convergence of a 3-D tunnel heading. Pinto et al. (2011) compared 3-D tunnel analysis by series of case studies. In general small number of input parameters needed for analytical method that lead to predict settlement without field test for preliminary design in Greenfield conditions. But in urban areas analytical method is not suitable where weight of building is not considered so it must consider the loads of building.

2.4 Prediction of Tunnel Settlements by Numerical Method

The use of numerical methods to calculate settlements due to tunneling is becoming a very important for engineering practice. Finite element methods are used in calculated of tunneling problems. Clough and Leca (1989) and Negro and de Queiroz (2000) use a finite element models for tunneling analyses. Plane strain analyses are commonly used using software. PLAXIS, OXFEM, FLAC, ABAQUS…etc, were developed and successfully used for the objective of prediction of tunnel settlement. When using finite elements for modeling tunnel there are a number variables to be considered. It has been found that considering soil is a linear elastic material is unsuitable when predicted displacements (Rowe et al., 1983, Rankin, 1988 and Chow, 1994). Linear elastic-perfectly plastic models are developed by Rowe et al. (1983) who found that they give much more actual surface settlements than elastic models. Also Chow (1994) notes that the use of a linear elastic model where stiffness increases linearly with depth provides improved results.

Gunn (1993) also used a model combining non-linear elasticity at small strains with a Tresca yield criterion which predicted wider troughs than the Gaussian profile but good ground loss values.

For 3D analyses where some authors proved that, there is no difference in settlement trough between 2D and 3D analyses. (Ref. J. Franzius 2003)

In summary tunnel case settlements for building can be remodeled in a numerical method.

Modeling the soil can be achieve by these models:

Linear elastic isotropic soil conditions.

Linear elastic soil with increasing Young's modulus at increasing depth.

Non-linear elastic plastic soil

Multi surface plasticity soil.

Spring model

Also the tunnel can be modeled in different ways:

Remove soil elements and apply radial stresses on the tunnel boundary.

Remove soil elements and lining activation.

Remove soil elements, lining activation and application of radial stresses on the boundary.

Contraction of the tunnel area.(which use in this research)

In general it is noted that numerical modeling usually give a wider settlement profile than the Peck-formula, which could affect the results.

Some numerical predictions results are different from field measurements this difference refer to flexibility of numerical simulation. This will become more clear when study the effect on existing buildings due to tunneling.

2.5 Finite Element Method

Numerical methods are used to provide approximate solutions within an acceptable accuracy to analyze complex material properties with certain boundary conditions. After spreading of computer numerical methods are developed, finite element method (FEM) has been developed which solved these complex problem. FEM can solve problems such as nonlinear stress–strain behavior, and complicated boundary conditions. FEM is suitable to most problems for engineering applications, since mid-1950s with the first work by Argyris (1960) and Clough and Penzien (1993). FEM was applied first to the solution of plane strain problems and then to the solution of plates, shells, and axisymmetric solids.

2.5.1 Basic Principle

The finite element method is based on dividing the divide the body to a subdivision called finite elements, as shown in Figure 2.10 These elements are connected at certain nodes. Displacement functions are chosen to approximate the variation of displacements over each finite element. Polynomial functions are commonly employed to estimate these displacements. Equilibrium equations for each element are obtained by means of the principle of minimum potential energy. These equations are formulated for the entire body by combining the equations for the individual elements so that the continuity of displacements is preserved at the nodes. The resulting equations are solved satisfying the boundary conditions in order to obtain the unknown displacements.

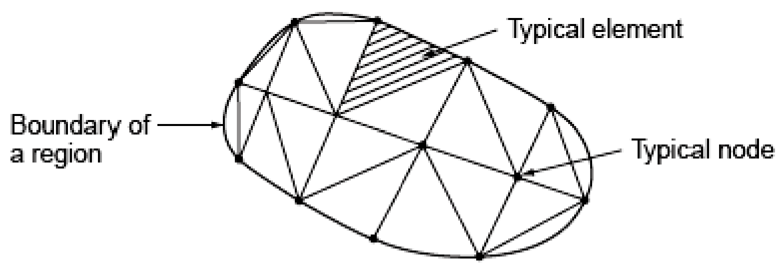


Figure 2.10: Assembly of subdivisions (Ref. PLAXIS reference manual 2012)

The entire procedure of the finite element method involves the following steps:

1. Structure subdivided into an equivalent system of finite elements.
2. Acceptable displacement function is chosen.
3. The element stiffness matrix is derived using a variational principle of mechanics, such as the principle of minimum potential energy.
4. The global stiffness matrix for the entire body is formulated.
5. The algebraic equations thus obtained are solved to determine unknown displacements.
6. The element strains and stresses are computed from the nodal displacements.

2.5.2 Choice of Element Shape and Size

A finite element generally has a simple one-, two-, or three-dimensional configurations. The boundaries of elements are often chosen as straight lines, and the elements can be one-, two-, or three-dimensional, as shown in Figure 2.11. While subdividing the continuum, one has to decide the number, shape, size, and configuration of the elements in such a way that the original body is simulated as closely as possible. Nodes must be located in positions where sudden changes in geometry, loading, and material properties occur. A node must be placed at the point of application of a concentrated load because all applied loads are converted into equivalent nodal-point loads.

It is easy to subdivide a continuum into regular elements having the same shape and size. But problems encountered in practice do not involve regular shape. They may have regions of steep gradients of stresses. A finer subdivision may be necessary in regions where stress concentrations are expected in order to obtain solutions that are more accurate. Typical examples of mesh selection are shown in Figure 2.12.

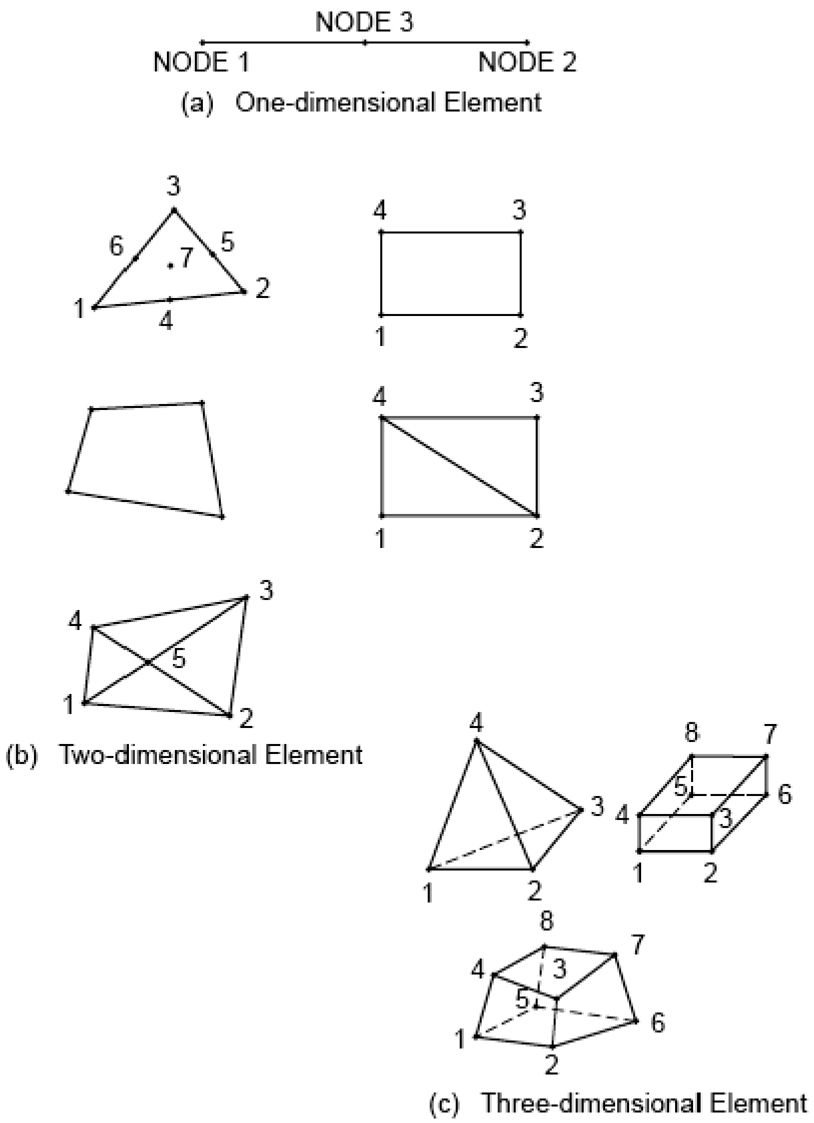


Figure 2.11: One-dimensional Element, (b) Two-dimensional Element, (c) Three-dimensional Element.(Ref. PLAXIS reference manual 2012)

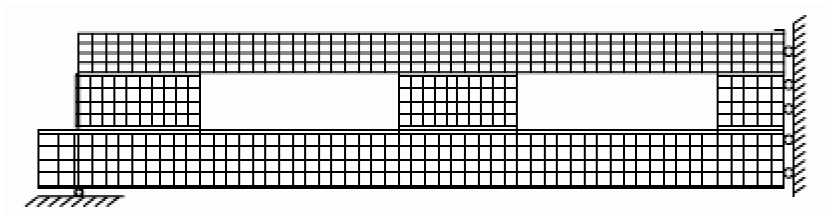


Figure 2.12: Typical example of finite element mesh.(Ref. PLAXIS reference manual 2012)

2.5.3 Soil Models

2.5.3.1 Mohr-Coulomb Model

It is known that, a point of Mohr's circle defines the normal stress and the corresponding shear stress on a certain plane. The stresses on all planes are formed Mohr's circle, because when a plane rotates the stress point traverses Mohr's circle.

Mohr-Coulomb failure criterion has been indicated in Figure 2.13, in the form of two straight lines, both of them making an angle ϕ with the horizontal axis. Their intersection with the vertical axis is at distances that equal the cohesion of soil (c). In order to indicate that failure of a soil is determined by the effective stresses, the stresses in this figure have been illustrated as σ'. There are two failure planes, defined by the points C and D in Figure 2.13, in which the stress state is critical. On all other planes the shear stress remains below the critical value. Thus it can be expected that failure will start to occur whenever Mohr's circle just touches the Mohr-Coulomb envelope.

The Mohr-Coulomb model requires five soil parameters, which are generally considered as the most parameters in geotechnical engineering. The required parameters can be obtained from basic soil tests. These parameters are as follows;

E = The Young's modulus of soil.

ν = Poisson's ratio of soil.

ϕ = The angle of internal friction of soil.

c = Cohesion of soil.

ψ = Dilatancy angle of soil.

The mathematical formulation of the Mohr-Coulomb failure criterion can be found by noting that the radius of Mohr's circle is equal $\frac{1}{2}\left(\sigma_1^{'}-\sigma_3^{'}\right)$, and that the distance from the origin to the circle center is equal to $\frac{1}{2}\left(\sigma_1^{'}+\sigma_3^{'}\right)$. Failure will occur if:

$$\sin\phi=\frac{\frac{1}{2}\left(\sigma_1^{'}-\sigma_3^{'}\right)}{c\cot\phi+\frac{1}{2}\left(\sigma_1^{'}+\sigma_3^{'}\right)}$$

………………2-10

This can also be re-written in the form:

$$\left(\frac{\sigma_1^{'}-\sigma_3^{'}}{2}\right)-\left(\frac{\sigma_1^{'}+\sigma_3^{'}}{2}\right)\sin\phi-c\cos\phi=0$$

……..…………..2-11

Using the above equation the value of $\sigma_3^{'}$ in the failure state can be expressed into $\sigma_1^{'}$,

$$\sigma_3^{'}=\sigma_1^{'}\frac{1-\sin\phi}{1+\sin\phi}-2c\frac{\cos\phi}{1+\sin\phi}$$

…………………..2-12

On the other hand, the value of $\sigma_1^{'}$ in the failure state can also be expressed into $\sigma_3^{'}$,

$$\sigma_1^{'}=\sigma_3^{'}\frac{1+\sin\phi}{1-\sin\phi}-2c\frac{\cos\phi}{1-\sin\phi}$$

…………………2-13

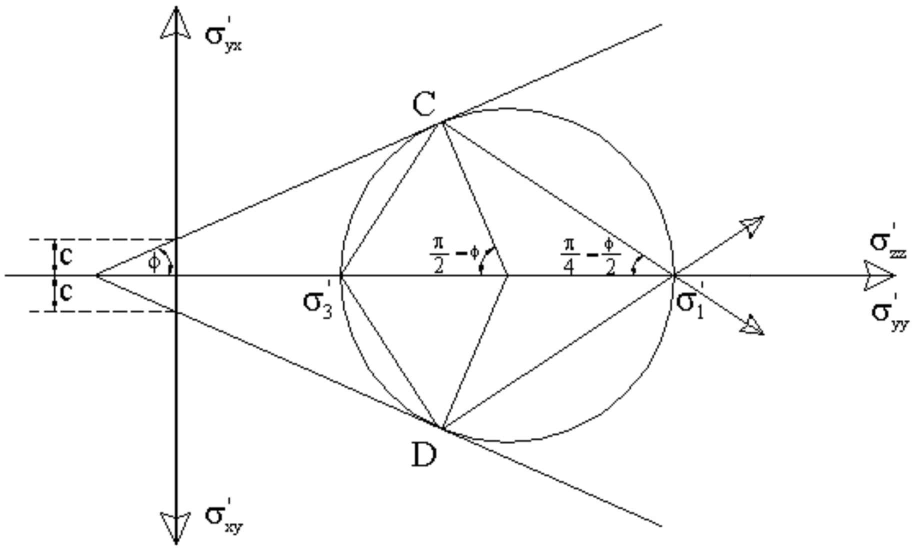


Figure 2.13: Mohr-Coulomb Failure Criterion

Friction angle

The friction angle determines the shear strength by means of Mohr's circles as shown in Figure 2-14. Part a corresponds to the friction angle used to model the effective friction of the soil, and part b shows how the friction angle is set to zero when cohesion parameter is equal to the un-drained shear strength of the soil.

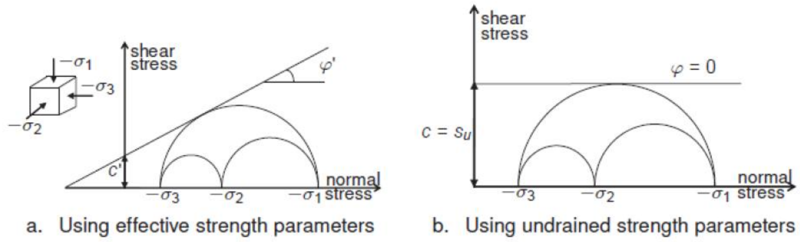


Figure 2.14: Stress circles at yield: one touches the Coulomb's envelope (Brinkgreve R.B.J 2004)

2.5.3.2 Hardening Soil Model (HS model)

Stiffness is the main difference between the hardening Soil Model (HS) which an advanced elasto- plastic soil model and the Mohr -Coulomb model. In HS model it is possible to model the soil more accurately with the use of three different input stiffness. So results of this model attempts a better approximation to real soil behavior as illustrated by Figure 2.15.

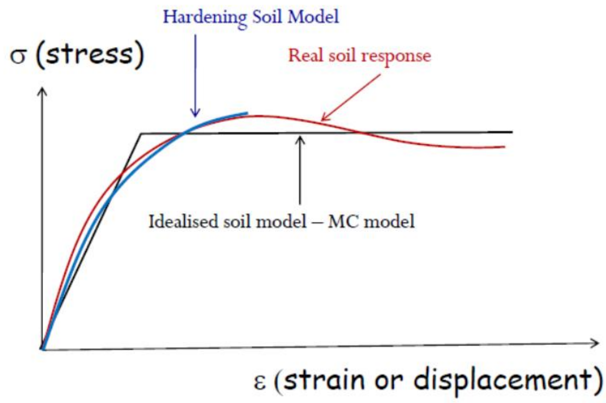


Figure 2.15: Comparison of HS and MC model with real soil response (Source: Ehsan. R 2012)

2.6 Assessment of Building Risk

Tunneling in urban areas affects the existing building with different degrees. So assessing the risk of damage is a very important for design the tunnel in urban area. This section will summarize the approach to predict and assess possibility of building damage.

2.6.1 Definition of Structure Deformation

Burland& Wroth1974 suggested parameters to define building deformation. Deformation parameters, shown in Figure 2.16, are defined:

- Settlement defines as positive values means down wards movement (Figure 2.16a).
- δS_v As shown in Figure 2.16a is the differential settlement between two settlement values.
- The slope angle θ denoted to the change in gradient of the straight line and two reference points in the structure (Figure 2.16a).
- Angular strain α denoted to the angle at turning as shown in (Figure 2.16a).
- Maximum relative deflection Δ describes the maximum of two reference points with a distance L as shown in (Figure 2.16b).
- Deflection ratio DR is defined as division of relative deflection Δ and length L: DR= Δ /L (Figure 2.16b).
- Tilt ω describes the rotation of structure rotation of the whole superstructure as shown in (Figure 2.16c).
- Relative rotation or angular distortion β is defined as the rotation of the straight line after rotation of structure (Figure 2.16c).
- Average horizontal strain ε_h develops as a change in length δL over the corresponding length L: $\varepsilon_h = \delta L/L$.

Previous definitions by Burland& Wroth (1974) are widely use in assessment of building damage.

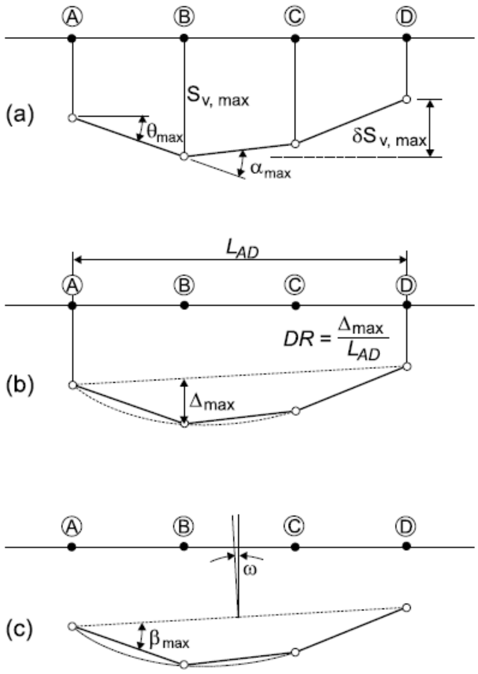


Figure 2.16: Definition of building deformation (Burland, 1995).

2.6.2 Risk Category

Cracks in the structure are the base of risk category which given by Burland *et al.* (1974). Rankin (1988) classified risk categories for structures with isolated foundations, where relative deflection values for settlement and angular deformation are produced. As shown in Table 2.2, the quantity of damage is classified as:

Aesthetic damages*:* which refer to slight cracking in the structures, where affecting on structure finishes. These effects repaired with low cost.

Functional damages: Parts of the structure loss of functionality by damages. These effects repaired with high cost.

Structural damages: big cracking or high deformation of structural elements. a collapse risk of the part or all structure.

Table 2.2: Relation between risk categories and counter-measures (M.Vahdatirad, H.Ghodrat, S.Firouzian and A.Barari 2010)

Risk category (*Burland*)	Measures to be applied before and/or during the excavation	Risk category (*Rankin*)
0 (aesthetic)	Any requirement	1 (aesthetic)/ **Negligible**
1 (aesthetic)		
2 (aesthetic)	Monitoring of the building and activation of the counter-measures if necessary	2 (aesthetic)/ **light**
3 (aesthetic /functional)	Safety measures (grouting or structure consolidation) to be realized before the execution of the new construction. Monitoring of the building and activation of the counter-measures if necessary	3 (functional)/ **medium**
4 (functional)		
5 (structural)		4 (structural)/ **high**

Classification proposed by Burland (1974) and Rankin (1988) are referred to buildings in good condition. This limit value shall be updated taking into account the vulnerability index of the buildings in the next section.

2.6.3 The Vulnerability Index I_v

Tunnel construction in urban area may affect damage the existing building. Therefore there is a need to investigate. Tunneling on existing building. Vulnerability is defined as the properties of exist and its vulnerability. The vulnerability is estimated by site investigation of the buildings that called Building Condition Survey (BCS). The properties of building classified by evaluating structural behavior based on number of floors, dimension of the building, foundation type, building utilization, age of the building, Orientation and the exact location of tunnel which cross under building. Vulnerability index identify by sum the weight of each previous item. Low values of the vulnerability mean that the building have high resistance for deformation. Table 2.3 shows a correlation between the threshold values by the Rankin and Burland formulation and the risk categories through a vulnerability index evaluation.

Table 2.3: Correlation between the threshold values by Rankin and Burland formulation and risk categories through vulnerability index evaluation (Chiriotti 2000).

Category of Damage	Negligible		Low		Slight		Medium		High	
	$0 < I_V < 20$		$20 < I_V < 40$		$40 < I_V < 60$		$60 < I_V < 80$		$80 < I_V < 100$	
	Control Parameter									
	$S_{max}(mm)$	β_{max}	$S_{max}(mm)$	β_{max}	$S_{max}(mm)$	β_{max}	$S_{max}(mm)$	β_{max}	$S_{max}(mm)$	β_{max}
1	<10	<1/500	<8	<1/625	<6.7	<1/750	<5.7	<1/875	<5	<1/1000
2	10-50	1/500-1/200	8-40	1/625-1/250	6.7-33	1/750-1/300	5.7-28.5	1/875-1/350	5-25	1/1000-1/400
3	50-75	1/200-1/50	40-60	1/250-1/63	33-50	1/300-1/75	28.5-43	1/350-1/88	25-37.5	1/400-1/100
4	>75	>1/50	>60	>1/63	>50	>1/75	>43	>1/84	>37.5	>1/100

2.6.4 Threshold Values

Once the risk category has been evaluated, it will be defined if the building needs special consolidation measures or monitoring during construction. There are three possible categories of actions listed in Table 2.4. These actions are associated to different risk categories.

Table 2.4 Actions related to the damages and risk categories in the building. (M.Vahdatirad, H.Ghodrat, S.Firouzian and A.Barari 2010)

Actions	Description	Risk Category
TYPE A	Special monitoring system and consolidation measures before the passage of the TBM	3-4
TYPE B	Special monitoring system and consolidation measures to be executed before the passage of the TBM in case the monitoring confirms the necessity	2
TYPE C	Buildings that require a light monitoring system and any consolidation measures	1

CHAPTER 3

Working Plan

3.1 Basic Definitions

"PLAXIS" is a finite element program, developed and carefully designed for modeling the stability problems in geotechnical engineering projects. The program is marked by the simple requirements for the input data and the enhanced outputs. The input data can be summarized in two requirements, the first is a simple graph representing the geometry of the problem, whereas, the second is the material model. The term "material model" means the physical properties of all the components of the problem. Most of the geotechnical problems are usually have two interactive components, soil and structure.

3.1.1 The Model Geometry

The geometry of any problem is introduced to the program, as graphical input data, via three components "Points", "Lines", and "Clusters". The points are basically define the ends of lines but can also be used for positioning the locations of some external effects such as concentrated loads and some internal effects such as points of fixation. The lines are used for defining physical boundaries and artificial model boundaries. The subsurface soil is introduced as clusters bounded by a set of intersecting lines. Within a cluster, soil is considered as a homogeneous material. So that a stratified soil deposit is introduced as a set of clusters, each cluster defines a layer of the deposit.

3.1.2 Finite Element Mesh

The stressed zone that confined by physical and artificial boundaries is automatically discretized into a finite element mesh of 15-node triangle element. It is available to refine the mesh and to increase the number of element nodes within the considered area. The mesh can be refined to medium, fine, and very fine levels of discretization. Also the number of element nodes can be decreased to 6-nodes. Besides the nodes, each element contains a number of stress points at which the stresses and strains can be calculated. 6-node elements can contain 3 stress points, whereas 15-node elements can contain 12 stress points, as shown in Figure 3.1.

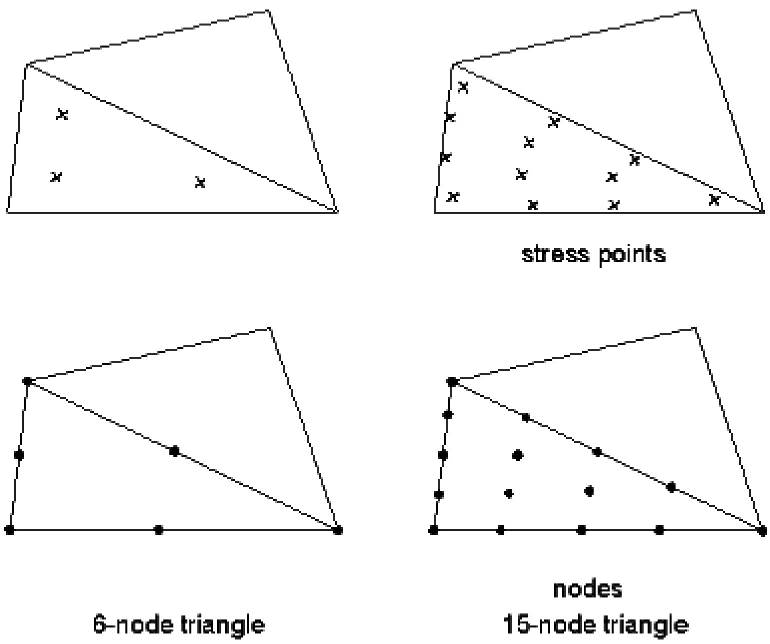


Figure 3.1 Nodes and Stress Points (Ref. PLAXIS reference manual 2012)

In addition, the mesh can be partially refined in selected area. This means that within specified boundaries the mesh can be much finer than outside these boundaries. This facility is useful for discretizing the critical and the highly stressed zones in the considered stability problems.

3.1.3 Material Model

In the geotechnical stability problems, there are many models can be used for introducing the soil. One of the well known models is the "Mohr-Coulomb Model" in this model, the failure criterion that considered is; $\tau_f = c + \sigma' \tan\phi$. Performing the program requires the following soil properties:

E = The Young's modulus of soil.

ν = Poisson's ratio of soil.

ϕ = The angle of internal friction of soil.

c = Cohesion of soil.

ψ = Dilatancy angle of soil.

For a specified case, the above properties can be measured during some laboratory soil test such as direct shear tests and/or triaxial compression tests.

3.2 Sensitivity Analysis

Sensitivity analyses are defined as conducting some numerical applications for a basic problem in order to obtain the most suitable parameters for numerical modeling. The choice of the basic parameters is depending upon the scope of the study. The current study is concerning with the tunneling effect on foundation settlement problems as shown in Figure 3.2.

The sensitivity analyses or the basic numerical tests were aimed to measure the effect of four factors on the stability of the outputs. The considered factors were, the mesh refinement, the horizontal boundary, the vertical boundaries and the considered clusters. During the sensitivity analysis, two types of elements were checked, 6-node elements and 15-node elements. The details of the sensitivity analysis are illustrated in the following sections.

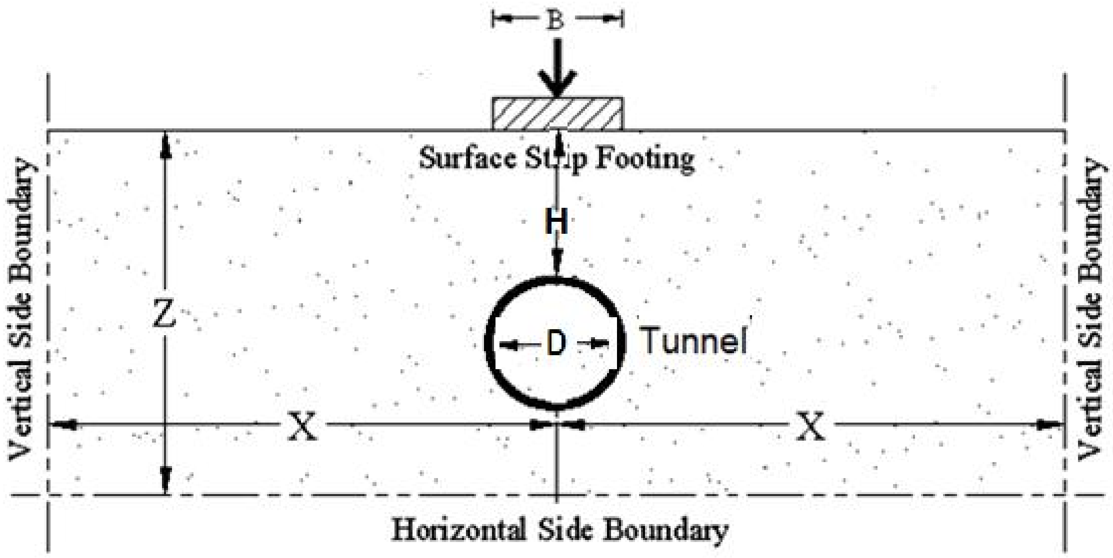


Figure 3.2 Basic Problem for Sensitivity Analysis

3.3 Numerical Modeling and Settlement Prediction

The objectives of this study are; investigating the effect of tunnels on various structures and infrastructural components such as depth and various diameters under the structures. The following variables will be considered:

Different Type of soils,

Tunnels depth,

Diameters of tunnels.

Settlement calculations of shallow foundations will performed applying numerical methods using the PLAXIS finite element software code. Therefore, a two-dimensional numerical modeling using finite element method will be considered.

The effect of different variables will be investigated as shown in Figure 3.3 below; The analysis will be based on the cases presented in Figure 3.4 below.

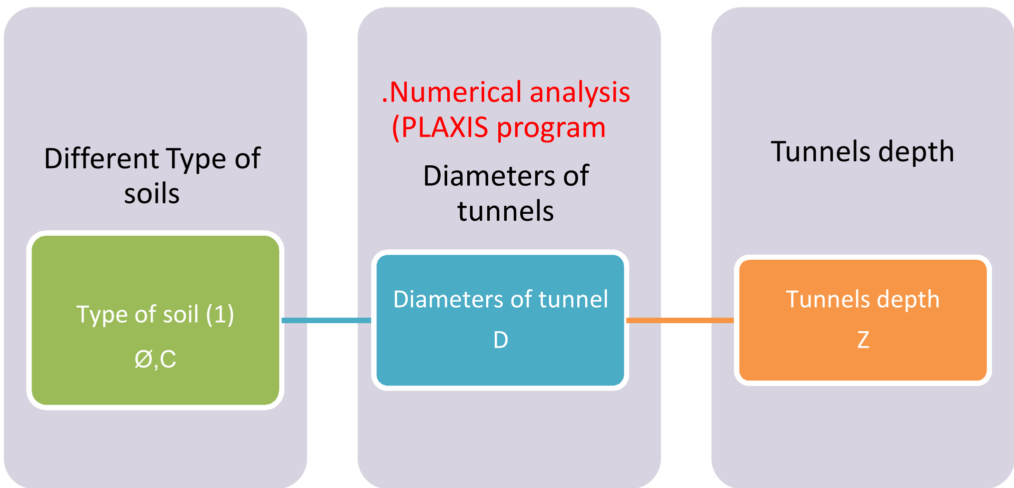


Figure 3.3 Basic of Empirical analysis

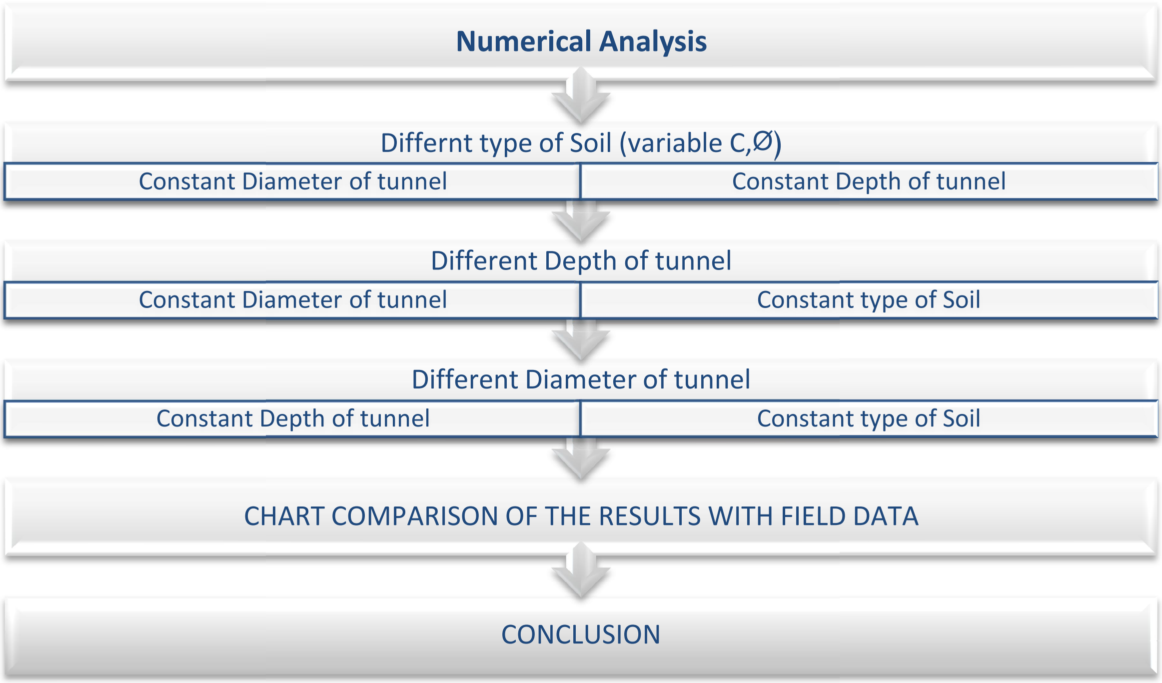


Figure 3.4 Basic of Numerical Analysis

3.4 <u>Model Geometry and Boundary Condition</u>

Model geometry and boundary condition are shown in Figure 3.5 The soil medium considered as a 100m by 60m (dimension area). The lateral and bottom boundaries are located (4 to 5) D where D is tunnel diameter so that the effects of boundaries on analysis would be insignificant. The lateral boundaries were assumed to be on rollers to move downward and the bottom boundary was fixed against translation. Tunnel was assumed at the center of this geometry with the variable diameters (5, 10, 15, and 20)m where maximum diameter 20mas shown in Figure 3.5. A concrete foundation with width 10m carry variable load from zero Load (Greenfield) to 2000 KN as a concentrated load.

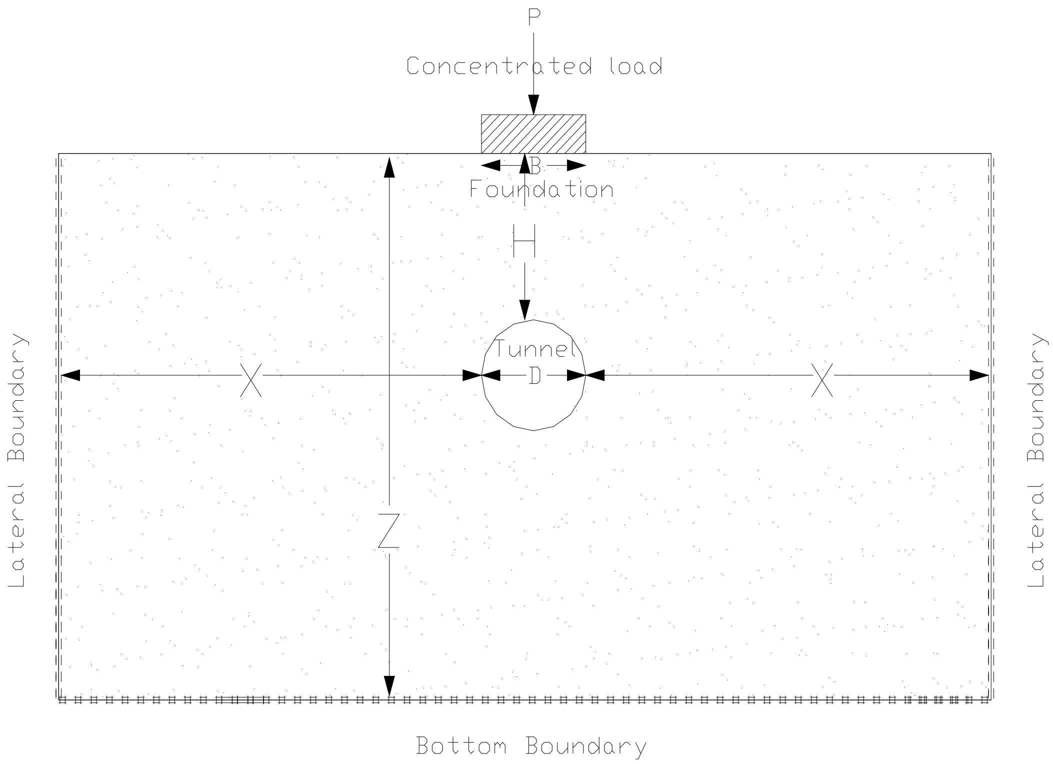


Figure 3.5: Basic Problem for Sensitivity Analysis

In order to make analysis model, following properties different type of soils Table 3.1 were used in PLAXIS, for different type of soils (Clay and Sand) these values provided from different references for each parameter used in analysis (as shown in Appendix B).

Table 3.1Material properties of soil

ID	Material Model	Type	γ_{Dry} (kN/m^3)	γ_{Sat} (kN/m^3)	K (m/day)	E (kN/m^2)	ν	C (kN/m^2)	Ø
Clay									
Soft	M.C	Drained	17.6	17.6	0.8	3500	0.25	50	0^o
Medium	M.C	Drained	18.54	18.54	0.8	8000	0.35	100	0^o
Hard	M.C	Drained	20.7	20.7	0.8	14000	0.49	200	0^o
Sand									
Loose	M.C	Drained	18.5	18.5	8.6	28000	0.2	0	32^o
Medium	M.C	Drained	19.95	19.95	8.6	50000	0.3	0	35^o
Dense	M.C	Drained	21	21	8.6	70000	0.4	0	40^o

CHAPTER 4

"RESULTS AND ANALYSIS"

4.1 <u>Settlement of Foundation Due to Tunneling</u>

Settlement has been calculated using PLAXIS code underneath foundations for different loads without the existence of tunneling. Table 4.1and Figure 4.1 show the results of settlement for footing setting on different types of soil under different loads. Results as expected, it increases with the increasing of loads and decreases as it moves from soft clay to dense sand.

Table 4.1: Foundation Settlement for different soils under different load values

	Settlement mm					
Load kN / **Type of soil**	**0 KN**	**100 KN**	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
Soft Clay	0	47.2	263.02	474	734.8	1180
Medium Clay	0	19.8	98.8	196.6	296.4	395.2
Hard Clay	0	8.9	43.4	86.3	130.4	173.8
Loose Sand	0	7	37.4	91.7	175.5	274.8
Medium Sand	0	3.8	20.2	49.9	91.3	141.5
Dense Sand	0	2.5	12.6	28.6	50.9	76.2

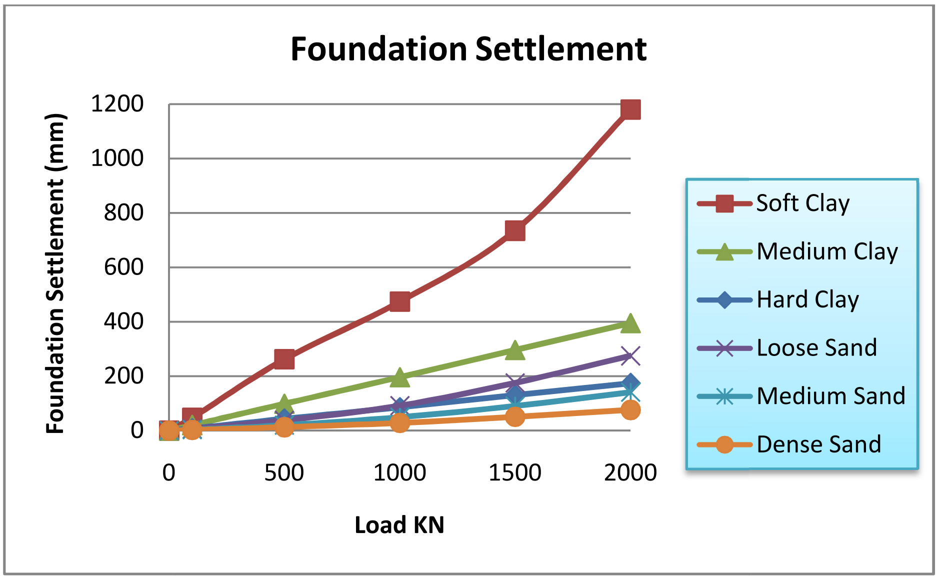


Figure 4.1:Foundation Settlement for different loads and different soils without tunnels

4.1.1 Settlement of Foundation Due to Tunneling Greenfield

At first stage analysis is carried out for tunnels with different depths and different diameters by PLAXIS for Greenfield where there is no concentrated load. Figures4.2 to 4.7indicated that foundation settlement decreased with the increasing in tunnel depth.

It is very clear from the results that as tunneling diameter increase the settlement increases by several folds for all type of soil used in this study. On the other hand results show that the settlement remain unchanged with depth of tunnels. Results also indicted an increase of settlement from sand toward clayey soil with highest settlement for soft clay**.** Results from Figures 4.2 to 4.7 for Greenfield condition for soft to medium clay indicated that settlement increases with increasing in tunnels diameter and it remain almost constant with depth of tunnels. Also results indicated a reduction in settlement values as we move from soft clay to hard clay and from loose sand to dense sand.

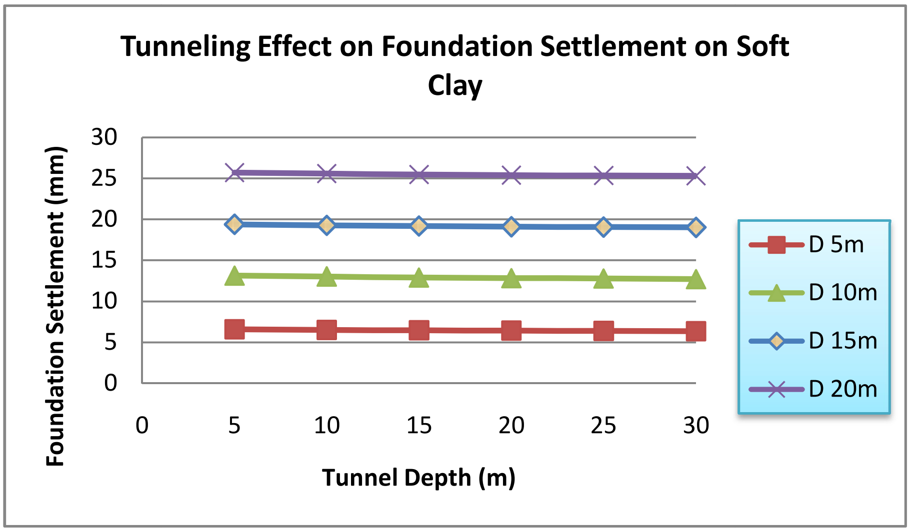


Figure 4.2: Foundation settlement in Greenfield condition in soft clay for different tunnel depth and diameter.

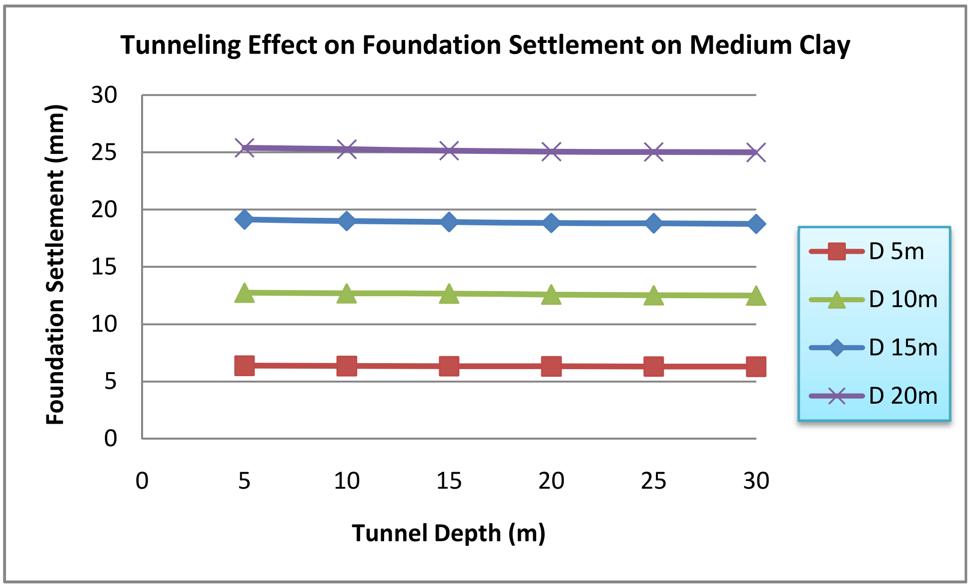


Figure 4.3: Foundation settlement in Greenfield condition in medium clay for different tunnel depth and diameter.

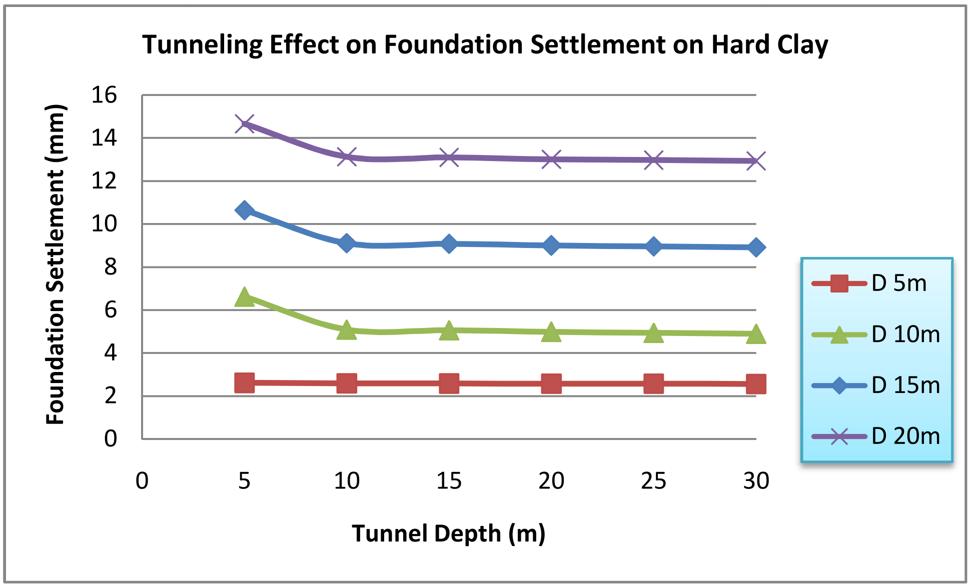


Figure 4.4: Foundation settlement in Greenfield condition in hard clay for different tunnel depth and diameter.

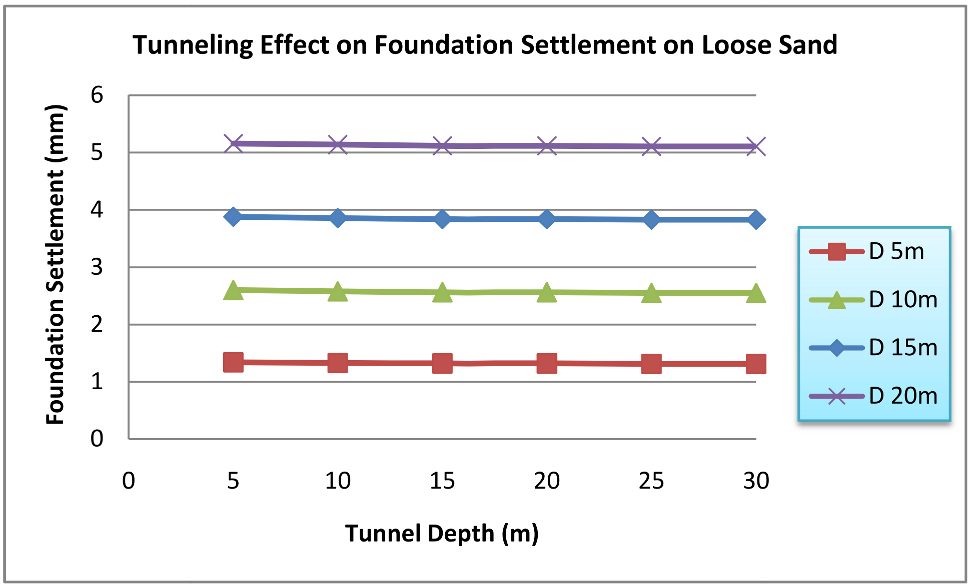


Figure 4.5: Foundation settlement in Greenfield condition in loose sand for different tunnel depth and diameter.

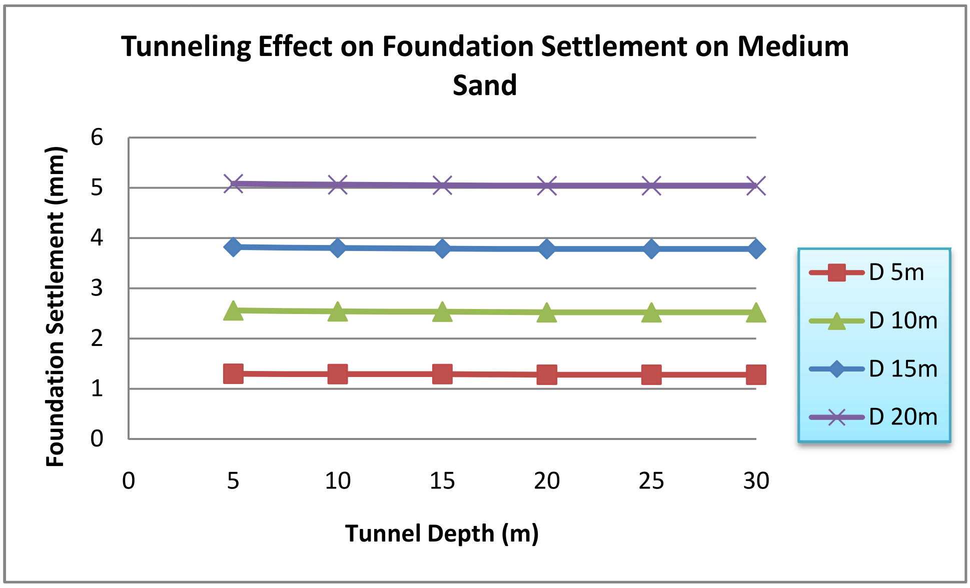


Figure 4.6: Foundation settlement in Greenfield condition in medium sand for different tunnel depth and diameter.

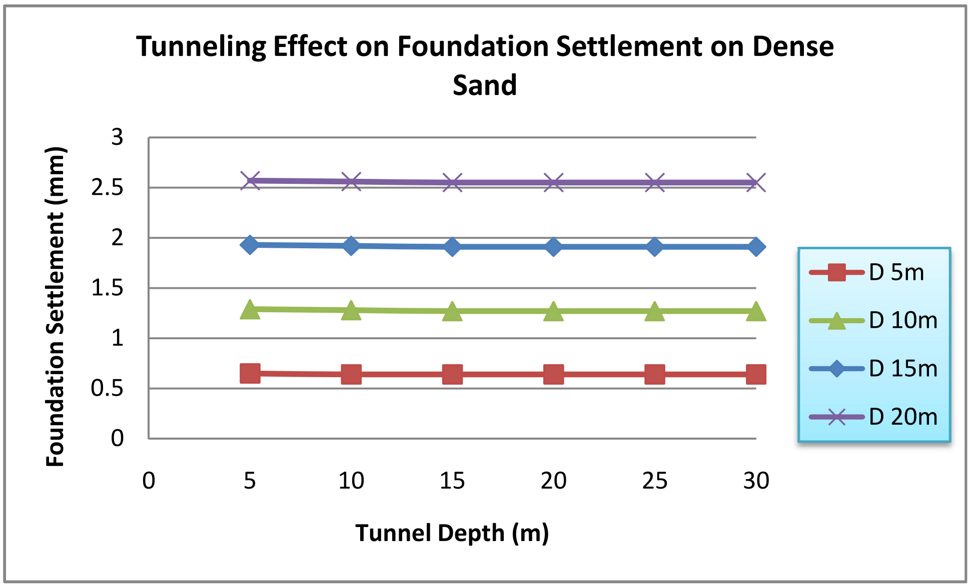


Figure 4.7: Foundation settlement in Greenfield condition in dense sand for different tunnel depth and diameter.

It is very clear from Figure 4.5 to 4.7 that settlement values of foundation on dense sand almost half the settlement of loose to medium dense sand for Greenfield condition Similar relationship was obtained for different stresses condition (50, 150, 200kN/m^2) for different type of soil. The results presented in appendix A.

4.1.2 <u>Prediction of Settlement of Foundation Due to Tunneling</u>

At this stage after analysis is done for tunnel with different depth and different diameter by PLAXIS due to more load (500KN, 1000KN, 1500KN, 2000KN). The relationship between H/D (Depth of tunnel/Diameter of tunnel) and settlement due to more load and prediction of settlement by different equations for different type of soil shown in Table 4.2 to Table 4.19 and Figure 4.8to Figure 4.25. It is clear from Figure 4.8 and Table 4.2 that settlement increases with increasing tunneling diameters and also increased as load increased. It is believed that settlement increases even with constant H/D because as Thickness increase the layer involve will be thicker and potential settlement will be higher (S1 to S7 versus settlement due to different loads as tunnels diameter and depth increases from S1 to S7 with constant ratio where H/D<1).

Table 4.2 Foundation settlement for H/D<1 in loose sand for different Loads.

	Foundation settlement in Loose Sand (mm) for H/D<1				
	Loads (KN)				
settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	1.33	38.05	92.74	171.68	270.44
S2	1.32	37.89	93.56	174.09	274.06
S3	2.55	38.11	94.69	178.33	280.03
S4	2.56	38.8	96.92	182.37	280.89
S5	2.56	39.56	100.03	184.41	285.01
S6	3.83	39.29	97.06	189.3	304.76
S7	5.11	41.03	99.48	196.4	316

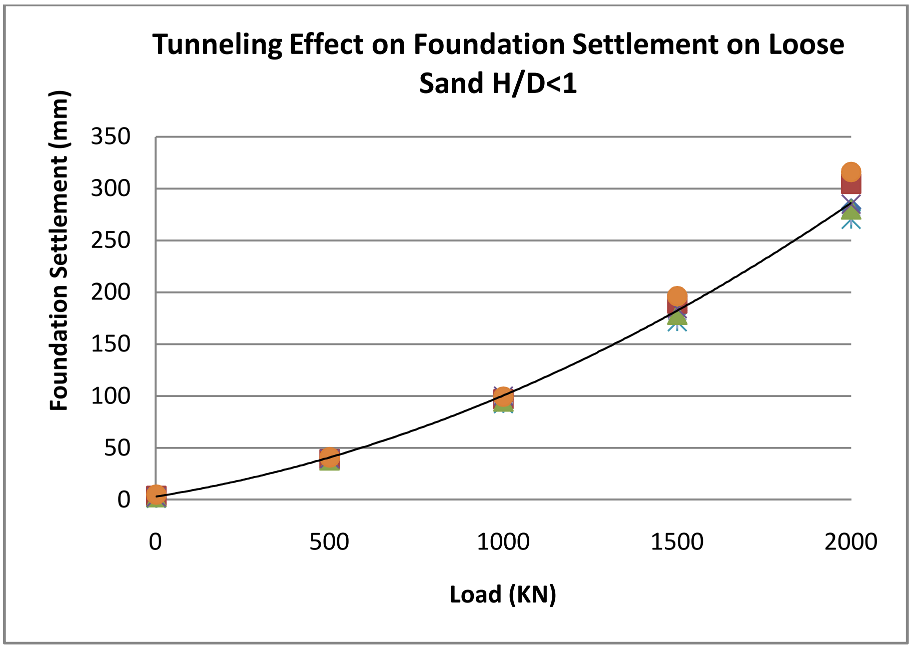


Figure 4.8: Foundation settlement for H/D<1 in loose sand for different Loads.

Equation 4.1 can be derived from relationships shown in Figure 4.8. So for loose sand settlement can be calculated from equation 4.1 where P is the external loads for the case of for H/D<1.

$$S_{max} = 5x10^{-05}xP^2 + 0.043P + 3.507 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 4\text{-}1$$

Table 4.3 Foundation settlement for H/D=1 in loose sand for different Loads.

	Foundation settlement in loose sand (mm) for H/D=1				
	Loads (KN)				
settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	1.34	38.63	92.8	171.74	277.04
S2	2.58	40.87	100.95	178.29	275.72
S3	3.84	42.77	112.5	202.64	299.94
S4	5.12	44.38	114.24	217.13	322.24

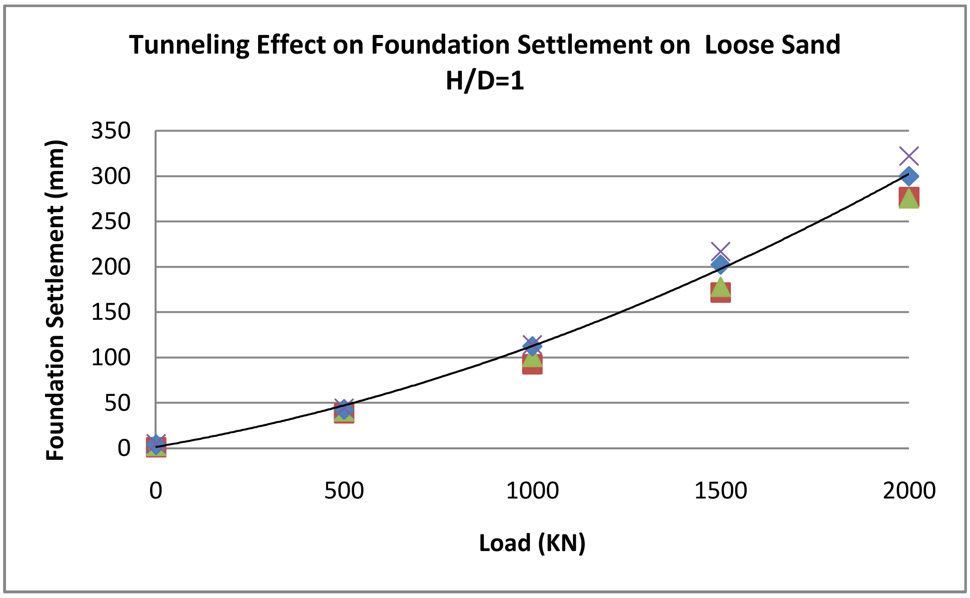


Figure 4.9: Foundation settlement for H/D=1 in loose sand for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in loose sand for H/D=1.

It is noted that settlement increases with increasing tunnel diameters and depth with same ratio H/D=1.

The results in Figure 4.9for H/D =1 indicated an increase in the vertical settlement underneath a foundation as the magnitude of the load increases. The same trend has been notice for different type of soils.

The maximum settlement for loose sand can be expressed in the form of equation 4.2

$$S_{max} = 4.5x10^{-05}xP^2 + 0.059P + 2.472 \ldots\ldots \quad 4\text{-}2$$

Table 4.4 Foundation settlement for H/D>1 in loose sand for different Loads.

	Foundation settlement in loose sand (mm) for H/D>1				
	Loads (KN)				
settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	**2.6**	44.52	100.75	173.95	263.61
S2	**5.14**	56.68	137.76	236.78	353.78

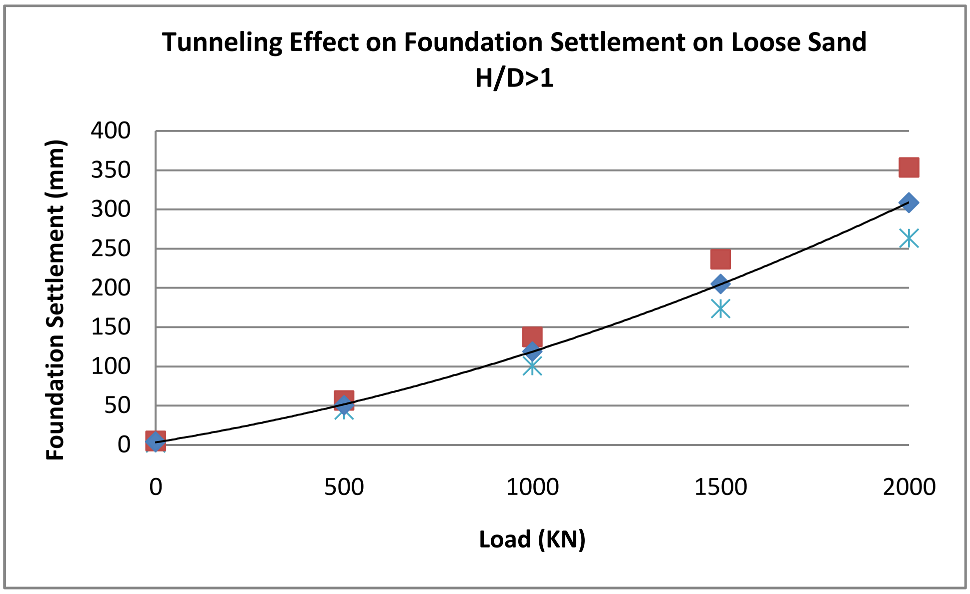


Figure 4.10 Foundation settlement for H/D>1 in loose sand for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in loose sand for H/D>1.

$$S_{max} = 4.5x10^{-05}xP^2 + 0.078P + 3.338 \ldots \quad 4\text{-}3$$

Table 4.5:Foundation settlement for H/D<1 in medium sand for different Loads.

	Foundation settlement in medium sand (mm) for H/D<1				
	Loads (KN)				
Settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	**1.29**	21.03	51.64	91.42	141.34
S2	**1.29**	20.7	52.23	90.95	142.13
S3	**2.52**	20.59	50.97	94.96	149.59
S4	**2.52**	21.02	53.13	100.55	152.06
S5	**2.53**	21.71	57.16	101.75	152.96
S6	**3.78**	21.31	51.96	99.57	161.17
S7	**5.04**	22.18	50.68	104.77	164.84

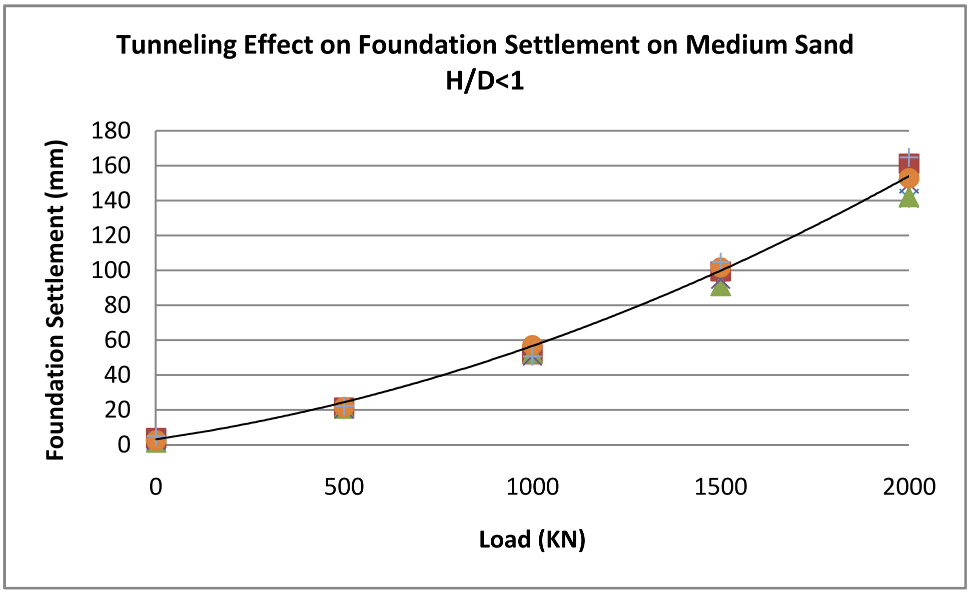


Figure 4.11: Foundation settlement for H/D<1 in medium sand for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation medium sand for H/D<1.

$$S_{max} = 2.5x10^{-05}xP^2 + 0.023P + 3.06 \quad \ldots\ldots\ldots\ldots \quad 4\text{-}4$$

Table 4.6:Foundation settlement for H/D=1 in medium sand for different Loads.

	Foundation settlement in medium sand (mm) for H/D=1				
	Loads (KN)				
Settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	**1.3**	22.61	52.63	91.82	144.11
S2	**2.54**	23.69	60.71	101.52	150.2
S3	**3.79**	23.52	64.97	117.86	171.36
S4	**5.04**	24.14	53.84	121.39	186.45

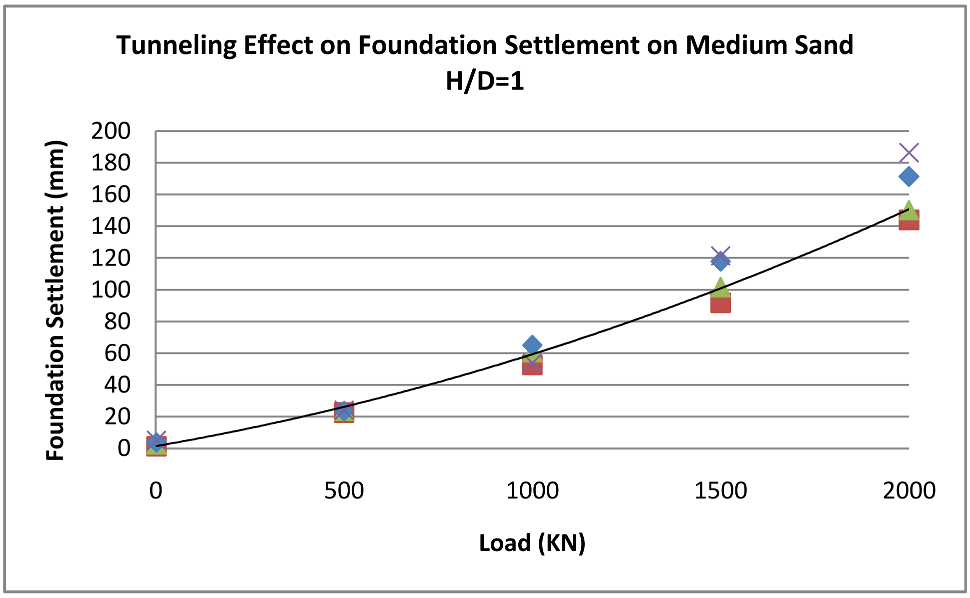


Figure 4.12: Foundation settlement for H/D=1 in medium sand for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in medium sand for H/D=1.

$$S_{max} = 3x10^{-05}xP^2 + 0.0235P + 3.149 \quad \ldots\ldots\ldots\ldots\ldots \quad 4\text{-}5$$

Table 4.7: Foundation settlement for H/D>1 in medium sand for different Loads.

	Foundation settlement in medium sand (mm) for H/D>1				
	Loads (KN)				
Settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	**2.56**	28.69	63.88	101.44	147.27
S2	**5.06**	28.77	70.76	148.11	200.5

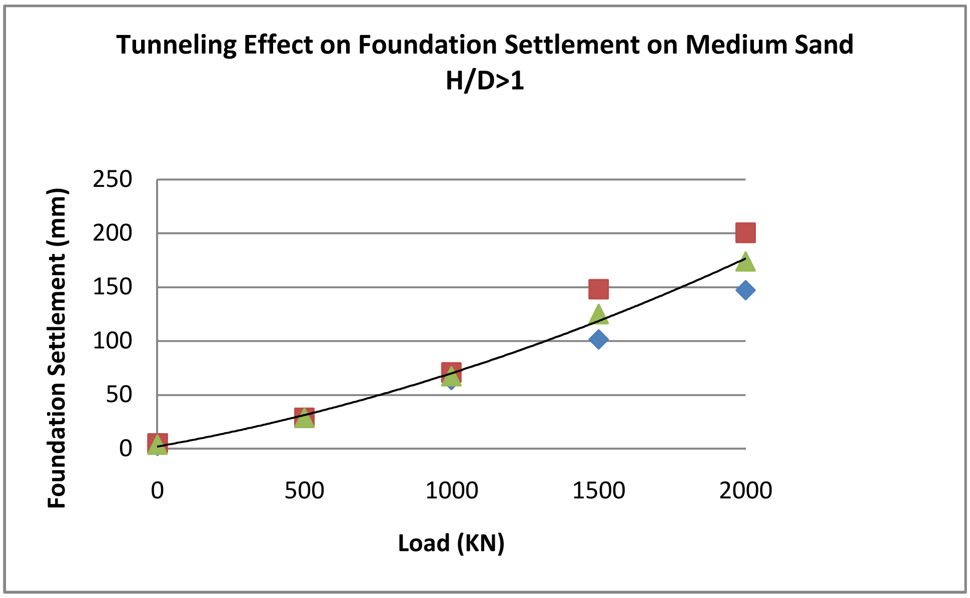


Figure 4.13: Foundation settlement for H/D>1 in medium sand for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in medium sand for H/D>1.

$$S_{max} = 2x10^{-05}xP^{2} + 0.0485P + 2.025 \qquad \text{4-6}$$

Table 4.8: Foundation settlement for H/D<1 in dense sand for different Loads.

	Foundation settlement in dense sand (mm) for H/D<1				
	Loads (KN)				
Settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	0.64	13.19	31.3	52.27	78.36
S2	0.64	13.06	30.8	53.36	78.84
S3	1.27	13.29	30.65	59.45	88.26
S4	1.27	12.94	29.11	52.07	83
S5	1.27	13.75	34.67	62.05	88.76
S6	1.91	13.39	29.97	53.74	90.64
S7	2.55	14	29.17	55.36	91.51

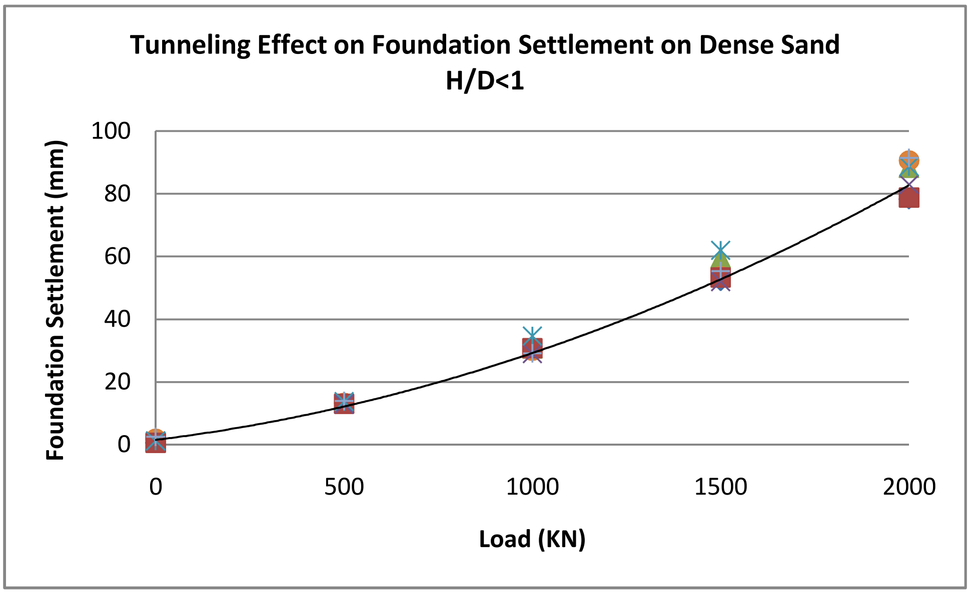


Figure 4.14: Foundation settlement for H/D<1 in dense sand for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in dense sand for H/D<1.

$$S_{max} = 1.45x10^{-05}xP^2 + 0.015P + 1.846 \qquad \text{4-7}$$

Table 4.9: Foundation settlement for H/D<1 in dense sand for different Loads.

	Foundation settlement in dense sand (mm) for H/D=1				
	Loads (KN)				
Settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	0.65	14.89	32.54	54.26	81.5
S2	1.28	15.42	39.67	64.48	89.87
S3	1.91	14.99	40.1	73.29	103.71
S4	2.55	15.36	31.25	67.07	104.85

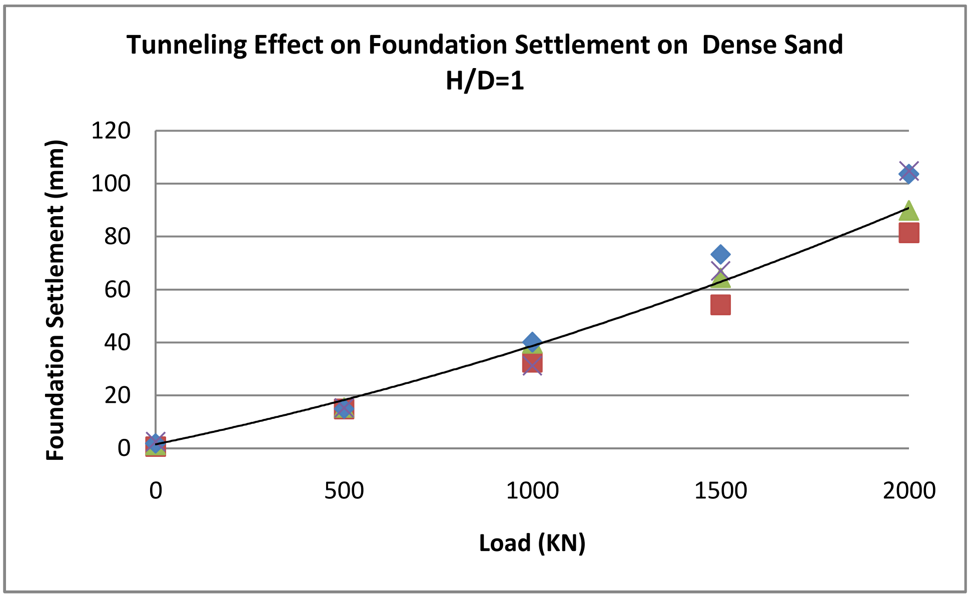


Figure 4.15: Foundation settlement for H/D=1 in dense sand for different Loads

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From fit line which show the relationship between the concentrated force and maximum settlement of foundation in medium sand for H/D=1.

$$S_{max} = 0.95x10^{-05}xP^2 + 0.025P + 0.6885 \ldots\ldots\ldots\ldots\ldots\ldots \quad 4\text{-}8$$

Table 4.10: Foundation settlement for H/D>1 in dense sand for different Loads.

	Foundation settlement in dense sand (mm) for H/D>1				
	Loads (KN)				
Settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	1.29	19.76	46.68	69.31	93.8
S2	2.56	18.78	59.33	81.52	114.71

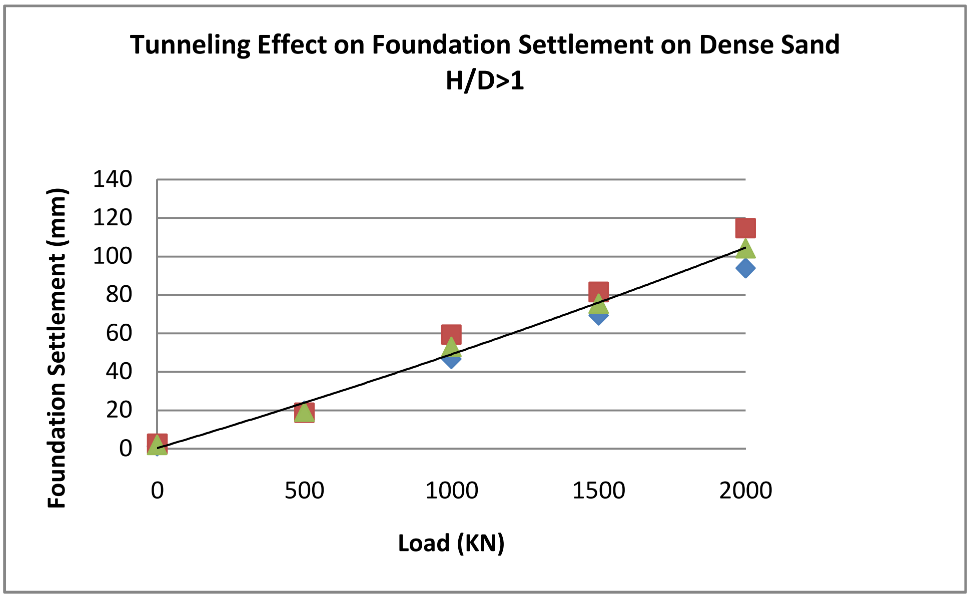

Figure 4.16 Foundation settlement for H/D>1 in dense sand for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in dense sand for H/D>1.

$$S_{max} = 3x10^{-06}xP^2 + 0.045P + 0.279 \ldots\ldots 4\text{-}9$$

Table 4.11: Foundation settlement for H/D<1 in soft clay for different Loads.

	Foundation settlement in soft clay (mm) for H/D<1				
	Loads (KN)				
Settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	6.51	234.83	470.25	723	1170
S2	19.01	243.39	479.45	735	1176
S3	12.72	237.81	474.48	729.6	1178
S4	12.84	237.19	472.16	725.44	1179
S5	6.47	234.85	472.8	728.84	1180
S6	12.92	236.55	468.4	717.53	1182
S7	25.31	252.71	493.09	752.02	1186

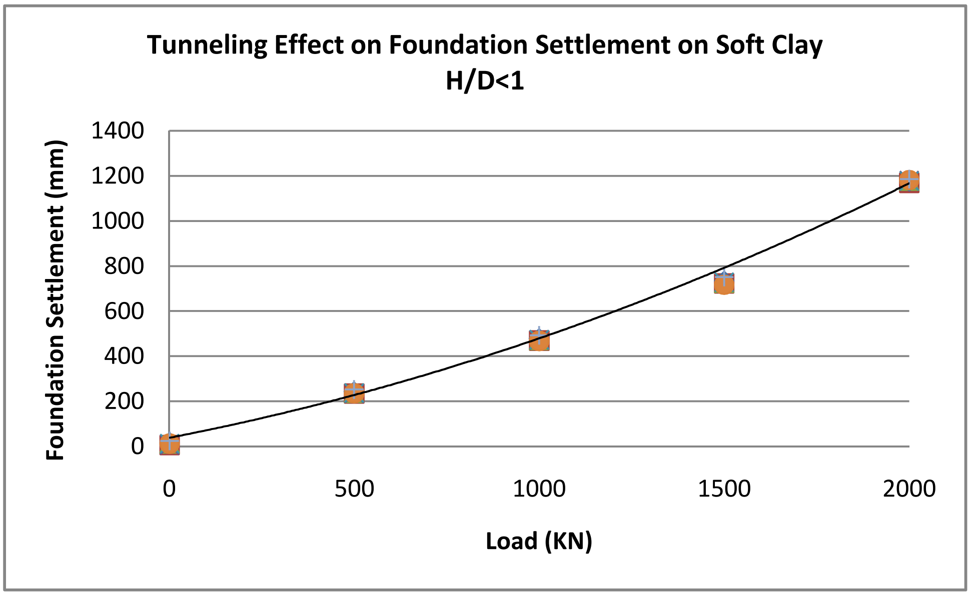

Figure 4.17 Foundation settlement for H/D<1 in soft clay for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in soft clay for H/D<1.

$$S_{max} = 10^{-04} x P^2 + 0.317P + 39.10 \quad \ldots\ldots \quad \textbf{4-10}$$

Table 4.12: Foundation settlement for H/D=1 in soft clay for different Loads.

	Foundation settlement in soft clay (mm) for H/D=1				
	Loads (KN)				
Settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	6.59	232.64	463.51	704.31	1100
S2	13.01	231.54	461.48	704.02	1162
S3	19.17	244.15	477.02	727.93	1165
S4	25.39	258.23	500.31	761.88	1210

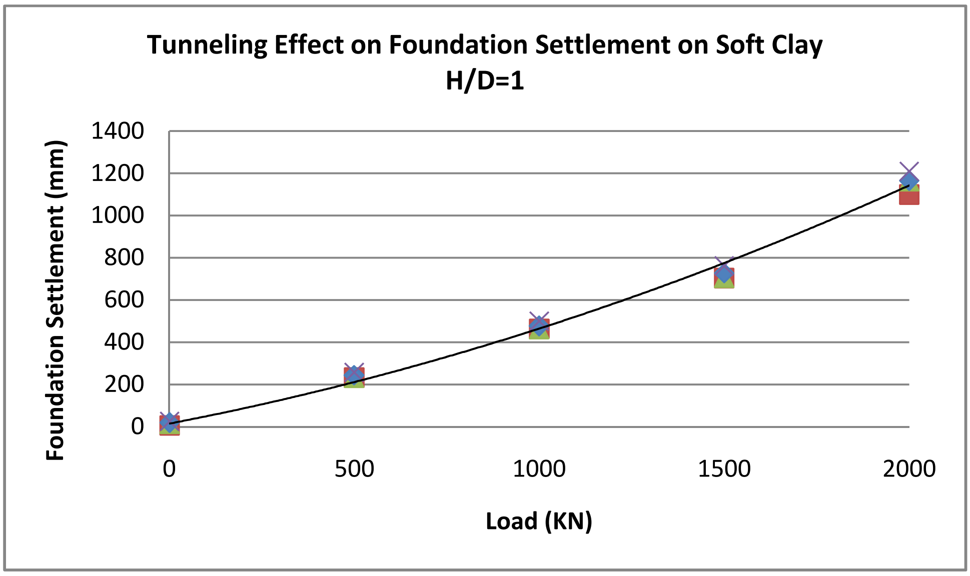

Figure 4.18: Foundation settlement for H/D=1 in soft clay for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in soft clay for H/D=1.

$S_{max} = 5x10^{-05}xP^2 + 0.324P + 29.97$ ……………………………….………4-11

Table 4.13: Foundation settlement for H/D>1 in soft clay for different Loads.

	Foundation settlement in soft clay (mm) for H/D>1				
	Loads (KN)				
Settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	1.29	19.76	46.68	69.31	93.8
S2	2.56	18.78	59.33	81.52	114.71

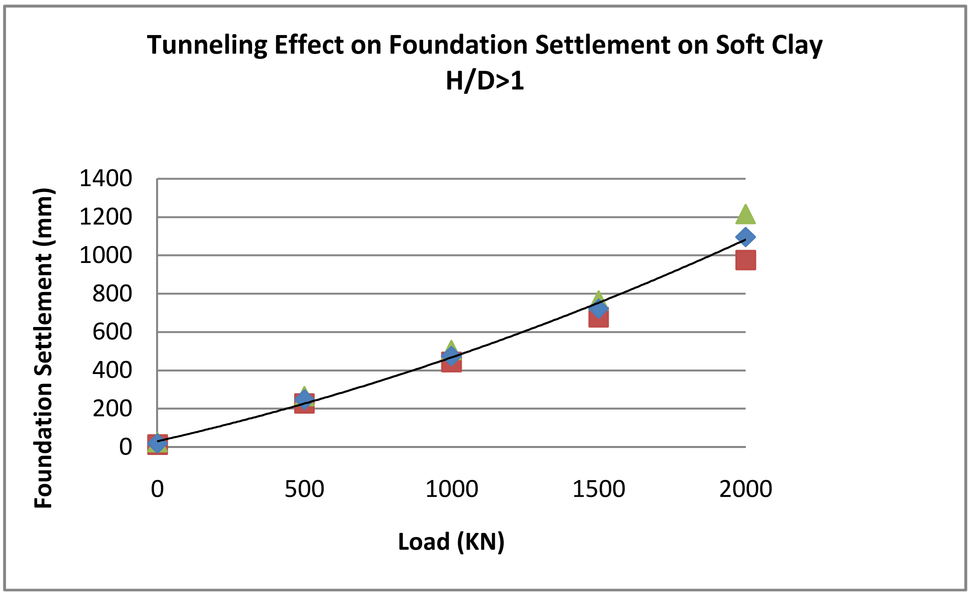

Figure 4.19: Foundation settlement for H/D>1 in soft clay for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in soft clay for H/D>1.

$$S_{max} = 7.5x10^{-05}xP^2 + 0.348P + 19.36 \quad \text{4-12}$$

Table 4.14: Foundation settlement for H/D<1 in medium clay for different Loads.

	Foundation settlement in medium clay (mm) for H/D<1				
	Loads (KN)				
Settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	6.35	100	197.09	294.31	391.51
S2	6.33	99.52	197.72	295.57	391.88
S3	12.66	103.57	201.29	298.65	395.95
S4	12.5	101.33	200.71	299.12	396.14
S5	12.58	102.67	201.05	298.92	396.7
S6	18.76	104.67	207.42	307.83	406.83
S7	25	109	216.65	318.31	418.46

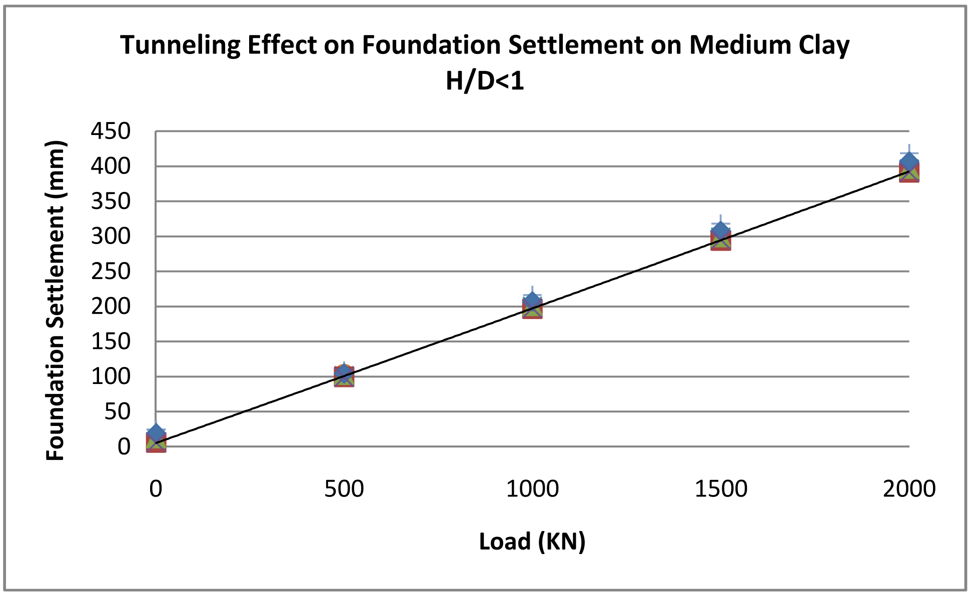

Figure 4.20: Foundation settlement for H/D<1 in medium clay for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in medium clay for H/D<1.

$$S_{max} = 4.5x10^{-06}xP^2 + 0.187P + 13.632 \quad \text{4-13}$$

Table 4.15: Foundation settlement for H/D=1 in medium clay for different Loads.

	Foundation settlement in medium clay (mm) for H/D=1				
	Loads (KN)				
Settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	6.38	99.48	195.47	287.58	387.13
S2	12.7	103.19	200.91	297.17	393.36
S3	18.92	110.66	213.5	314.03	408.83
S4	25.08	116.56	224.99	327.95	430.43

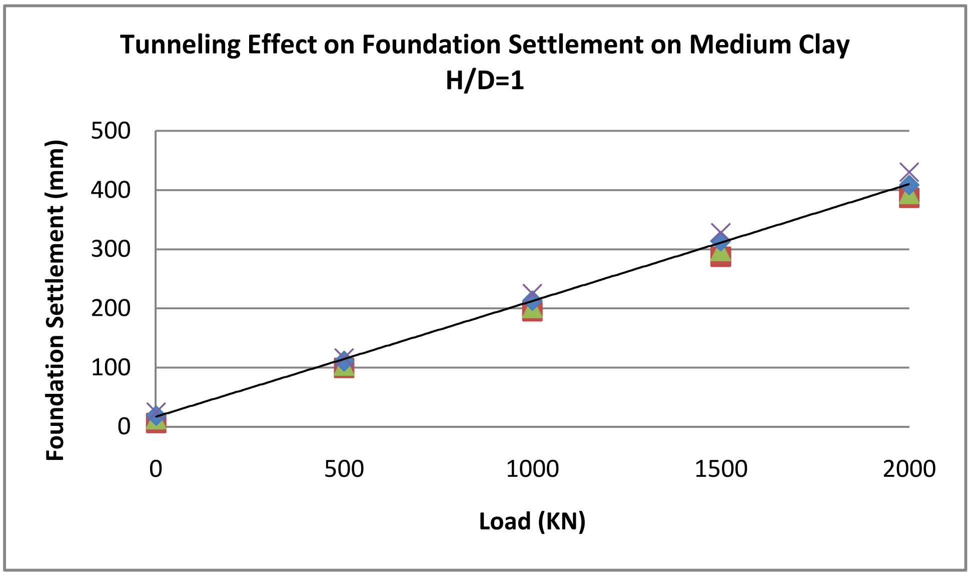

Figure 4.21: Foundation settlement for H/D=1 in medium clay for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in medium clay for H/D=1.

$$S_{max} = 3.5x10^{-06}xP^2 + 0.19P + 14.586 \quad \text{4-14}$$

Table 4.16: Foundation settlement for H/D>1 in medium clay for different Loads.

	Foundation settlement in medium clay (mm) for H/D>1				
	Loads (KN)				
Settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	12.76	104.99	198.95	292.61	386.18
S2	25.28	131.75	241.82	348.66	455.63

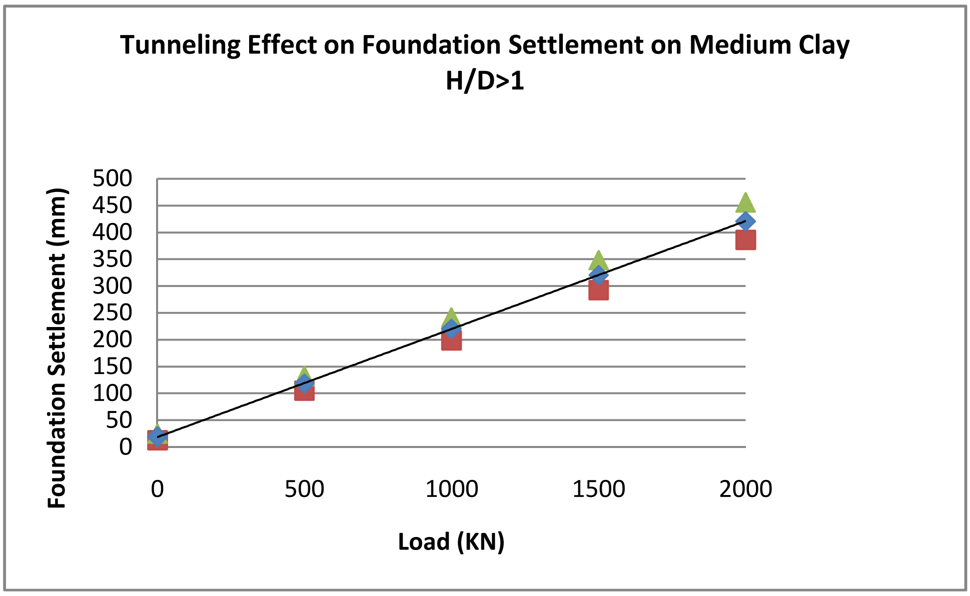

Figure 4.22 Foundation settlement for H/D>1 in medium clay for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in medium clay for H/D>1.

$$\mathbf{S_{max} = -10^{-07} x P^2 + 0.201P + 18.485} \quad \ldots\ldots\ldots\ldots \quad \mathbf{4\text{-}15}$$

Table 4.17: Foundation settlement for H/D<1 in hard clay for different Loads.

	Foundation settlement in hard clay (mm) for H/D<1				
	Loads (KN)				
Settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	2.59	45.08	88.1	130.99	171.34
S2	2.58	44.43	87.98	131.11	173.22
S3	4.9	45.05	89.4	133.09	178.41
S4	4.98	45.96	89.89	133.6	179.79
S5	5.06	47.15	90.36	134.06	180.72
S6	8.92	47.01	93.3	137.48	186.6
S7	12.94	49.68	97.94	143.07	187.54

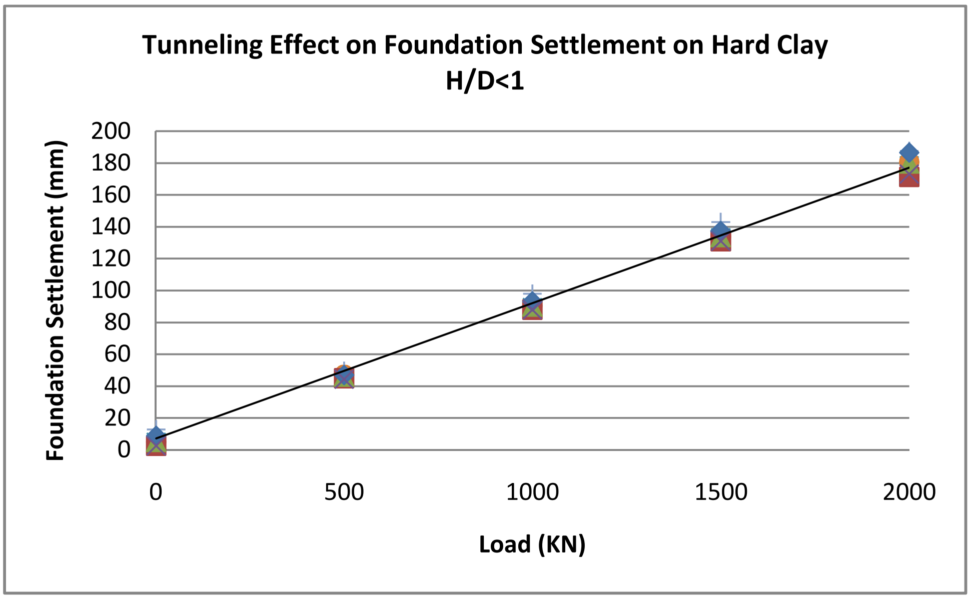

Figure 4.23 Foundation settlement for H/D<1 in hard clay for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in hard clay for H/D<1.

$$S_{max} = 2.99x10^{-06}xP^2 + 0.081P + 5.243 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 4\text{-}16$$

Table 4.18: Foundation settlement for H/D=1 in hard clay for different Loads.

	Foundation settlement in hard clay (mm) for H/D=1				
	Loads (KN)				
Settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	2.61	45.62	87.83	126.72	172.08
S2	5.09	47.8	90.41	138.89	182.8
S3	9.08	50.24	97.66	143.16	192.63
S4	13.02	53.67	102.12	148.51	194.38

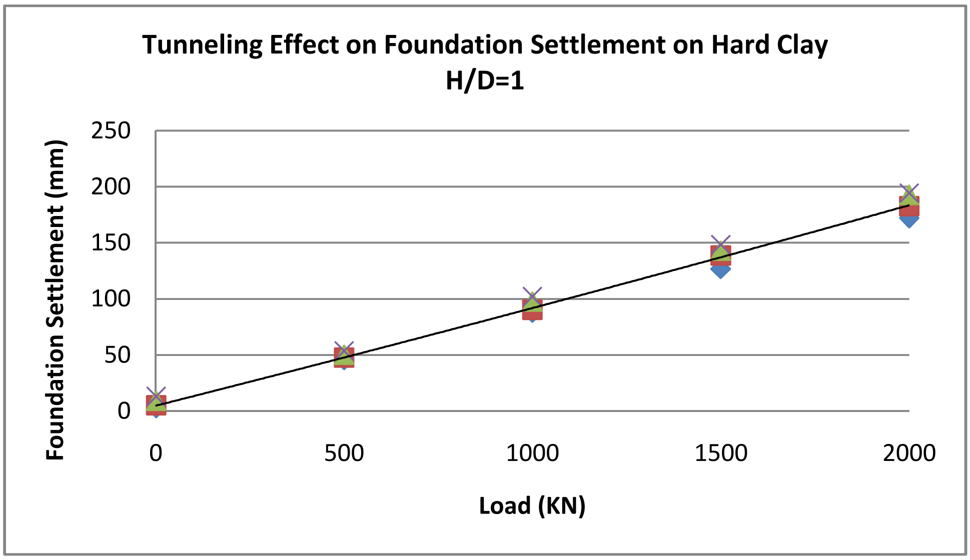

Figure 4.24: Foundation settlement for H/D=1 in hard clay for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in hard clay for H/D=1.

$$S_{max} = 1.15x10^{-06}xP^{2} + 0.0845P + 7.514 \quad \text{4-17}$$

Table 4.19: Foundation settlement for H/D>1 in hard clay for different Loads.

	Foundation settlement in hard clay (mm) for H/D>1				
	Loads (KN)				
Settlement	0KN	**500 KN**	**1000 KN**	**1500 KN**	**2000 KN**
S1	6.63	48.34	90.45	141.87	185.54
S2	13.13	55.51	108.28	159.72	206.71

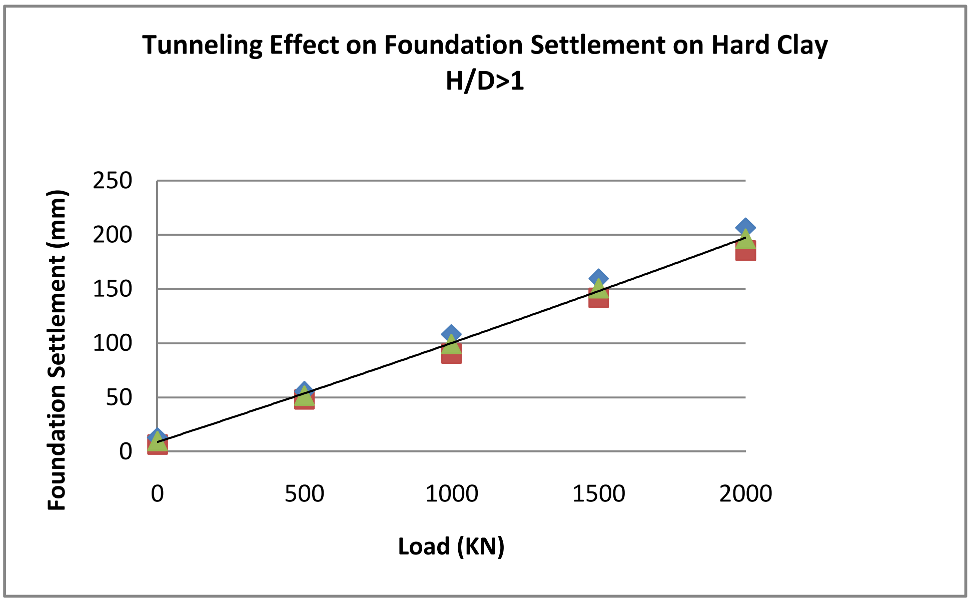

Figure 4.25: Foundation settlement for H/D>1 in hard clay for different Loads

.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in hard clay for H/D>1.

$$S_{max} = 3x10^{-06}xP^2 + 0.0875P + 8.771 \quad \text{...4-18}$$

In general analysis for tunnel with different depth and different diameter by PLAXIS due to more loads can be summarized in six equations (4-19 to 4-24) These equations can be used to predict settlement of foundation due to tunneling with high accuracy for different type of soils. Knowing the axial load on foundation setting over a tunnel, settlement can be estimated as shown Figure 4.26 to Figure4.31.

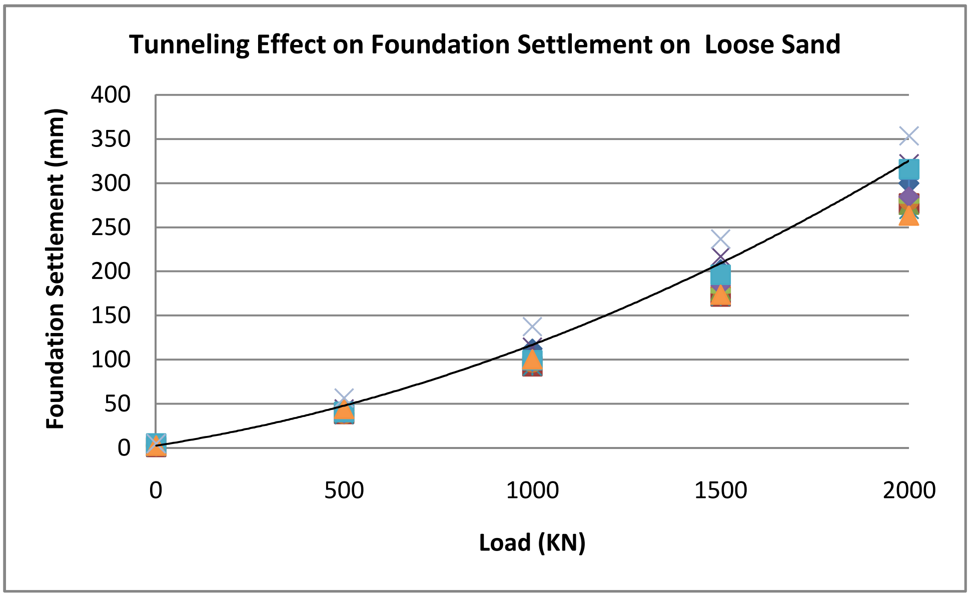

Figure 4.26: Foundation settlement in Loose Sand for different Loads

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in loose sand.

$$S_{max} = 3.5x10^{-05}xP^{2} + 0.078P + 3.268 \ldots\ldots \quad \textbf{4-19}$$

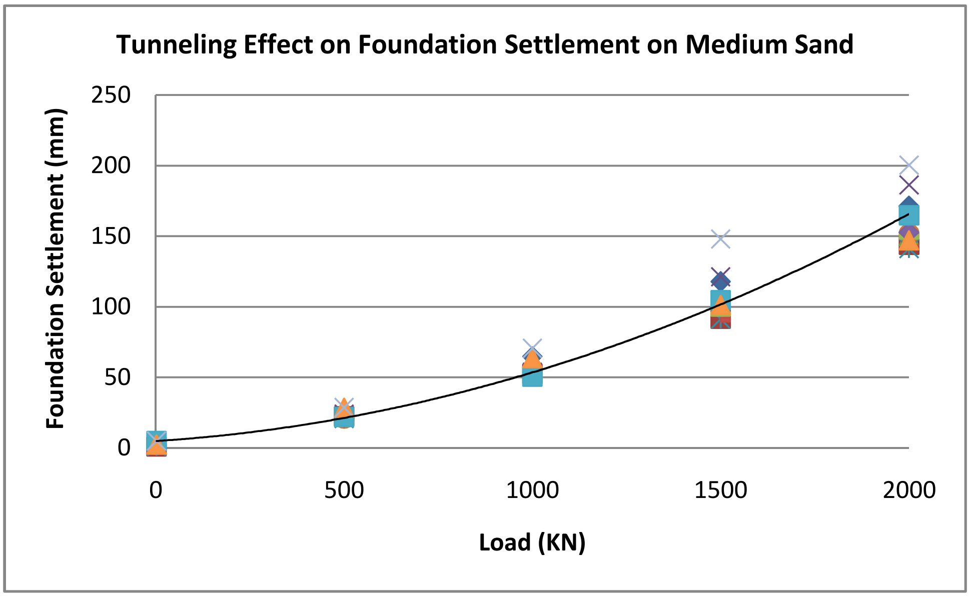

Figure 4.27: Foundation settlement in Medium Sand for different Loads

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in medium sand.

$$S_{max} = 2.5x10^{-05}xP^2 + 0.0395P + 1.434 \ldots\ldots\ldots\ldots \quad 4\text{-}20$$

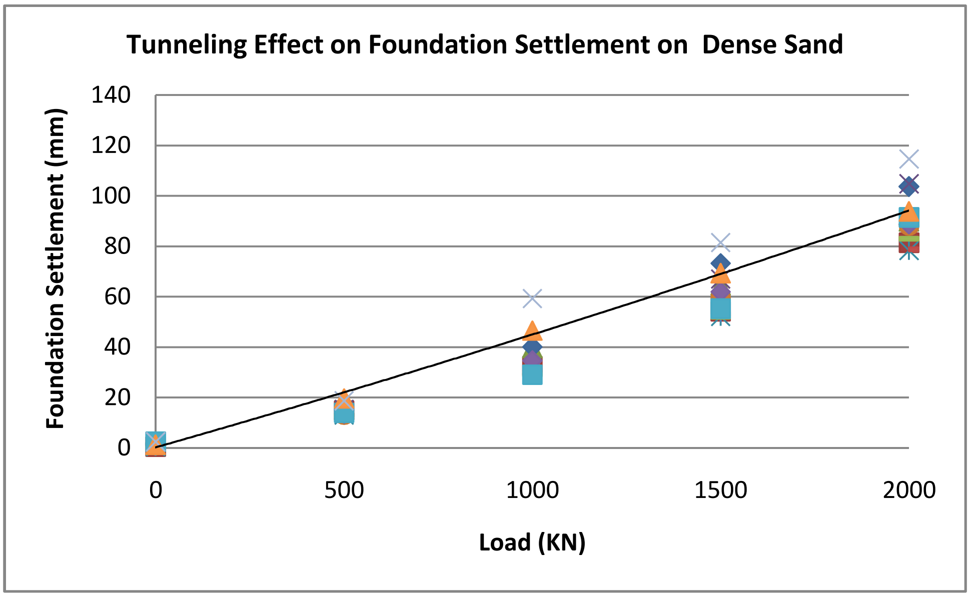

Figure 4.28: Foundation settlement in Dense Sand for different Loads.

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in dense sand.

$$S_{max} = 6.5x10^{-06}xP^2 + 0.0345P + 0.361 \ldots \quad 4\text{-}21$$

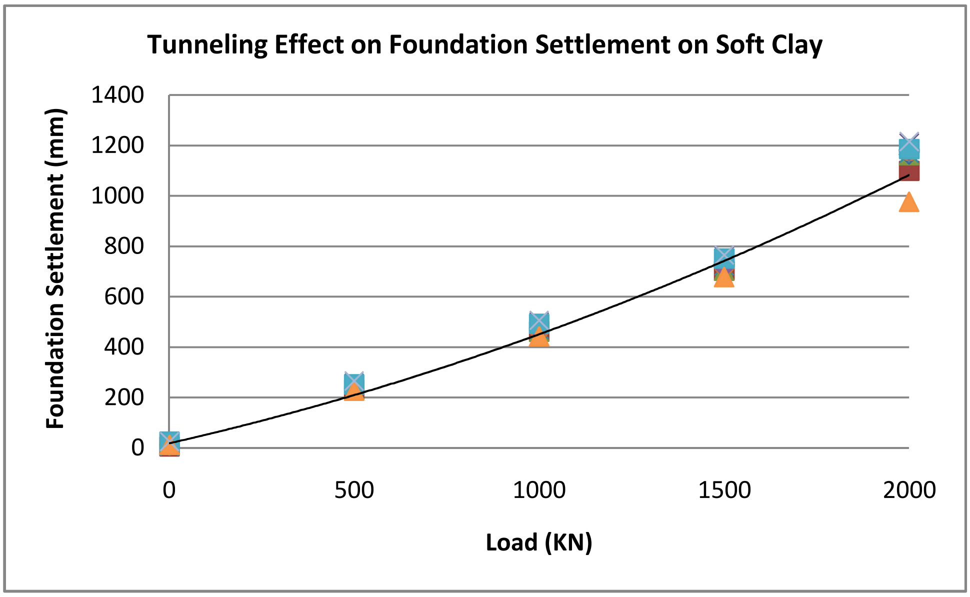

Figure 4.29 Foundation settlement in soft clay for different Loads

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in soft clay.

$$S_{max} = 7.5x10^{-05}xP^2 + 0.348P + 30.475 \qquad \text{4-22}$$

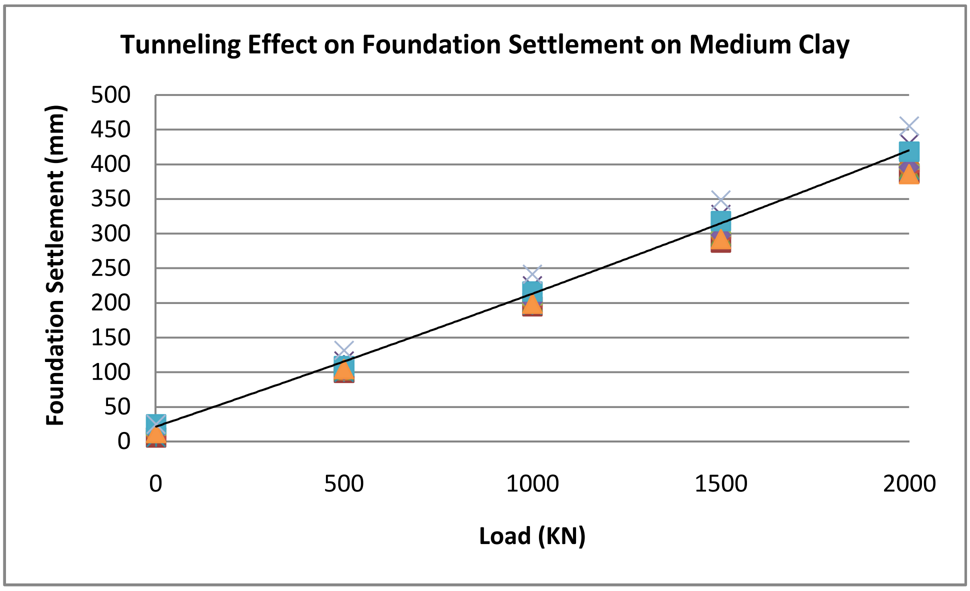

Figure 4.30 Foundation settlement in medium clay for different Loads

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in medium clay.

$$S_{max} = -10^{-07} x P^2 + 0.201x + 18.485 \qquad \text{4-23}$$

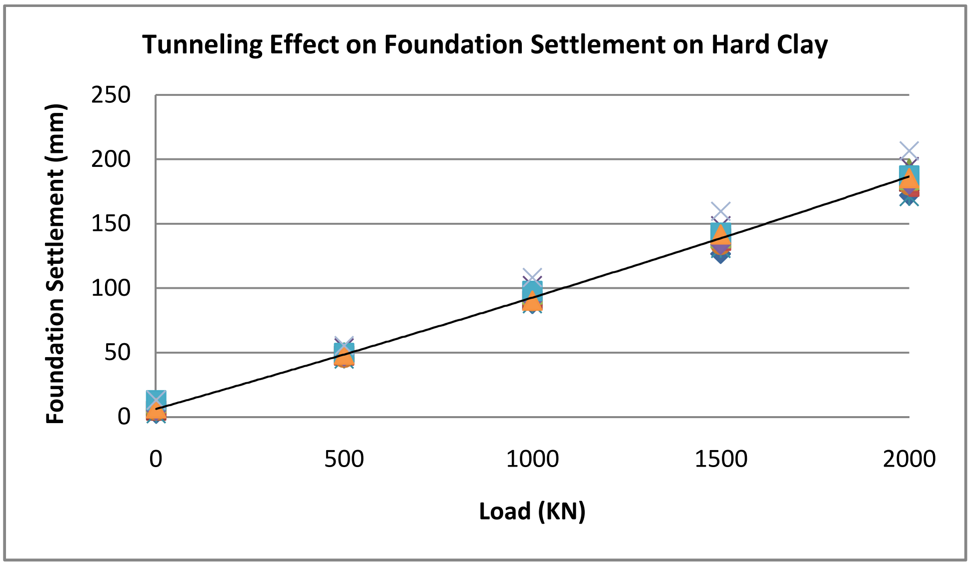

Figure 4.31: Foundation settlement in hard clay for different Loads

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in hard clay.

$$S_{max} = 0.5x10^{-06}xP^2 + 0.09P + 6.83 \quad \text{4-24}$$

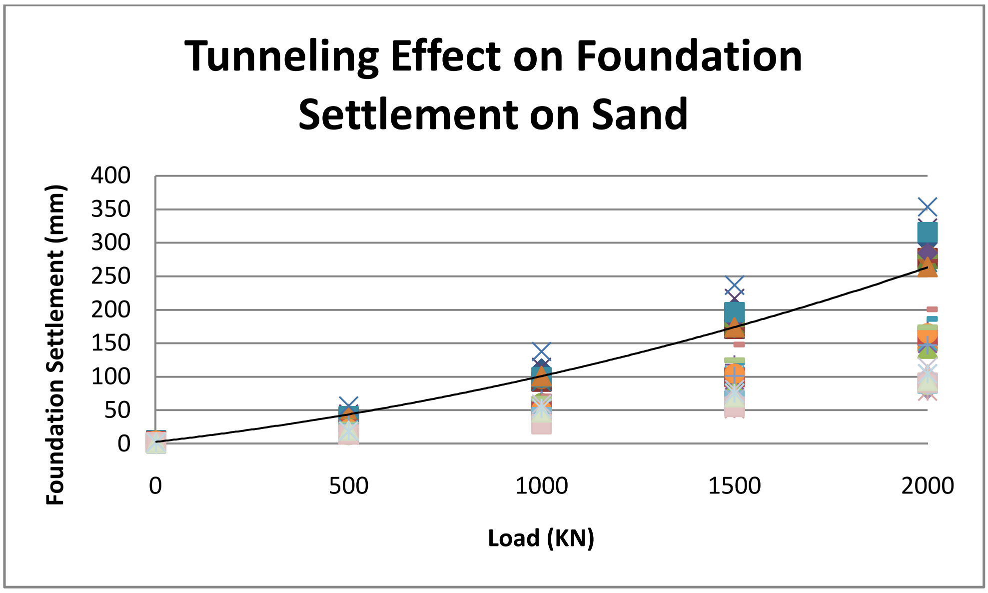

Figure 4.32: Foundation settlement in sand for different Loads

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in sand.

$$S_{max} = 2.45x10^{-05}xP^2 + 0.056P + 3.02 \ldots\ldots\ldots\ldots \quad 4\text{-}25$$

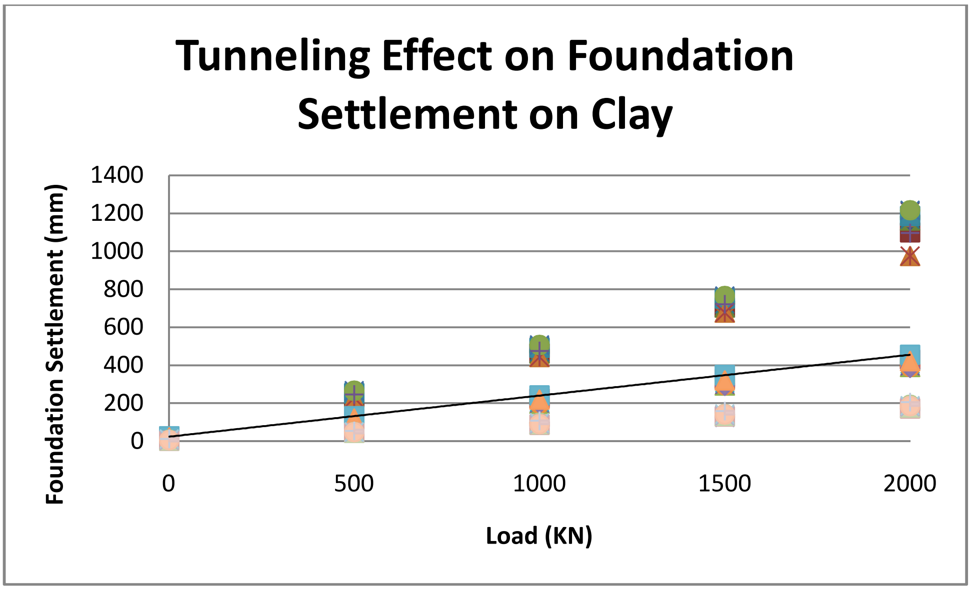

Figure 4.33: Foundation settlement in clay for different Loads

From fit line which show the relationship between the concentrated force and maximum settlement of foundation in clay.

$$S_{max} = -8x10^{-07}xP^2 + 0.217P + 24.594 \ldots\ldots\ldots\ldots \quad 4\text{-}26$$

4.2 Comparison Between In Situ Measured Values and Finite Element (PLAXIS)

To validate the numerical analysis, a comparison will be done between the results obtained by Shiraz metro field data as shown in Figure 4.34 where geological profile shown in Figure 4.35. Two dimensional analysis studies are done using PLAXIS software to evaluate the settlement of foundation due to tunnel , many factors affect settlement calculation such as buildings weight, tunnels depth, tunnels diameters, and type of soil. The ground water is not considered in this study. by using the results from numerical simulations for various type of soil and different depth and diameter of tunnels, six equations are developed and suggested for predicting maximum foundation settlements and green-field conditions to predict settlement in design stage.

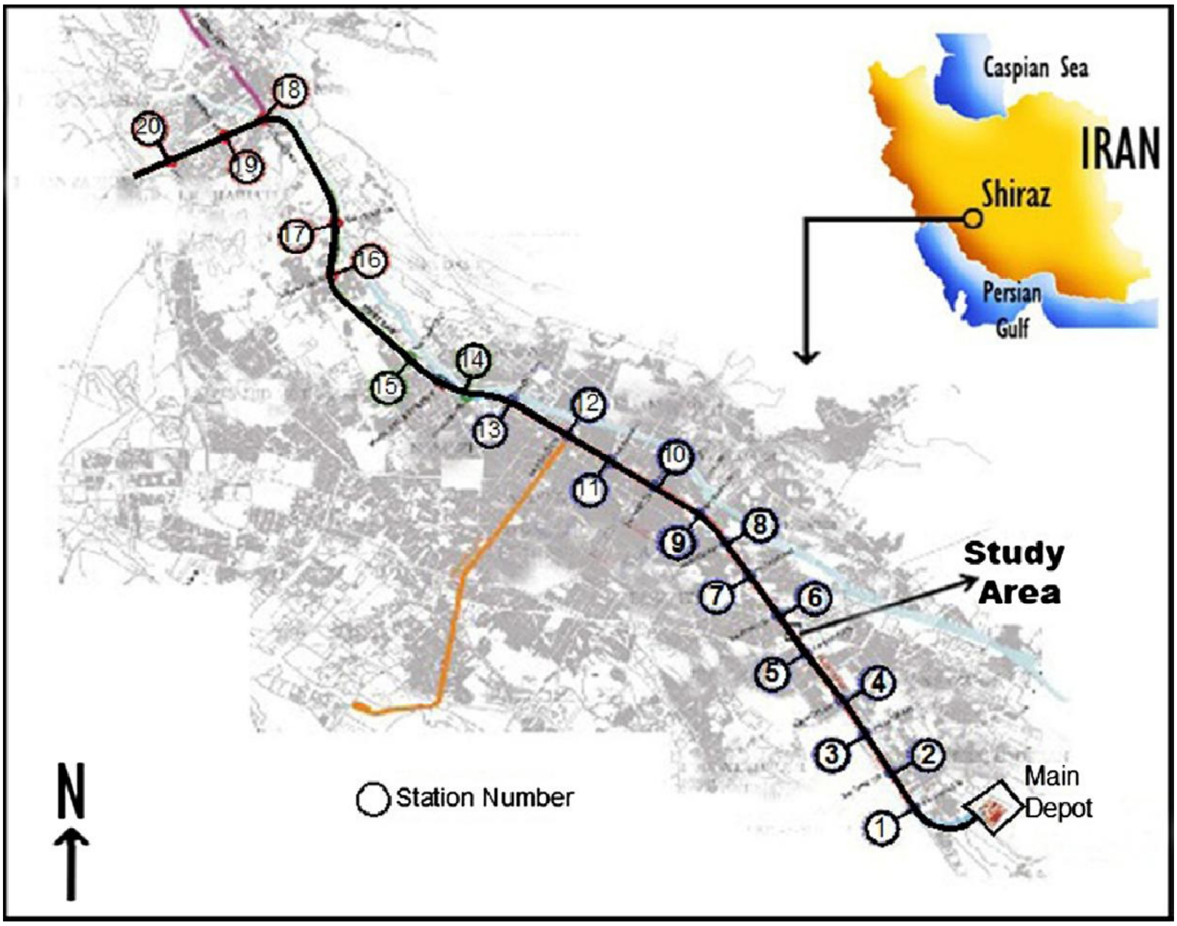

Figure 4.34: General layout of Shiraz metro line 1

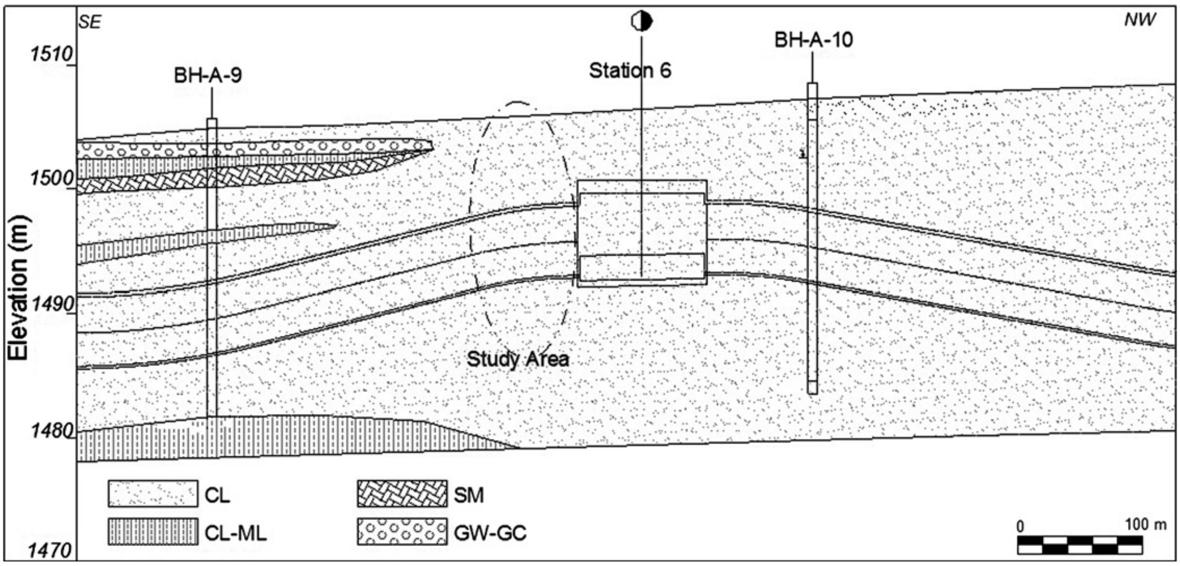

Figure 4.35: Geological profile of Shiraz metro line 1 in the study area (SURO, 2003).

4.2.1 Characteristics of Shiraz Metro Line1

In Shiraz three metro routes of which line 1.15 km length of this line was studied. This part consist of twin tunnels where constructed using two TBMs each with diameter of 6.9 m. thickness of tunnels, was 30 cm precast concrete. Horizontal distances between centerlines of the tunnels range between 13 m to 17 m with varying depth up to 23 m. Soil properties are shown in Table 4.20; which categorize as medium clay and the soil deposit is assumed to be homogenous and isotropic soil types along the route. Ground water neglected. Measurements of surface settlement at the control points began 3 days before TBMs arrival and after a month after passing of the machines. No monitoring tools were installed beneath the adjacent buildings.

Table 4.20: Material Properties of Soil.

	Clayey soil (CL)
Unit weight, γ_{dry}, (kN/m3)	17
Saturated unit weight, γ_{Sat}, (kN/m^3)	20.7
Total cohesion, Cu, (kN/m^2)	100
Effective cohesion, C' (kN/m^2)	10
Total friction angle, Øu (°)	0
Effective friction angle, Ø' (°)	30
Young's modulus, E, (MPa)	20
Poisson's ratio, ν	0.25

Figure 4.36 shows the measured data results for the settlement where the maximum settlement 19 mm.

For numerical analysis soil properties for line 1 of Shiraz metro classified as medium clay as shown in Table 3.1. For medium clay settlement of foundation can be estimated from equation 4-23, so for Greenfield settlement value of foundation is 18.48mm

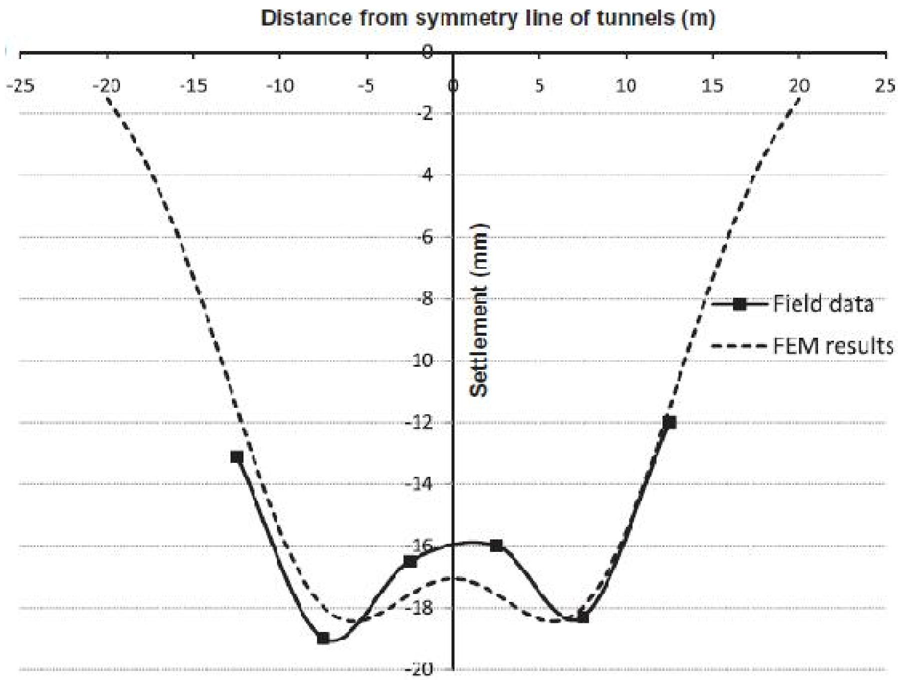

Figure 4.36: Measured data results for the settlement.

The results of this research compared very well with Shiraz metro measurement by A. Mirhabibi, A. Soroush, (2012). The results present here for medium clay show maximum settlement of 18.5 mm while the measured settlement by Shiraz meter case was 19 mm. This is clear indication of the validity of the results presented in this research using numerical methods (PLAXIS code).

CHAPTER 5

"Conclusions & Recommendations "

5.1 Conclusions

The main objective of this thesis is to evaluate settlement of foundations due to tunneling. Investigating the effect of tunnels on various structures and infrastructural components such as depth and various diameters under the structures. Different variables were considered such as type of soils (sand and clay), tunnels depth and diameters. Settlement has been calculated for different type of soils ranging from soft clay to dense sand. Tunnels with different diameters were investigated at different depth. Load was a factor and it has been changed along the analysis. The settlement was calculated using numerical solution by using PLAXIS code. Assumptions used in this study for soils such as soil is homogenous, isotropic and classical Mohr–Coulomb failure criteria is valid.

A Comparison between field data of the Shiraz metro line 1 and two dimensional numerical models (PLAXIS) were studied to verify the results of the numerical model.

From thesis the engineers will be able to predict the effect of tunnel on building. Predictions of maximum settlements of foundation for green-field conditions and different loads due to tunneling during preliminary design phases will be possible.

A parametric study was carried out using a finite element method via the well established program PLAXIS, which is intended for the analysis of deformation and stability in geotechnical engineering projects The parametric study revealed the following conclusions:

- In general the existing of tunnels under foundation will increase the settlement compared to the green-field condition.
- Tunnel diameter is a major geometrical parameter which increase the effect of settlement.
- Loads on foundation must be considered in simulation to assure reliable results were with more loads the settlement will increase.
- Soil type is another important factor which has significant effects on the tunneling–building interaction behavior.
- The increase of tunnel depth, surface distance of foundation from the upper face of tunnels decrease the effect of settlement.

- Increase of tunnel depth, decrease of tunnels diameter, reduction loads from building and soil stiffness decrease the effect of buildings on settlement curve.
- Six equations was developed for predicting the maximum settlements of foundation to use in preliminary design stage.
- General two equations are developed for sand and clay to predict maximum settlement under different load as an average values.
- For sand settlement of foundation range from 0.60mm to 5.12 mm in Greenfield, but in clay settlement of foundation range from 11.4mm to 42.3mm.
- In soft clay it noted that value of maximum settlement very high specially with increase of axial load on foundation.
- The ratio of depth of tunnel to diameter for various type of soil has an affect on settlement of foundation .

5.2 Recommendations

At this research developed equations will use to predictions of maximum settlements of foundation for green-field conditions and different loads due to tunneling during preliminary design phases. Before use these equations it must to understand the assumptions and all of various parameters which affect in choice which of equation use and then results. In general equations was developed to predict maximum settlements of foundation by known soil type, depth of tunnel to diameter of tunnel and the concentrated load which concentrated on foundation with 10m width which mean that the stress will be conceder before use the developed equations. According to the results in this research:

- To validate numerical calculations was need more field data or experimental test to satisfy the accuracy of developed equations.
- Volume loss must consider from field test and verify the actual value of volume loss.
- Geometry and stiffness of Building is an important factor that effects foundation settlement should be investigated.
- Differential settlement of foundation should be considered in future research
- Design charts should be developed in future research .

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Appendix A

Table A.1 Foundation settlement in Greenfield condition in soft clay for different tunnel depth and diameter.

Foundation settlement in Soft Clay (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	6.59	6.51	6.47	6.42	6.4	6.36
10	13.13	13.01	12.92	12.84	12.8	12.72
15	19.39	19.27	19.17	19.09	19.05	19.01
20	25.71	25.59	25.47	25.39	25.35	25.31

Table A.2 Foundation settlement in Greenfield condition in medium clay for different tunnel depth and diameter.

Foundation settlement in Medium Clay (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	6.38	6.35	6.33	6.32	6.3	6.29
10	12.76	12.7	12.66	12.58	12.54	12.5
15	19.14	19.02	18.92	18.84	18.8	18.76
20	25.4	25.28	25.16	25.08	25.04	25

Table A.3 Foundation settlement in Greenfield condition in hard clay for different tunnel depth and diameter.

Foundation settlement in Hard Clay (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	2.61	2.59	2.58	2.57	2.57	2.56
10	6.63	5.09	5.06	4.98	4.94	4.9
15	10.65	9.11	9.08	9	8.96	8.92
20	14.67	13.13	13.1	13.02	12.98	12.94

Table A.4 Foundation settlement in Greenfield condition in loose sand for different tunnel depth and diameter.

Foundation settlement in Loose Sand (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	1.34	1.33	1.32	1.32	1.31	1.31
10	2.6	2.58	2.56	2.56	2.55	2.55
15	3.88	3.86	3.84	3.84	3.83	3.83
20	5.16	5.14	5.12	5.12	5.11	5.11

Table A.5 Foundation settlement in Greenfield condition in medium sand for different tunnel depth and diameter.

Foundation settlement in Medium Sand (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	1.3	1.29	1.29	1.28	1.28	1.28
10	2.56	2.54	2.53	2.52	2.52	2.52
15	3.82	3.8	3.79	3.78	3.78	3.78
20	5.08	5.06	5.05	5.04	5.04	5.04

Table A.6 Foundation settlement in Greenfield condition in dense sand for different tunnel depth and diameter.

Foundation settlement in Dense Sand (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	0.65	0.64	0.64	0.64	0.64	0.64
10	1.29	1.28	1.27	1.27	1.27	1.27
15	1.93	1.92	1.91	1.91	1.91	1.91
20	2.57	2.56	2.55	2.55	2.55	2.55

Table A.7 Foundation settlement due to concentrated load 500 KN in soft clay for different tunnel depth and diameter.

Foundation settlement in Soft Clay (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	232.64	234.83	234.85	236.24	236.44	236.48
10	227.8	231.54	236.55	237.19	237.79	237.81
15	241.58	244.29	244.15	244.52	243.96	243.39
20	269.22	266.45	262.35	258.23	255.41	252.71

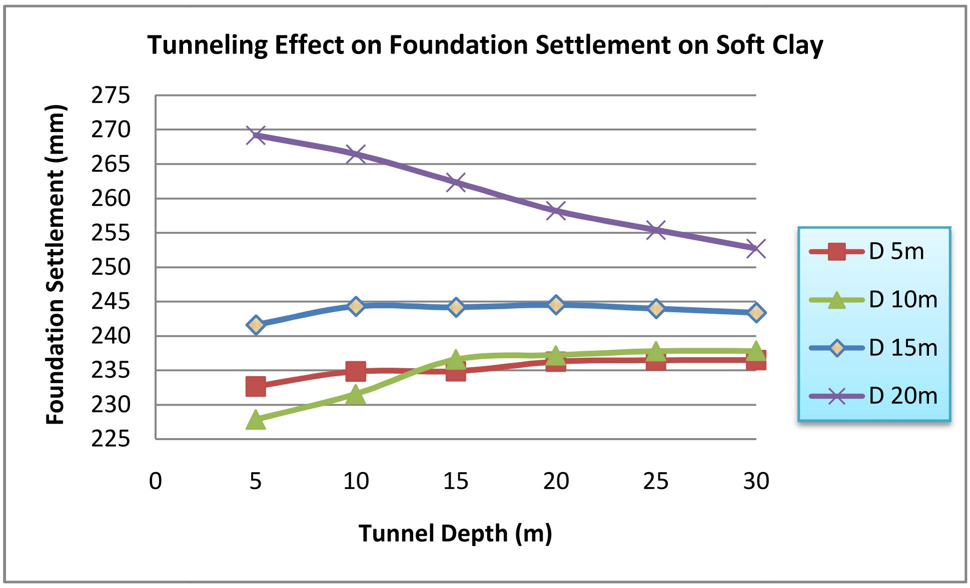

Figure A.1 Foundation settlement due to concentrated load 500 KN in soft clay for different tunnel depth and diameter.

Table A.8 Foundation settlement due to concentrated load 500 KN in medium clay for different tunnel depth and diameter.

Foundation settlement in Medium Clay (mm)						
Depth (m) Diameter (m)	5	10	15	20	25	30
5	99.48	100	99.52	100	99.68	99.54
10	104.99	103.19	103.57	102.67	101.96	101.33
15	120.53	114.69	110.66	108.16	106.11	104.67
20	144.43	131.75	122.87	116.56	112	109

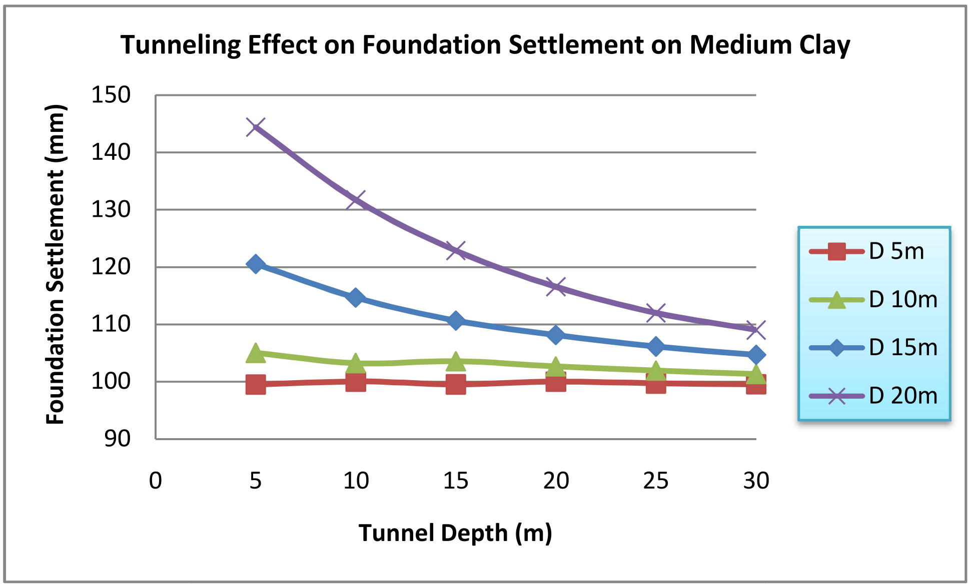

Figure A.2 Foundation settlement due to concentrated load 500 KN in medium clay for different tunnel depth and diameter.

Table A.9 Foundation settlement due to concentrated load 500 KN in hard clay for different tunnel depth and diameter.

Foundation settlement in Hard Clay (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	45.62	45.08	44.43	44.22	44.03	43.9
10	48.34	47.8	47.15	45.96	45.4	45.05
15	51.43	50.89	50.24	49.05	47.77	47.01
20	56.05	55.51	54.86	53.67	51.19	49.68

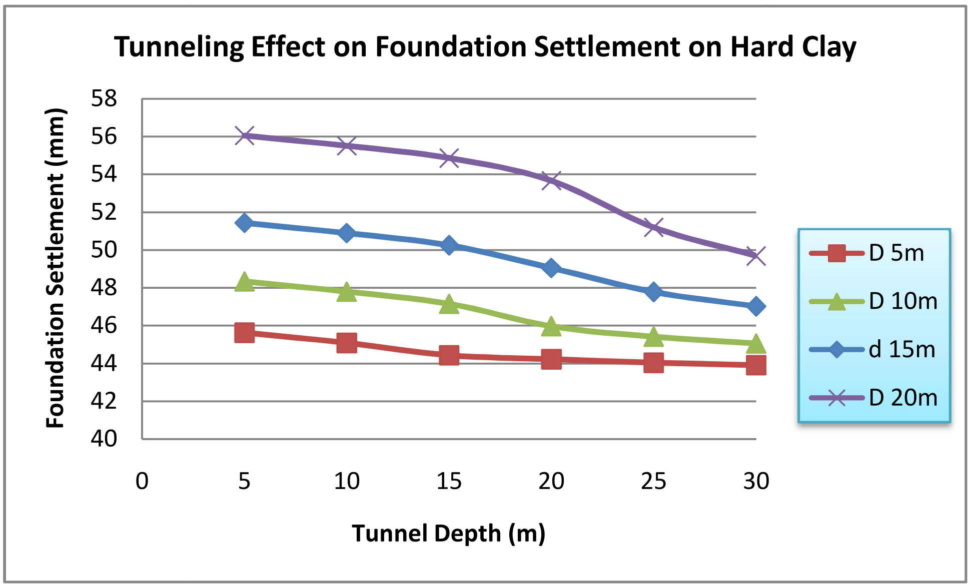

Figure A.3 Foundation settlement due to concentrated load 500 KN in hard clay for different tunnel depth and diameter.

Table A.10 Foundation settlement due to concentrated load 500 KN in loose sand for different tunnel depth and diameter.

Foundation settlement in Loose Sand (mm)						
Depth (m) / Diameter (m)	5	10	15	20	25	30
5	38.63	38.05	37.89	37.67	37.51	37.54
10	44.52	40.87	39.56	38.8	38.32	38.11
15	55.49	46.68	42.77	41.03	39.98	39.29
20	65.49	56.68	52.77	44.38	42	41.03

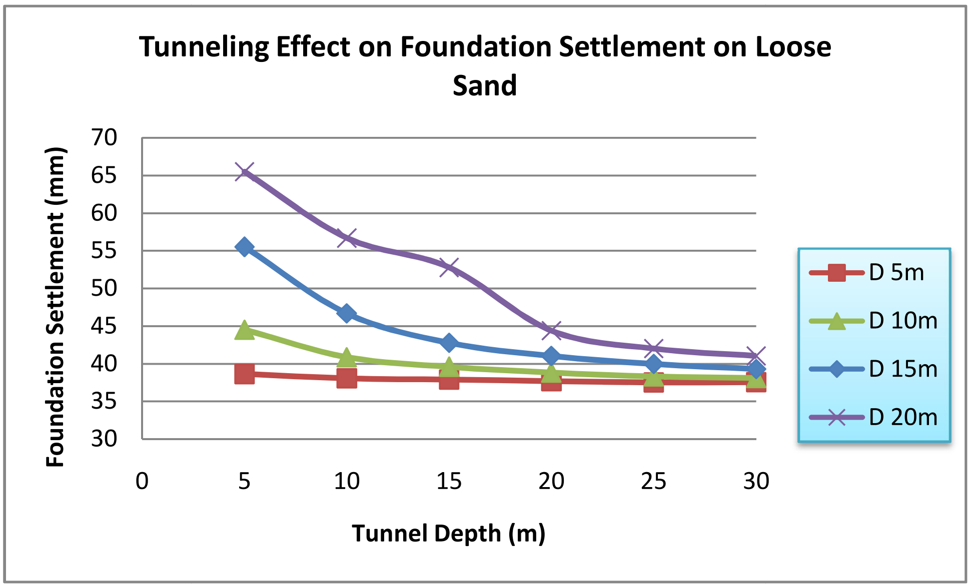

Figure A.4 Foundation settlement due to concentrated load 500 KN in loose sand for different tunnel depth and diameter.

Table A.11 Foundation settlement due to concentrated load 500 KN in mediumsand for different tunnel depth and diameter.

Foundation settlement in Medium Sand (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	22.61	21.03	20.7	20.37	20.25	20.29
10	28.69	23.69	21.71	21.02	20.74	20.59
15	36.55	26.96	23.52	22.33	21.7	21.31
20	38.36	28.77	25.33	24.14	22.95	22.18

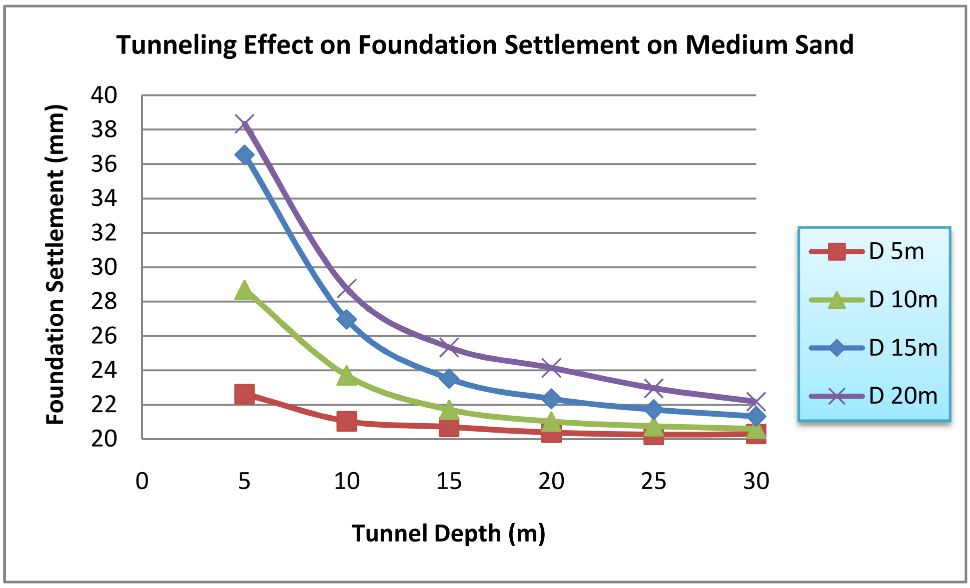

Figure A.5 Foundation settlement due to concentrated load 500 KN in medium sand for different tunnel depth and diameter.

Table A.12 Foundation settlement due to concentrated load 500 KN in dense sand for different tunnel depth and diameter.

Foundation settlement in Dense Sand (mm)						
Depth (m) / **Diameter (m)**	5	10	15	20	25	30
5	14.89	13.19	13.06	12.73	12.69	12.68
10	19.76	15.42	13.75	13.29	13.05	12.94
15	26	17.1	14.99	14.9	13.67	13.39
20	32.24	18.78	16.23	15.36	14.51	14

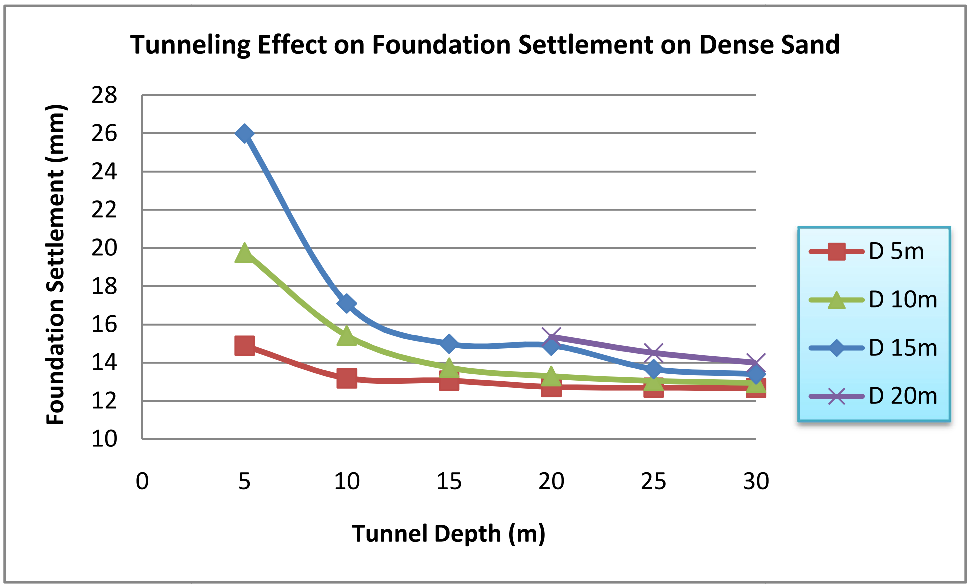

Figure A.6 Foundation settlement due to concentrated load 500 KN in dense sand for different tunnel depth and diameter.

Table A.13 Foundation settlement due to concentrated load 1000 KN in soft clay for different tunnel depth and diameter.

Foundation settlement in Soft Clay (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	463.51	470.25	472.8	474.55	474.93	475.06
10	443.79	461.48	468.4	472.16	474.32	474.48
15	461.98	472.86	477.02	479.33	479.21	479.45
20	504.16	506.91	504.47	500.31	496.87	493.09

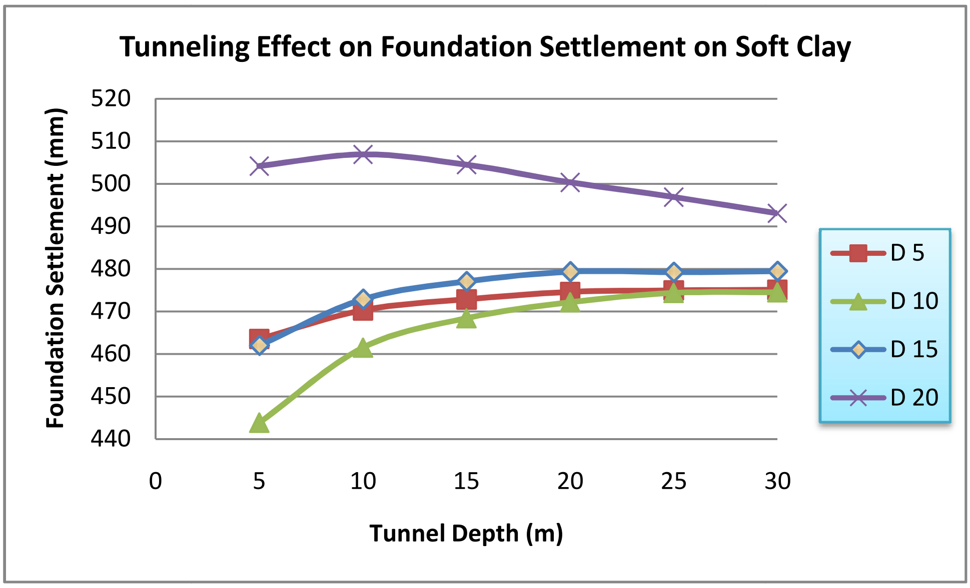

Figure A.7 Foundation settlement due to concentrated load 1000 KN in soft clay for different tunnel depth and diameter.

Table A.14 Foundation settlement due to concentrated load 1000 KN in medium clay for different tunnel depth and diameter.

Foundation settlement in Medium Clay (mm)						
Depth (m) / **Diameter (m)**	5	10	15	20	25	30
5	195.47	197.09	197.72	198.07	197.87	198.21
10	198.95	200.91	201.29	201.05	200.92	200.71
15	223.23	217.99	213.5	210.87	208.79	207.42
20	256.26	241.82	231.92	224.99	220.38	216.65

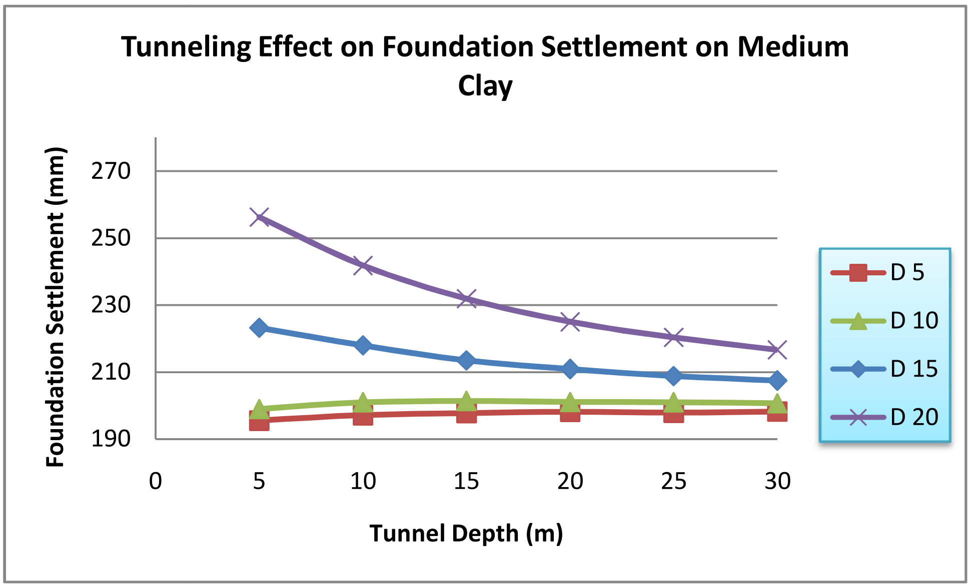

Figure A.8 Foundation settlement due to concentrated load 1000 KN in medium clay for different tunnel depth and diameter.

Table A.15 Foundation settlement due to concentrated load 1000 KN in hard clay for different tunnel depth and diameter.

Foundation settlement in Hard Clay (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	87.83	88.1	87.98	88.03	87.92	87.78
10	90.45	90.41	90.36	89.89	89.64	89.4
15	108.7	101.65	97.66	95.49	94.16	93.3
20	115.33	108.28	104.29	102.12	99.59	97.94

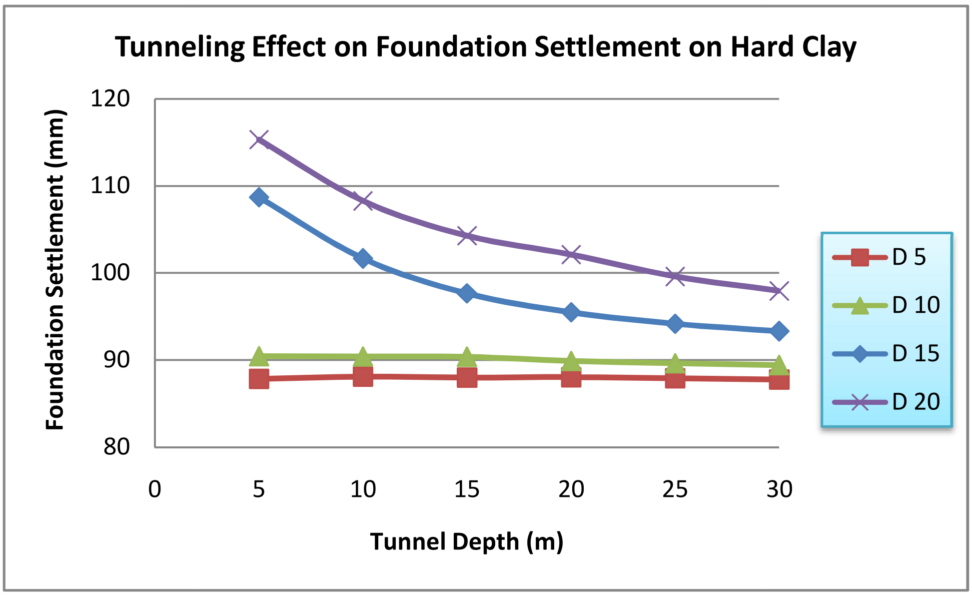

Figure A.9 Foundation settlement due to concentrated load 1000 KN in hard clay for different tunnel depth and diameter.

Table A.16 Foundation settlement due to concentrated load 1000 KN in loose sand for different tunnel depth and diameter.

Foundation settlement in Loose Sand (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	92.8	92.74	93.56	93.23	92.97	93.17
10	100.75	100.95	100.03	96.92	95.01	94.69
15	122.38	119.98	112.5	105.95	98.32	97.06
20	146	137.76	125.97	114.24	105	99.48

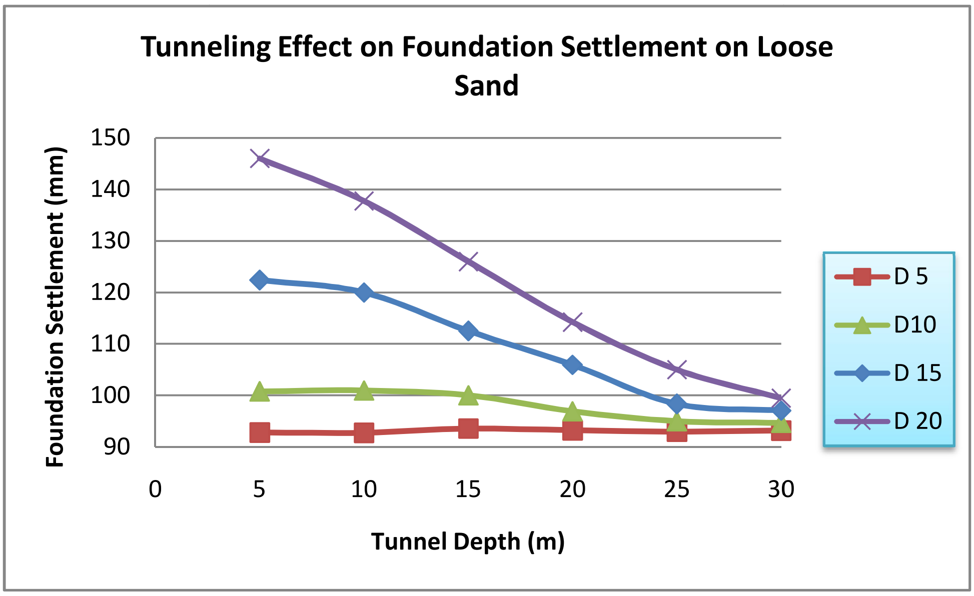

Figure A.10 Foundation settlement due to concentrated load 1000 KN in loose sand for different tunnel depth and diameter.

Table A.17 Foundation settlement due to concentrated load 1000 KN in mediumsand for different tunnel depth and diameter.

Foundation settlement in Medium Sand (mm)						
Depth (m) / **Diameter (m)**	5	10	15	20	25	30
5	52.63	51.64	52.23	50.95	50.55	50.56
10	63.88	60.71	57.16	53.13	51.29	50.97
15	80.59	73.98	64.97	57.06	52.95	51.96
20	77.37	70.76	61.75	53.84	52.2	50.68

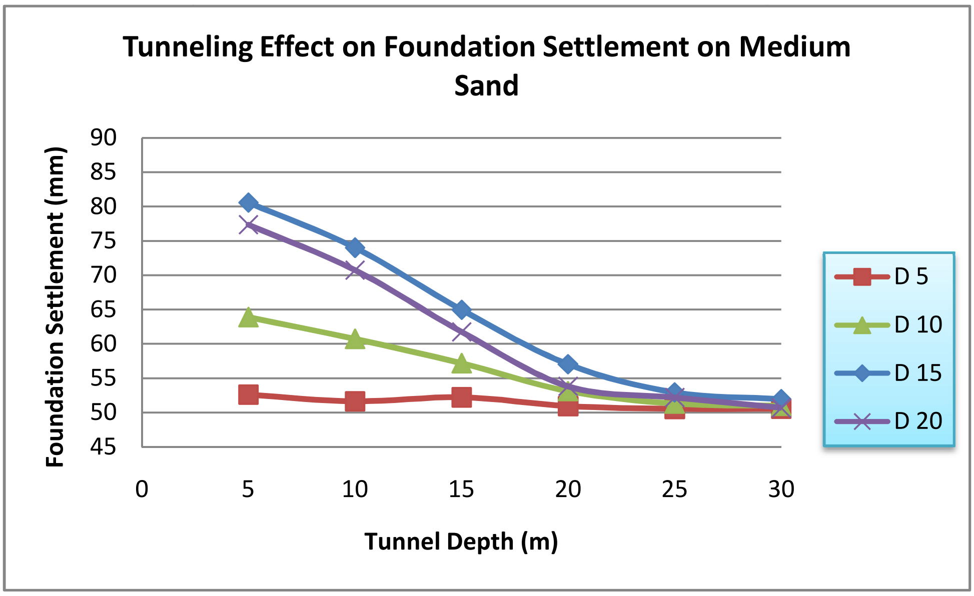

Figure A.11 Foundation settlement due to concentrated load 1000 KN in medium sand for different tunnel depth and diameter.

Table A.18 Foundation settlement due to concentrated load 1000 KN in dense sand for different tunnel depth and diameter.

Foundation settlement in Dense Sand (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	32.54	31.3	30.8	29.33	29.11	28.89
10	46.68	39.67	34.67	30.65	29.45	29.11
15	58.72	49.53	40.1	33.4	30.84	29.97
20	68.76	59.33	32.33	31.25	30.17	29.17

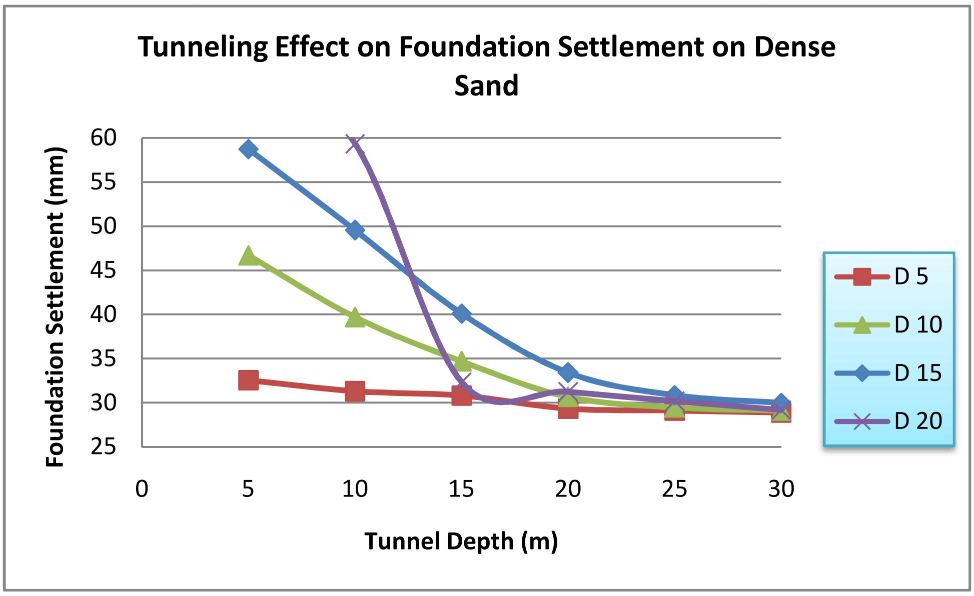

Figure A.12 Foundation settlement due to concentrated load 1000 KN in dense sand for different tunnel depth and diameter.

Table A.19 Foundation settlement due to concentrated load 1500 KN in soft clay for different tunnel depth and diameter.

Foundation settlement in Soft Clay (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	704.31	723	728.84	732.34	734.24	732.94
10	678.01	704.02	717.53	725.44	729.79	729.6
15	697.15	717.44	727.93	732.46	731.83	735
20	754.98	765.76	764.01	761.88	754.03	752.02

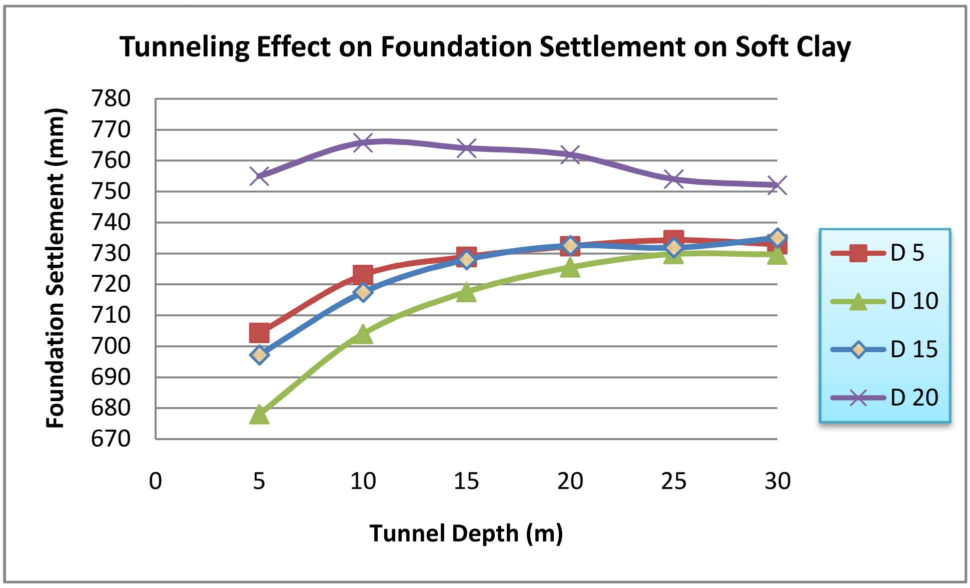

Figure A.13 Foundation settlement due to concentrated load 1000 KN in soft clay for different tunnel depth and diameter.

Table A.20 Foundation settlement due to concentrated load 1500 KN in medium clay for different tunnel depth and diameter.

Foundation settlement in Medium Clay (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	287.58	294.31	295.57	296.28	296.58	296.71
10	292.61	297.17	298.65	298.92	299.14	299.12
15	323.66	318.69	314.03	311.45	309.26	307.83
20	366.24	348.66	336.49	327.95	322.45	318.31

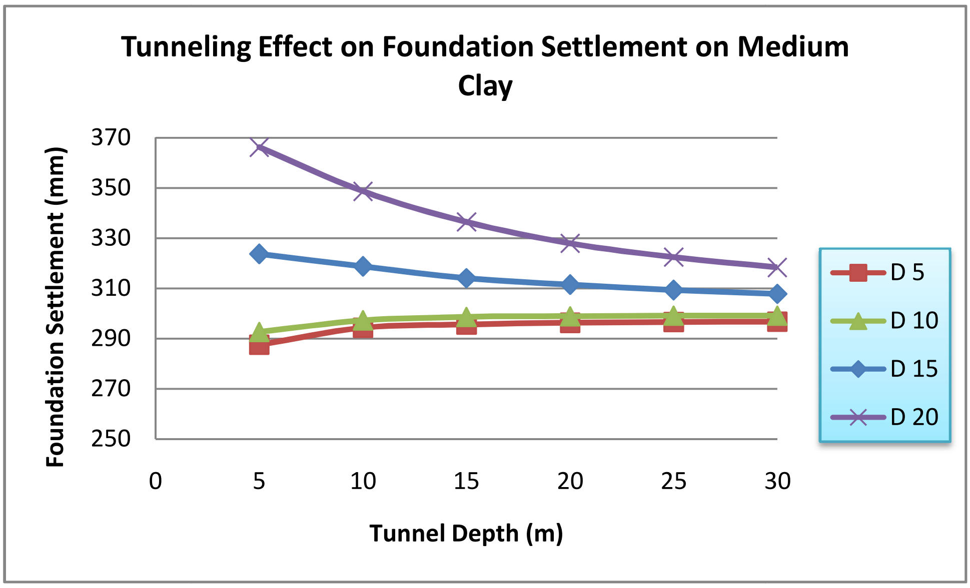

Figure A.14 Foundation settlement due to concentrated load 1500 KN in medium clay for different tunnel depth and diameter.

Table A.21 Foundation settlement due to concentrated load 1500 KN in hard clay for different tunnel depth and diameter.

Foundation settlement in Hard Clay (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	126.72	130.99	131.11	131.38	131.38	131.29
10	141.87	138.89	134.06	133.6	133.36	133.09
15	157.65	148.4	143.16	140.35	138.62	137.48
20	168.97	159.72	154.48	148.51	145.18	143.07

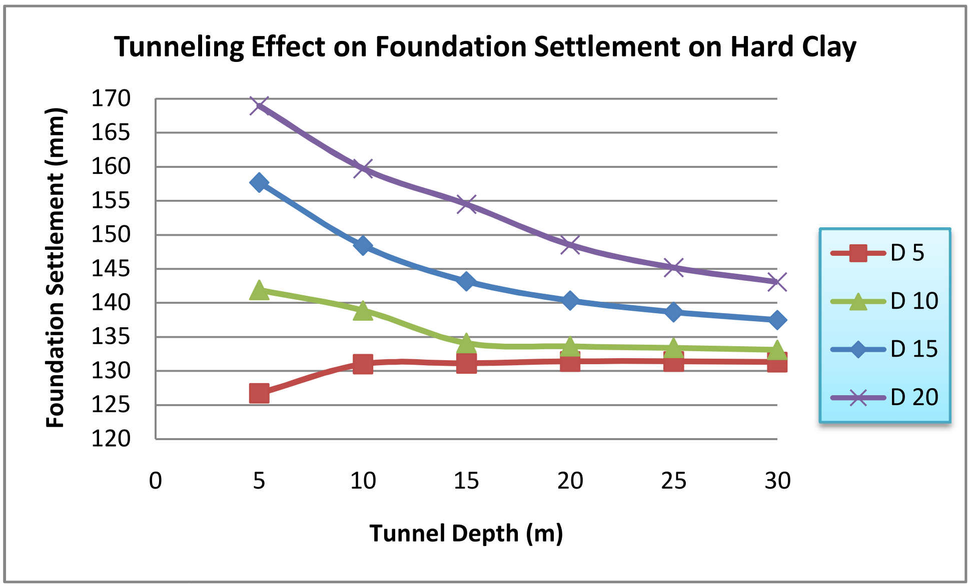

Figure A.15 Foundation settlement due to concentrated load 1500 KN in hard clay for different tunnel depth and diameter.

Table A.22 Foundation settlement due to concentrated load 1500 KN in loose sand for different tunnel depth and diameter.

Foundation settlement in Loose Sand (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	171.74	171.68	174.09	173.04	173.34	174.53
10	173.95	178.29	184.41	182.37	179.04	178.33
15	203.38	206.6	202.64	203.45	196.47	189.3
20	246.39	236.78	227.08	217.13	210.31	196.4

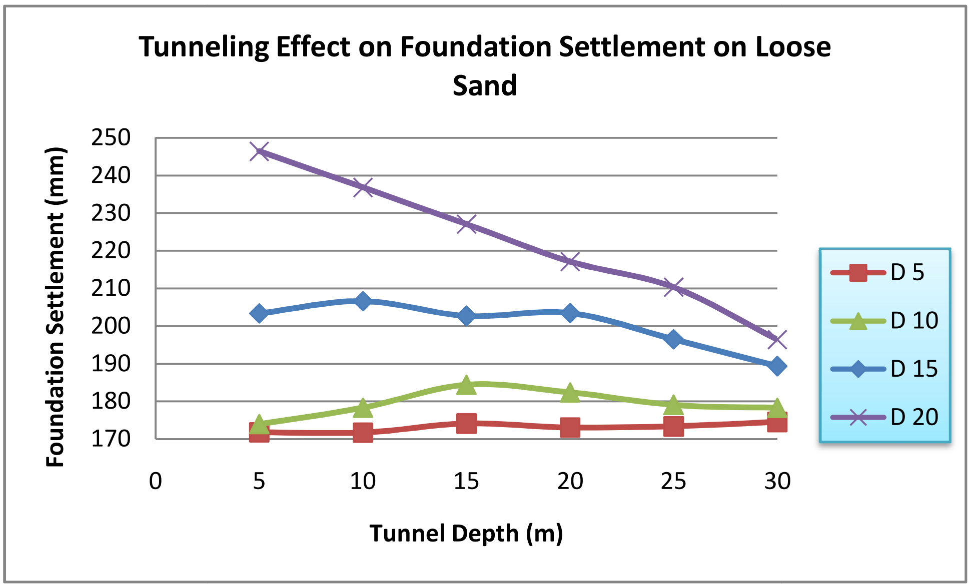

Figure A.16 Foundation settlement due to concentrated load 1500 KN in loose sand for different tunnel depth and diameter.

Table A.23 Foundation settlement due to concentrated load 1500 KN in mediumsand for different tunnel depth and diameter.

Foundation settlement in Medium Sand (mm)						
Depth (m) / Diameter (m)	5	10	15	20	25	30
5	91.82	91.42	90.95	91.09	91.56	91.6
10	101.44	101.52	101.75	100.55	97.28	94.96
15	126.83	122.8	117.86	112.66	107.46	99.57
20	167.05	148.11	135.72	121.39	114.35	104.77

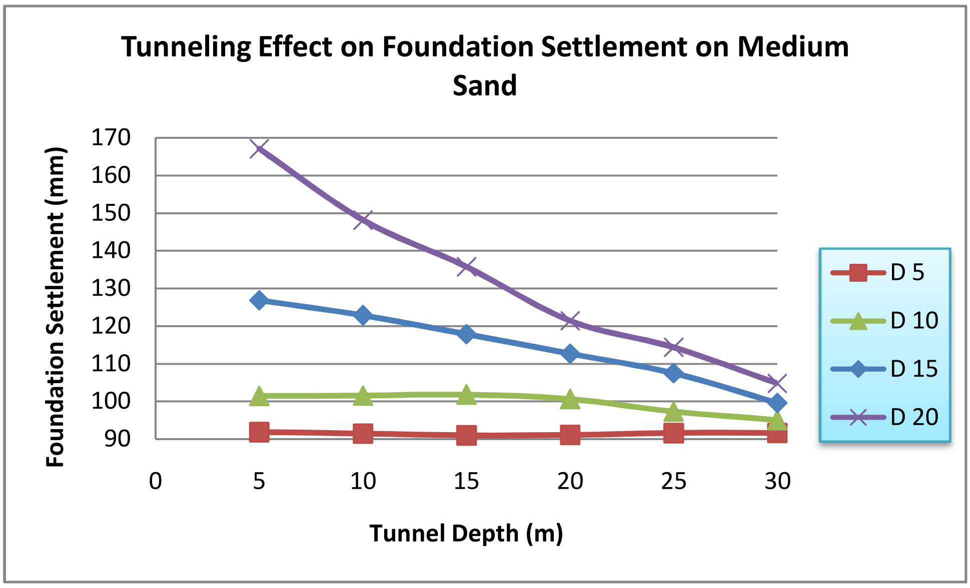

Figure A.17 Foundation settlement due to concentrated load 1500 KN in medium sand for different tunnel depth and diameter.

Table A.24 Foundation settlement due to concentrated load 1500 KN in dense sand for different tunnel depth and diameter.

Foundation settlement in Dense Sand (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	54.26	52.27	53.36	53.05	51.38	51.02
10	69.31	64.48	62.05	59.45	55.32	52.07
15	87.16	80.93	73.29	66.48	60.58	53.74
20	87.75	81.52	73.88	67.07	61.69	55.36

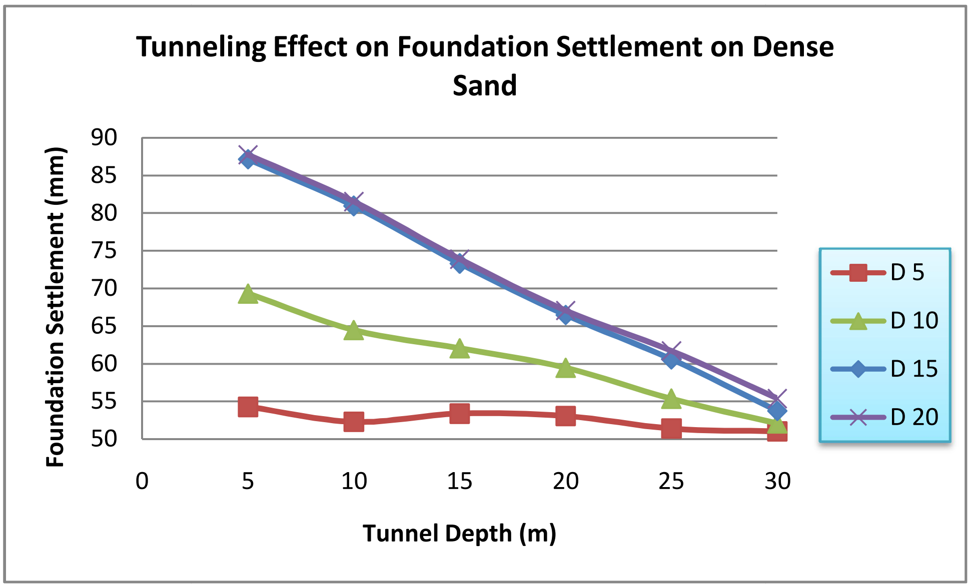

Figure A.18 Foundation settlement due to concentrated load 1500 KN in dense sand for different tunnel depth and diameter.

Table A.25 Foundation settlement due to concentrated load 2000 KN in soft clay for different tunnel depth and diameter.

Foundation settlement in Soft Clay (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	1100	1170	1180	1200	1205	1180
10	976.07	1162	1182	1179	1193	1178
15	996.6	1146	1165	1182	1167	1176
20	1116	1216	1232	1210	1204	1186

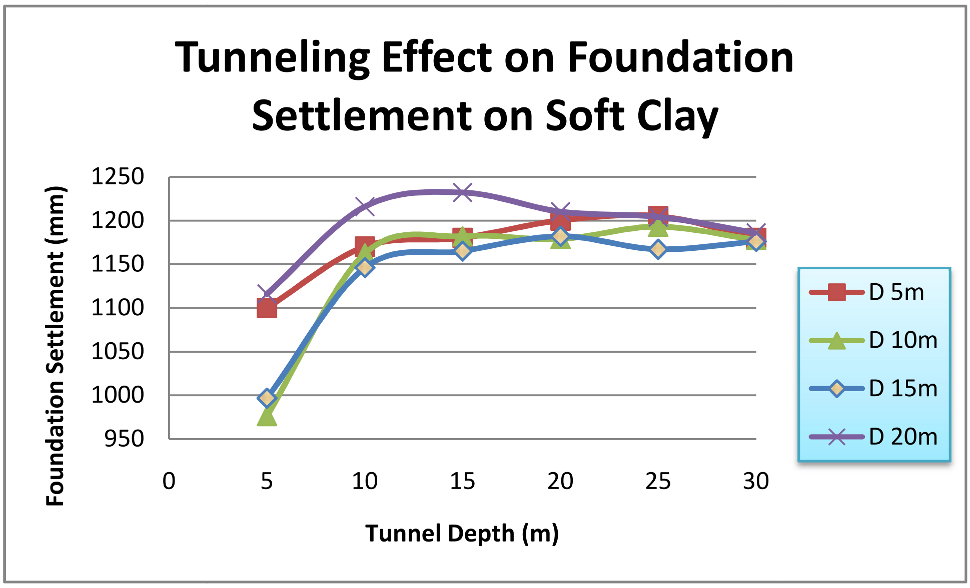

Figure A.19 Foundation settlement due to concentrated load 2000 KN in soft clay for different tunnel depth and diameter.

Table A.26 Foundation settlement due to concentrated load 2000 KN in medium clay for different tunnel depth and diameter.

Foundation settlement in Medium Clay (mm)						
Depth (m) / **Diameter (m)**	5	10	15	20	25	30
5	387.13	391.51	391.88	394.48	394.95	394.65
10	386.18	393.36	395.95	396.7	397.3	396.14
15	410.72	419.03	408.83	411.02	408.51	406.83
20	477.22	455.63	440.62	430.43	423.48	418.46

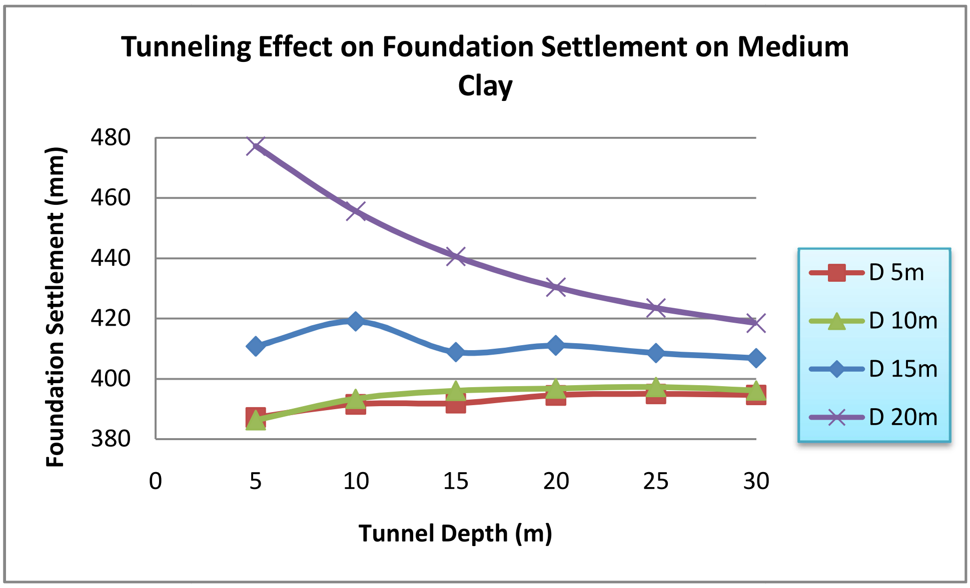

Figure A.20 Foundation settlement due to concentrated load 2000 KN in medium clay for different tunnel depth and diameter.

Table A.27 Foundation settlement due to concentrated load 2000 KN in hard clay for different tunnel depth and diameter.

Foundation settlement in Hard Clay (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	172.08	171.34	173.22	174.69	174.44	174.47
10	185.54	182.8	180.72	179.79	179.16	178.41
15	198.53	203.3	192.63	190.97	188.32	186.6
20	201.94	206.71	196.04	194.38	190.21	187.54

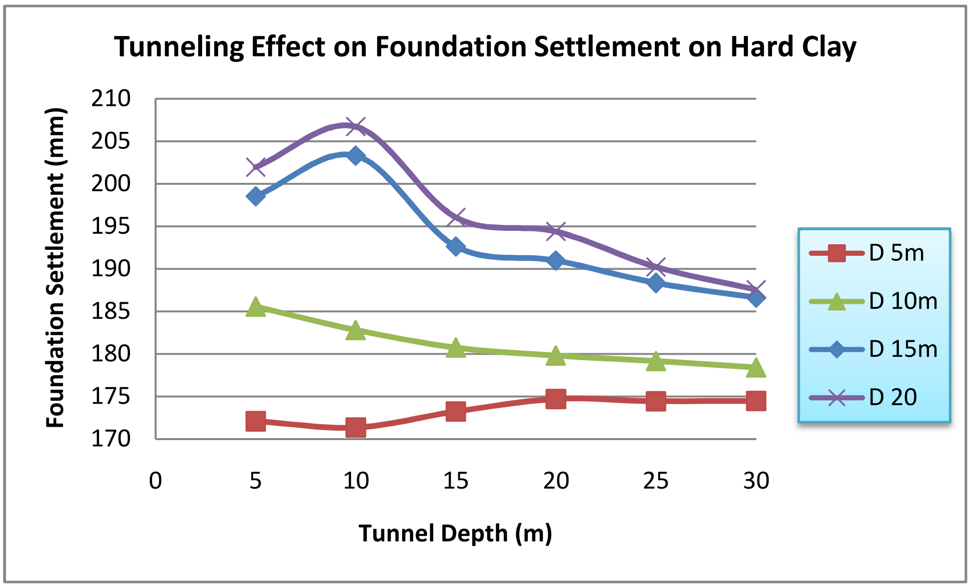

Figure A.21 Foundation settlement due to concentrated load 2000 KN in hard clay for different tunnel depth and diameter.

Table A.28 Foundation settlement due to concentrated load 2000 KN in loose sand for different tunnel depth and diameter.

Foundation settlement in Loose Sand (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	277.04	270.44	274.06	273.22	274.31	274.5
10	263.61	275.72	285.01	280.89	282.28	280.03
15	303.7	304.41	299.94	310.33	306.36	304.76
20	389.04	353.78	346.41	322.24	332.27	316

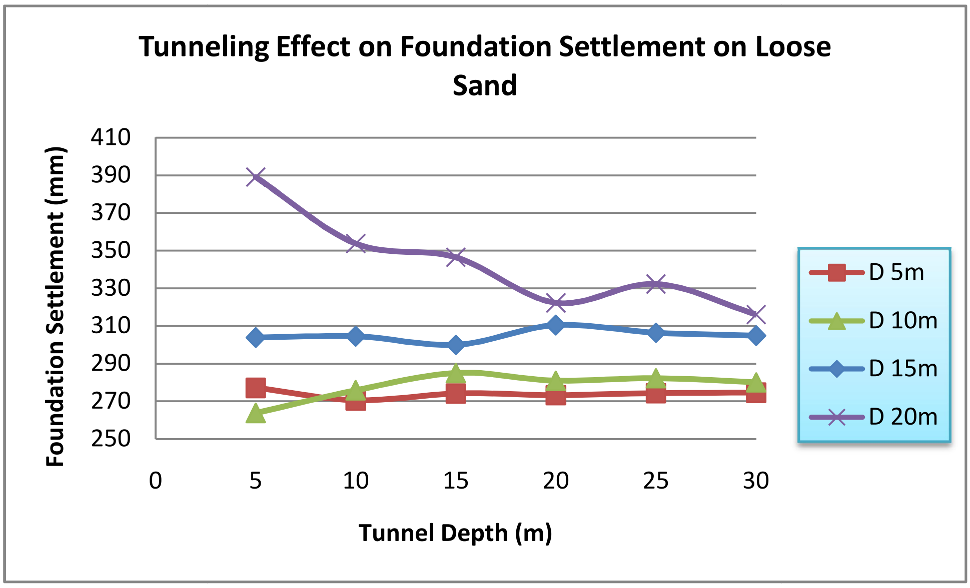

Figure A.22 Foundation settlement due to concentrated load 2000 KN in loose sand for different tunnel depth and diameter.

Table A.29 Foundation settlement due to concentrated load 2000 KN in medium sand for different tunnel depth and diameter.

Foundation settlement in Medium Sand (mm)						
Depth (m) **Diameter (m)**	5	10	15	20	25	30
5	144.11	141.34	142.13	143.03	141.48	143.74
10	147.27	150.2	152.96	152.06	150.66	149.59
15	188.17	175.35	171.36	170.72	166.63	161.17
20	229.07	200.5	189.76	186.45	172.94	164.84

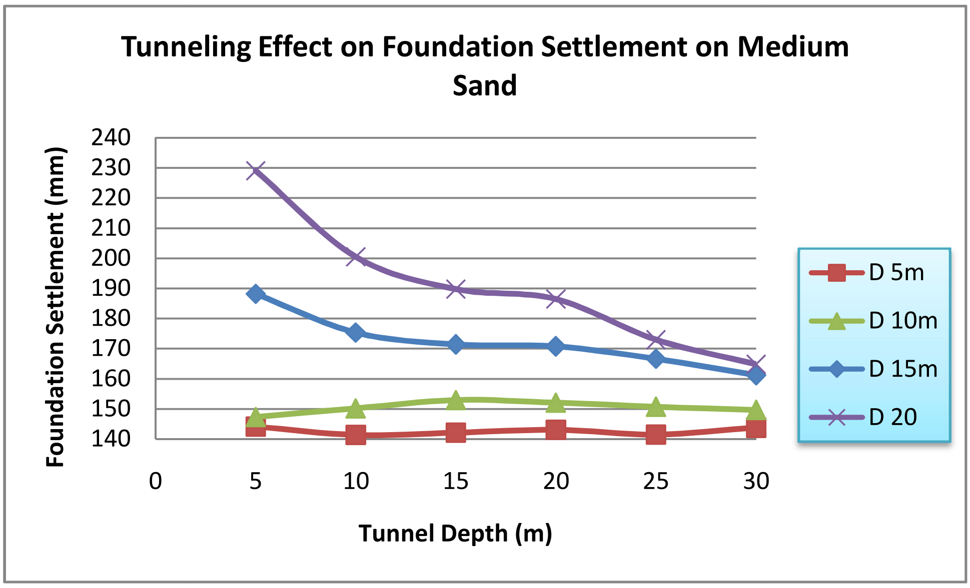

Figure A.23 Foundation settlement due to concentrated load 2000 KN in medium sand for different tunnel depth and diameter.

Table A.30 Foundation settlement due to concentrated load 2000 KN in dense sand for different tunnel depth and diameter.

Foundation settlement in Dense Sand (mm)						
Depth (m) / **Diameter (m)**	5	10	15	20	25	30
5	81.5	78.36	78.84	78.69	79.32	79.34
10	93.8	89.87	88.76	88.26	86.16	83
15	119.5	111.24	103.71	101.38	98.08	90.64
20	122.97	114.71	107.18	104.85	97.96	91.51

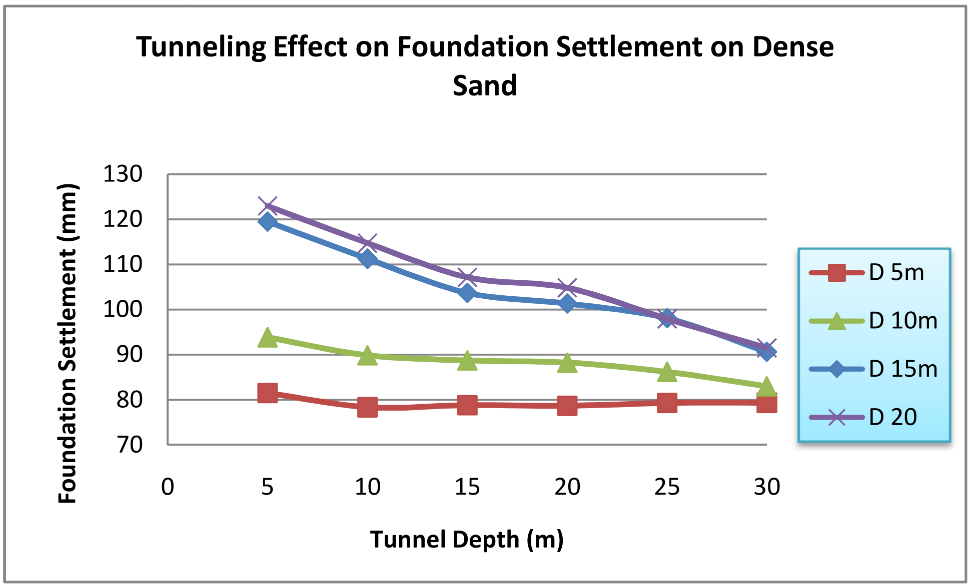

Figure A.24 Foundation settlement due to concentrated load 2000 KN in dense sand for different tunnel depth and diameter.

Appendix B

Table B-1 Typical mass densities of basic soil types (Das 2010)

Type of Soil	*Mass density ρ (Mg/m³)**			
	Poorly graded soil		*Well-graded soil*	
	Range	*Typical value*	*Range*	*Typical value*
Loose sand	1.70–1.90	1.75	1.75–2.00	1.85
Dense sand	1.90–2.10	2.07	2.00–2.20	2.10
Soft clay	1.60–1.90	1.75	1.60–1.90	1.75
Stiff clay	1.90–2.25	2.00	1.90–2.25	2.07
Silty soils	1.60–2.00	1.75	1.60–2.00	1.75
Gravelly soils	1.90–2.25	2.07	2.00–2.30	2.15

*Values are representative of moist sand, gravel, saturated silt, and clay.

Table B-2 Typical values of Poisson's ratio (μ)for soils (Bowles, J.E. 1982)

Type of soil	μ
Clay (saturated)	0.4 – 0.5
Clay (unsaturated)	0.1 – 0.3
Sandy clay	0.2 – 0.3
Silt	0.3 – 0.35
Sand (dense)	0.2 – 0.4
Course (void ratio = 0.4 – 0.7)	0.15
Fine grained (void ratio = 0.4 – 0.7)	0.25
Rock	0.1–0.4 (depends on type of rock)
Loess	0.1 – 0.3
Ice	0.36
Concrete	0.15

Table B-3 Typical Values of Hydraulic Conductivity of Saturated Soils (Das 2010)

	k	
Soil type	**cm /sec**	**ft /min**
Clean gravel	100–1.0	200–2.0
Coarse sand	1.0–0.01	2.0–0.02
Fine sand	0.01–0.001	0.02–0.002
Silty clay	0.001–0.00001	0.002–0.00002
Clay	<0.000001	<0.000002

Table B-4 Representative Values of the Modulus of Elasticity of Soil (Das 2010)

Soil type	E_s kN/m²	E_s lb/in.²
Soft clay	1,800–3,500	250–500
Hard clay	6,000–14,000	850–2,000
Loose sand	10,000–28,000	1,500–4,000
Dense sand	35,000–70,000	5,000–10,000

Table B-5SPT – based soil and rock classification systems

Sands	$(N_1)_{60}$ 0-3	Very loose
	3-8	Loose
	8-25	Medium
	25-42	Dense
	42-58	Very dense
Clays	N_{60} 0-4	Very soft
	4-8	Soft
	8-15	Firm
	15-30	Stiff
	30-60	Very stiff
	>60	Hard

Table B-6Typical values of drained angle of friction for sands interpretation from SPT (Mayne and Kemper (1988))

N	Ø	consistency
0-4	25-30	very loose
4-10	27-32	loose
10-30	30-35	medium
30-50	35-40	dense
>50	38-43	very dense

Table B-7Typical values of cohesion for clay interpretation from SPT (Mayne and Kemper (1988))

N	Cu (kPa)	consistency	visual identification
0-2	0 - 12	very soft	Thumb can penetrate > 25 mm
2-4	12-25	soft	Thumb can penetrate 25 mm
4-8	25-30	medium	Thumb penetrates with moderate effort
8-15	50-100	stiff	Thumb will indent 8 mm
15-30	100-200	very stiff	Can indent with thumb nail; not thumb
>30	>200	hard	Cannot indent even with thumb nail