ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE ENGINEERING AND TECHNOLOGY

TBM TUNNEL EXCAVATION INDUCED SETTLEMENTS AND CORRELATION WITH ACTUAL SITE MEASUREMENTS

M.Sc. THESIS

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Date of Submission : 15 December 2014
Date of Defense : 19 January 2015

FOREWORD

Tunnelling-induced settlements can damage to adjacent structures and utilities and disrupt the daily life. To reduce the cost and risk associated with tunnelling and protect the built enviroment, the good estimation for the ground settlement caused by the tunnel excavation is needed.

Urban tunnels are often excavated adjacent to sensetive structures such as high-rise buildings, beneath highways and bridge structures, and over or under other tunnels serving transportation or private and public utilities, vulnerable buildings. The increasing need for urban tunnelling in such densely developed underground areas and the associated risks are leading to seek consultants equipped with innovative yet proven methods for assessing these risks and for mitigating them through design before construction begins. Key to developing such designs is an understanding of tunnelling induced ground loss mechanisms and the associated displacements, and the risks they pose to adjacent buildings, structures and utilities.

Predicting the ground settlements made by means of three, Emprical Method, Analytical Solutions and Numerical Method. Although all these methods are well proven methods, by using the numerical method more reliable, sensitive analysis can be made under the complicated, heterogenous soil conditions.

December 2014

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TABLE OF CONTENTS

Page

ABBREVIATIONS

TBM **:** Tunnel Boring Machine
App **:** Appendix

LIST OF TABLES

LIST OF FIGURES

Page

TBM TUNNEL EXCAVATION INDUCED SETTLEMENTS AND CORRELATION WITH ACTUAL SITE MEASUREMENTS

SUMMARY

The Marmaray Project is a modern fast rail-track transportation scheme of 76.3 km connecting the European and Asian sides of the Istanbul City underneath the Bosphorus at a depth of 60 m. The objective of the Project is to establish a transportation capacity for about 150.000 passengers per hour in both directions and to reduce the present total travelling time of 185 minutes by nearly half. In Marmaray project, the tunnels were driven by one Earth Pressure Balance (EPB) and four Slurry Tunnel Boring Machine (TBM) under a shallow cover.

The tunelling process induces surface settlements that can cause damage to underground utilities and foundations and buildings and/or disrupt daily life damaging roads and pavements. Because of that reason, the sensetive settlement analysis should be made to take protective and precautionary measure in order to prevent the damages to built environment. Estimation of ground settlement caused by the tunnel excavation is important engineering point.

Although there are a large number of sources or causes of settlement, they can be conveniently lumped into two broad categories: those caused by ground water depression and those caused by lost ground.

Groundwater depression may be caused by intentional lowering of the water during construction of the tunnel itself acting as a drain. When either of these occurs, the effective stress in the ground increases. Basic soils mechanics can then be applied to estimate the resulting settlement. For tunnels in granular soil the settlement due to this increase in effective stress is usually reflected as an elastic phenomenon requiring knowledge of the low stress modulus of the ground and calculation of the change in effective stress. Unless the soil contains silt or very fine sand, this elastic settlement will typically represent the majority of the total but its absolute value will also be relatively small.

Lost ground causes are lumped into three groups: face losses, shield losses and tail losses.

Face losses results from movement in front of and into the shield. This includes running, flowing, caving, and/or squeezing behavior of the ground itself or simply mining more ground than displaced by the tunneling machine.

Shield losses occur between the cutting edge and the tail of the shield. All shields employ some degree of overcut so that they can be maneuvered. In addition, any time a shield is off alignment, the shield yaws, pitches, or plows when brought back to alignment.

Tail losses are similar to shield losses in that they are caused by the space being vacated by the tail itself as well as the extra space that must be provided between the tail and the support elements so those elements can be erected and so that they don't become "iron bound" and seize the tail shield. However, like the shield losses, these

tail voids will rapidly fill with soil if they are not first eliminated by grouting and/or expansion of the tunnel support elements.

In literature, three methods are exist to estimate the settlements caused by tunnelling, Emprical Method, Analytical Solutions and Numerical Method. Each of these methods are widely used.

The empirical method is the most common method for predicting the tunnel induced settlements. For a single tunnel in soft ground conditions, it is typically assumed the volume of surface settlement is equal to the volume of lost ground. However, the relationship between volume of lost ground and volume of surface settlement is complicated. Ground loss will produce a settlement trough at the ground surface where it can potentially impact the settlement behavior of any overlying or adjacent bridge foundations, building structures, or buried utilities transverse or parallel to the alignment of the proposed tunnel excavation. Empirical data suggests the shape of the settlement trough typically approximates the shape of an inverse Gaussian curve which may not provide required accuracy in the predictions.

Analytic method, is 2 dimensional and considers the supports are lineer elastically placed without any gap in the ground.

Numeric methods, model the geometry and ground which enables higher accuracy compared to analytical and numerical methods.

Some numerical modelling methods are Finite Elements Method, Finite Differences Method, Boundary Elements Method, Distinctive Elements Method, etc… The most commonly used numerical modelling method is Finite Element Method which also have been used for this thesis.

In Finite Element Methods, the geometry of the model have been defined. The outer borders of the model have been considered large enough to run the model sucessfully.

After the definition of the geometry, the boundary conditions have been defined.

The preloading conditions have been simulated considering the pre-excavated situation.

The underground water level have been defined.

The ground conditions have beend efined and Mohr-Coulomb material model have been chosen. Regarding the concrete and steel components, a lineer elastic material model have beend defined.

The excavation have been modelled considering the actual excavation stages. The tunnels have been modeled in the excavated order. The tunnel excavations have been modelled and settlement figures of some sections have been provided.

The numerical modelling results have been correlated with actual site measurements and empirical results. The numerical results tend to be much more similar with actual site measurements and empirical results tend to be too excessive in settlement figures.

In this respect, it is obvious that numerical results will provide much more reliable surface settlement and tunnel convergence values which will also result in providing much more accurate loading conditions on the support structures. Therefore, numerical models contribute optimisation of tunnel designs.

Precision of the predicted settlements becomes a very crucial issue for urban tunneling in case of a vulnerable built environment. The diversity of the involved parameters particularly under complex and heterogeneous soil matrix may intricate the prediction process. With good modeling of the conditions, the numerical calculations provided much more reliable results compared empiricial methods.

TBM TÜNELİ KAZISI KAYNAKLI OTURMALAR VE GERÇEKLEŞEN SAHA ÖLÇÜMLERİYLE KARŞILAŞTIRILMASI

ÖZET

Marmaray projesi Asya ile Avrupa'yı boğazın 60 m altından batırma tüp ile birbirine bağlayan 76.3 km'lik modern hızlı demiryolu taşımacılık sistemidir. Projenin amacı yolcu kapasitesini iki yönde saatte 150.000 yolcuya çıkarmak, toplam yolculuk süresini 185 dakikadan yaklaşık yarısına azaltmaktır. Marmaray projesinde 1 adet Toprak Basıncı Dengeleme (EPB) ve dört adet Bulamaç tipi Tünel Açma Makinesi (TBM) kullanılmıştır.

Tünel açımı nedeniyle kaynaklanan yüzey oturmaları altyapılara, yollara, temellere ve binalara zarar vererek günlük hayatı etkilemektedir. Bu nedenle, hassas oturma analizi yaparak yapısal çevreyi korumak için gerekli önlemleri önceden planlamak gerekmektedir. Tünel kaynaklı oturmayı tahmin etmek mühendislik açısından önemli bir konu olmaktadır.

Oturmaya sebep olan bir çok neden varolmakla beraber, iki ana grupta toplamak mümkündür. Bunlardan birincisi zemin suyu kaybı nedeniyle çökmesi, ikincisi ise zemin kaybı nedeniyle olan oturmadır.

Zemin suyu kaybı nedeniyle oluşan oturmada, tünelin kendisi drenajı sağlayarak, yer altı su seviyesinin düşmesine neden olur. Granüler zeminlerde, su kaybı nedeniyle oluşan oturmalar, ince daneli zeminlere göre daha kritik seviyelerde olabilir.

TBM tünellerinde zemin kaybı nedeniyle oluşan oturmalar; kazı aynası kaynaklı kayıplar, kalkan kaynaklı kayıplar ve kuyruk kaynaklar olarak üçe ayrılabilir.

Kazı aynası kaynaklı kayıplar genelde kazının kalkanın içine doğru girmesiyle oluşur. Herhangi bir şekilde zeminin akması, göçmesi, sıkışması tüm bu kayıpları arttırıcı niteliktedir.

Kalkan kaynaklı kalıplar; kesici kafa ve kuyruk arasındaki kayıplardır. Tüm kalkanlar belli bir oranda fazla kazarak, kesici kafanın manevra kabiliyetini arttırırlar.

Kuyruk kayıpları ise kalkan kayıplarına benzer olarak kuyruğun, yerleştirilmiş iksa elemanları ile arasında oluşmuş boşluklar nedeniyle oluşur.

Literatürde, tünel kaynaklı oturmaların tahmininde üç ana metod kullanılmatadır. Bunlar, ampirik metod, analitik çözümler ve nümerik metodlardır. Bu üç yöntem de yaygın bir şekilde kullanılmaktadır.

Ampirik yöntemler, tünel kaynaklı oturmaların tahmininde en çok kullanılan yöntemdir. Tek bir tünel kesitinin olduğu bir durumda hacim kaybı kaybolan zemine denk kabul edilebilir. Bu basitleştirilmiş kabulle, karmaşık olmayan yüzey oturması konuları çözülebilir. Bunanla beraber, yüzeysel temellerde çok, köprü ayakları, gömülü başka yapılar gibi oturmanın çalışılacağı başka konularda, bu basitleştirilmiş

yaklaşım gerekli hassasiyette sonuç vermeyebilir. Ampirik method yüzey oturmasının ters bir Gauss eğrisi şeklinde olacağını öngörürken, bu ters Gauss eğrisi dışında bir oturma şekli beklenecek geometrilerde istenilen hassasiyet edinilemeyebilir.

Ayrıca yaklaşım temelinde, daha önceden edinilmiş deneyimlerle geliştirilmiş ampirik formüllere dayandığı için, özellikle çok değişkenlik gösteren yumuşak zeminlerde, yeterli hassaslıkta sonuçlar veremeyebilir.

Analitik yöntem, ise iki boyutlu ve lineer elastik olarak iksaların zemine boşluksuz olarak yerleştiği bir yaklaşım kabulü yapar. Analitik yöntem ile yüzey oturması hesaplandıktan sonra, değişik derinlikler için oturma, yüzey oturması göz önüne alınarak tahmin edilir.

Nümerik yöntemler ise geometri ve zemin durumunu modelleyerek, amprik ve analitik yöntemlerin hassas olamadığı durumlarda daha başarılı sonuçlar verir.

Nümerik modelleme, bir çok karmaşık 2 ve 3 boyutlu durumu çözebildiği için tercih edilmekte, hatta nümerik modelleme olmadan günümüzde yapılan tünel tasarımı yok denecek kadar azdır.

Nümerik modelleme yöntemleri arasında Sonlu Elemanlar Yöntemi, Sonlu Farklar Yöntemi, Sınır Şartlar Yöntemi, Ayrık Elemanlar Yöntemi gibi birçok yöntem olmakla beraber, en çok kullanılan yöntemler Sonlu Elemanlar ve Sonlu Farklar Yöntemleridir. Bu çalışma da, Sonlu Elemanlar Yöntemi kullanılmıştır.

Sonlu Elemanlar Yönteminde önce çalışma alanının yapılacağı geometri modellenmiştir. Geometri belirlenirken, kazısı yapılan tünellerin çevresinde modelin sağlıklı çalışacağı öngörülen mesafeler bırakılmıştır.

Geometri belirlendikten sonra sınır şartlar belirlenmiştir.

Önyükleme koşulları, kazı öncesi durumu göz önüne alınarak uygulanmıştır.

Yer altı su seviyesi tanımlanmıştır.

Zemin parametreleri tanımlanmış ve Mohr-Coulomb malzeme modeli uygulanmıştır. Betonarme ve çelik elemanlar için lineer elastik bir malzeme modeli uygulanmıştır.

Kazı modellemesi, gerçekte oluşan kazı adımları dikkate alınarak sırasıyla tanımlanmıştır. Sırasıyla tünellerin kazıları modellenmiş, çeşitli kesitlerin oturma durumları gösterilmiştir.

Çalışma sonrası elde edilen nümerik yöntem sonuçları, daha önceden yapılmış ampirik sonuçlar ve gerçekleşen oturmalar ile karşılaştırılmış ve nümerik yöntemin gerçekleşen oturmalara yakın sonuçlar verdiği, ampirik sonuçların ise gerçekleşenlerden genelde çok yüksek tahmin edildiği görülmüştür.

Bu bağlamda nümerik yöntemler ile yüzey oturmalarının ve tünel konverjanlarının gerçekleşen durumu daha yakın olarak tahmin edildiği görülmüştür. Oturmaların gerçeğe daha yakın tahmin edilmesi, iksa sistemleri üzerinde oluşacak yükleme durumlarının da gerçeğe daha yakın tahmin edilebilmesi anlamına gelmektedir. Dolayısıyla nümerik yöntemler ile diğer yöntemlere göre daha optimum tünel tasarımları yapılabilmesi mümkündür.

Sonlu Elemanlar Yöntemiyle çalışırken, 2 yerine 3 boyutlu modellemeler kullanılması gerçeğe daha yakın sonuçlar alınmasını sağlamaktadır. Aynı şekilde kullanılacak ileri malzeme modelleri de sonuçların hassasiyetini arttıracaktır.

Şehir tünelciliğinde, özellikle zayıf yapısal çevrede, tahmin edilen oturmanın hassasiyeti önem arz etmektedir. Şartların nümerik modellemelerle daha iyi modellenmesi sayesinde, tünellerin etrafındaki yapılara verilebilecek olası hasarlar önceden tahmin edilip, bu hasarları önleyecek tasarımlar yapılabilir. Bu sayede, tünel yapısı çevresindeki yapılarda olumsuz etkiler oluşturmadan inşa edilebilir.

1. INTRODUCTION

1.1 Purpose of Thesis

In modern city, the fast and mass transportation is needed. Because of the restraints on gorunds and geology, the construction of tunnel is requred. The tunelling process induces surface settlements that can cause damage to underground utilities and foundations and buildings and/or disrupt daily life damaging roads and pavements. Due to that reason, the sensetive settlement analysis should be made to take protective and precautionary measure in order to prevent the damages to built environment.

In this thesis, source of ground settlements and method of analysis methods are discussed. Under this discussion, it is purposed to make a corrolation between the settlement analysis results and the actual site settlements data induced by TBM at Marmaray Project in this Master Thesis.

1.2 Literature Review

A.R. Salimi, M. Esmaeili, and B. Salehi presented a case study about analysis of a TBM tunnelling effection on surface subsidence. Their study includes prediction of surface subsidence caused by tunneling in one section of seventh line of Tehran subway. On the basis of geotechnical conditions of the region, tunnel with the length of 26.9km has been excavated, applying a mechanized method using an EPB-TBM with a diameter of 9.14m. In this regard, settlement is estimated utilizing both analytical and numerical finite element methods. The numerical method shows that the value of settlement in this section is 5cm. Besides, the analytical consequences (Bobet and Loganathan-Polous) are 5.29 and 12.36cm, respectively. According to results of this study, due to saturation of this section, there are good agreement between Bobet and numerical methods. Therefore, tunneling processes in this section needs a special consolidation measurement and support system before the passage of tunnel boring machine.

S.Möler studied tunnel induced settlements and structural forces in lining in his Ph.D Thesis. His thesis focuses on shollow tunnelling in soil. The thesis begins with a summary of modern tunnelling methods, distinguishing between installation procedures of open face and closed face tunnelling. Subsequently elementary methods of analysis for both settlements and lining forces are reviewed, placing emphasis on installation procedures. After that, the focus is on the use of three and two dimensional finite element analysis. It is shown that installation procedures are very important to be considered in order to arrive at proper prediction for tunnelling settlements, horizontal ground movements and lining forces.

Lee van Kessel consists of a case study thesis for the building response due to tunnel induced settlements in his MSc Thesis. A case study is done to examine the response of facades in the Daniel Stalpertstraat due to settlements caused by the tunnelling process. This field data is used to find a calibrated 2D numerical model of the facades. The calibrated numerical model is then subjected to larger settlements to obtain the behaviour of the facades at large settlements. Using linear and nonlinear analyses of the numerical model it is evaluated how accurate and how conservative. With linear analyses it is checked how reliable the methods are in terms of strains and deformation under the imposed settlements. With nonlinear analyses the conservativeness of each method in terms of damage is evaluated.

Adolfo Camilo Torres Prada, Fernando Alberto Nieto Castañeda studied the settlements induced by TBM in soft grounds in BOGOTÁ – COLOMBIA. The present settlement studies were conducted using three research methods: strict monitoring of ground surface movements in an observation area of 40.5m by 16m over the sewer system tunnel “Fucha – Tunjuelo” in Bogota, Colombia of 3,75m internal diameter and 9 m depth, excavated with TBM type EPB through soils like sand and clay; a physical tridimensional model was constructed on a reduced geometric scale al = 10 in 1g type testing. The large scale model was constructed in a 2,4m by 2,5m in plan and 1,6m deep bin at GeoLab (Laboratory of Geotechnical Models) of the La Salle University in Bogota. A mixture of sand, oil, and bentonite was used to simulate natural soil; this mixture was precisely calculated applying the laws of scales. The surface movements were registered by LVTD´s, internal pressures of soil were measured by load cells, and vectors of movement were registered using close range photography at the surface and in the cross sections (the visible sides) of the model. The simulation of the tunneling process was conducted with a simple model of TBM prototype and control of pressure in the cut head. Also, a series of numerical models in 3D using FEM were made. Finally, this paper presents the comparison of all results obtained using the three research methods, and a particular behavior of the analyzed soil during excavation is defined. The study of settlements induced by tunneling where new technologies are used is important to develop in a safe way the modernization of primary service tunnel systems in Bogota.

Wanling Chong studied Tunnel Boring Machine (TBM) performance in Singapore's Mass Rapid Transit (MRT) system in his MSc Thesis. The thesis summarizes and evaluates the performance of Tunnel Boring Machine construction in Singapore's MRT network. Surface settlement induced by the tunnelling process can cause damage to underground utilities and foundations and buildings and/or distrupt daily life damaging roads and pavements, and is used as a measure of performance in his thesis. The influence of encountered geology and adopted construction methods on settlement is discussed. The dominant construction method on all four existing MRT lines involved the use of shield TBMs, with the main difference being the method of face support adopted. The variability in ground conditions which resulted from mixed weathering grades and mixed face is a direct result of extensive tropical weathering of Singapore's soils , and poses a challenge to the performance of EPB machine during construction.

Mohammed Y. Fattah, Kais T. Shlash amd Nahla M. Salim presented a study on the prediction of settlement trough induced by tunneling in cohesive ground. Surface settlements of soil due to tunneling are caused by stress relief and subsidence due to movement of support by excavation. There are significant discrepancies between empirical solutions to predict surface settlement trough, because of different interpretations and database collection by different authors. In their study, the shape of settlement trough caused by tunneling in cohesive ground is investigated by different approaches, namely analytical solutions, empirical solutions, and numerical solutions by the finite element method. The width of settlement trough was obtained by the finite element method through establishing the change in the slope of the computed settlement profile. The finite element elastic-plastic analysis gives better predictions than the linear elastic model with satisfactory estimate for the displacement magnitude and slightly overestimated width of the surface settlement trough.

M.Karakuş studied predicting horizontal movement for a tunnel by empirical and FE Methods. There are a number of benefits in constructing transport tunnels in highly populated areas for relieving traffic congestion and increasing the speed of travel for commuters. Instead of constructing a single tunnel, multiple tunnels constructed side by side offer more benefits. However, to avoid any adverse effects of tunnels on one another, more attention should be given to estimation of horizontal movement, which will ease not only support design, but also help to determine critical regions around a tunnel. Thus, a number of Finite Element Methods (FEM) as well as empirical analyses were conducted in this research to estimate horizontal movement profiles for a tunnel. The results of both analyses were compared with field measurements as well as each other. The comparison showed that the empirical models could be used to estimate the far-field settlement profiles, but that they could not be used for the near-field ground response to tunnelling. In the study, it is concluded that finite element analyses were in very good agreement with not only far-field but also near-field ground response tunnelling.

Ian Turner and Michael Yap study on assessment of ground movement impacts on existing tunnels. As an increasing number of tunnelling schemes are being designed and constructed in congested cities around the world, the complications involved as a consequence of tunnelling on existing sub-surface structures are becoming ever more challenging for engineers. Most would agree that the impact of tunnelling-induced ground movement on existing tunnels is an important issue, and the need for improved technical knowledge for solving this complex soil-structure interaction problem is essential. The potential effects of these ground movements on the longitudinal and transverse response on the structural behaviour of the existing tunnels are then investigated. The shortcomings of commonly adopted assessment methods which may result in non-conservatism and mask critical issues of existing tunnels at tunnel crossings are highlighted throughout the discussion. Their study demonstrates the use of modern computing to do powerful complex analysis to solve the complexities of soil-structure interaction and manage risks associated with tunnel crossings.

C. Pound, Y. S. Hsu and G R Walker study on predicted and observed ground movements around a Tunnel Boring Machine at Heathrow Airport. The Airside Road Tunnel (ART) at Heathrow Airport comprises twin 8.1m diameter tunnels constructed by TBM in London Clay at shallow depth beneath taxiways and above an existing rail tunnel. To assess the effect of construction of the ART on the Heathrow Express (HEX) rail tunnel, a series of three-dimensional numerical analyses were carried out using the finite difference program FLAC3D. The analyses adopted a non-linear model to replicate the small strain stiffness of the over-consolidated soil stratum at tunnel horizon. Three construction methods were considered, comprising open face, compressed air and earth pressure balance tunnelling. The analyses demonstrated the significant contribution of closure of the ground around the tunnel shield on the overall ground movements caused by the tunnelling and the benefit of the use of ground support pressures in reducing the total magnitude of ground movements. The analyses predicted that the loads and deformations induced in the underlying HEX tunnel would be small whatever construction method was adopted and that they would be within operating tolerances for the railway. Instrumentation installed in the early part of the ART showed that heave was occurring at the ground surface ahead and over the tunnel face. The analyses were rerun to investigate this effect and the movements of the HEX tunnel were again predicted to be small. Monitoring of the HEX tunnel as the ART TBM passed above showed small movements and no damage or distress was caused to the tunnel or unacceptable movement of the rail tracks. The monitoring did indicate that the tunnel settled as the TBM passed and this could not be replicated in the numerical modelling.

Snežana Maraš-Dragojević study on analysis of ground settlement caused by tunnel construction. Prediction of ground settlement is considered as highly significant in the design of tunnels located in urban areas. 2D and 3D modelling of tunnel construction, as needed for settlement analysis, is made according to the finite-element method. The ground settlement profiles, obtained during simulation of the small-depth openface tunnel excavation in clayey-marly terrain, are presented. Settlement cross sections obtained by 2D and 3D analyses are compared.

Ahmet Unlutepe, Volkan Tellioglu, Basar Arioglu study on predicted and observed ground deformations due to TBM tunnel excavation on Izmir Metro project

(Stage 1). Stage 1 metro construction for the city of Izmir, which has the third highest population in Turkey, has been successfully completed. It included a 2.75 km tunnel excavated by tunnel-boring machine (TBM) using the earth pressure balance (EPB) method in soft ground conditions with shallow overburden. Their study revisits the surface settlement values predicted by the empirical approach, adds numerical analysis, describes the geotechnical instrumentation for field performance and then compares the observed deformations with empirically and numerically predicted ground deformations.

Ngoc-Anh Do, Daniel Dias, Pierpaolo Oreste and Irini Djeran-Maigre study on 3D modelling for mechanized tunnelling in soft ground-influence of the constituve model. The main purpose of their study was to provide a three-dimensional numerical model which would allow to show the influence of the constitutive model of the ground on the tunnel lining behaviour and displacement field surrounding the tunnel. Most of the processes that occur during mechanized excavation have been simulated in this model. Besides the well-known elastic linear perfectly plastic model with Mohr Coulomb failure criteria (MC), the Cap-Yield model (CYsoil), which is a strain hardening constitutive model that takes into consideration the hyperbolic behaviour of the soil has also been adopted. The results have shown a considerable effect of the constitutive model on the tunnel behavior and the ground displacement field. Generally, the CYsoil model produces higher structural force and ground settlement values than those predicted by the MC model.

Lily Chow study on the prediction of surface settlements due to tunneling in soft ground. The investigation of various soil models for the prediction of surface settlement profiles due to the excavation of a single tunnel in soft ground is described. Both analytical and numerical methods are used. Emphasis is placed on the establishment of the width of the predicted settlement trough. In the analytical analysis, the gorund is assumed to be an elastic half-space. The problem is approached using two methods: (i) the application of a line load to model the uplift of soil caused by excavation; (ii) the application oof method which models the soil deformation due to ground loss.

R.K. Rowe study on a theoretical examination of the settlements induced by tunnelling. The application of the procedure for predicting and designing for settlement above tunnels constructed in soft ground is discussed and its range of applicability is examined by consideration of four case histories. It is suggested that the procedure will provide a conveinent means of estimating settlement induced by tunneling in many different soil deposits provided that reasonable construction procedures are adopted.

2. SETTLEMENT

2.1 Tunneling Induced Settlement

Ground settlement is of greater concern for soft ground tunnels than for rock for two reasons:

- Settlements are nearly always greater for soft ground tunnels.
- Typically more facilities that might be negatively impacted by settlements exist near soft ground tunnels than near rock tunnels.

With modern means and methods, both the designer and the contractor are now better equipped to minimize settlements and, hence, their impact on other facilities.

2.2 Sources of Settlement

Although there are a large number of sources or causes of settlement, they can be conveniently lumped into two broad categories: those caused by ground water depression and those caused by lost ground.

2.2.1 Groundwater depression

Groundwater depression may be caused by intentional lowering of the water during construction or by the tunnel itself (or other construction) acting as a drain. When either of these occurs, the effective stress in the ground increases. Basic soils mechanics can then be applied to estimate the resulting settlement. For tunnels in granular soil the settlement due to this increase in effective stress is usually reflected as an elastic phenomenon requiring knowledge of the low stress modulus of the ground and calculation of the change in effective stress. Unless the soil contains silt or very fine sand, this elastic settlement will typically represent the majority of the total but its absolute value will also be relatively small.

For fine grained soils, the situation is a bit more challenging but certainly manageable using normal soil mechanics approaches. With fine-grained soils, the conditions are reversed. In most instances, the settlement is mostly due to consolidation brought on by the changes in effective stress and hence is analyzed by the usual soil mechanics consolidation theories. In some instances, primarily if lenses of sands are contained in the soil, there may also be a relatively small contribution by elastic compression. In comparison to the settlement of granular soils, consolidation can lead to several inches of settlement when the consolidating soils are thick and the change in effective stress is significant.

2.2.2 Lost ground

Lost ground has a number of root causes and is usually responsible for the settlements that make the headlines. By definition, lost ground refers to the act of taking (or losing) more ground into the tunneling operation than is represented by the volume of the tunnel. Thus it is highly reflective of construction means and methods. Modern tunnelling machines can be a great help in controlling lost ground but in the end it usually comes down to quality of workmanship.

The causes of lost ground are lumped into three groups: face losses, shield losses and tail losses.

- Face losses results from movement in front of and into the shield. This includes running, flowing, caving, and/or squeezing behavior of the ground itself or simply mining more ground than displaced by the tunneling machine.
- Shield losses occur between the cutting edge and the tail of the shield. All shields employ some degree of overcut so that they can be maneuvered. In addition, any time a shield is off alignment, the shield yaws, pitches, or plows when brought back to alignment. Mother Nature abhors a vacuum and the surrounding soils begin to fill these planned or produced voids the instant they are produced.
- Tail losses are similar to shield losses in that they are caused by the space being vacated by the tail itself as well as the extra space that must be provided between the tail and the support elements so those elements can be erected and so that they don't become "iron bound" and seize the tail shield. However, like the shield losses, these tail voids will rapidly fill with soil if they are not first eliminated by grouting and/or expansion of the tunnel support elements.

2.3 Settlement Analysis Methods

2.3.1 Empirical methods

The Volume Loss method is the most common empirical method for predicting the tunnel induced settlements.

The volume of ground loss experienced during tunneling can be related to the volume of settlement expected at the ground surface (Peck, 1969). For a single tunnel in soft ground conditions, it is typically assumed the volume of surface settlement is equal to the volume of lost ground. However, the relationship between volume of lost ground and volume of surface settlement is complicated. Volume change due to bulking or compression is typically not estimated or included in the calculations. Ground loss will produce a settlement trough at the ground surface where it can potentially impact the settlement behavior of any overlying or adjacent bridge foundations, building structures, or buried utilities transverse or parallel to the alignment of the proposed tunnel excavation. Empirical data suggests the shape of the settlement trough typically approximates the shape of an inverse Gaussian curve.

The vagaries of nature and of construction are such that settlements cannot be estimated in soft ground tunnels to the same level of confidence as, say, the settlement of a loaded beam. Thus, there are two related methods to attack the problem: experience and empirical data as mentioned in Table 2.1

Table 2.1 : Relationship between Volumes Loss and Construction Practice and Ground Conditions

Case	*VL* (%)
Good practice in firm ground; tight control of face pressure within closed face machine in slowly raveling or squeezing ground	0.5
Usual practice with closed face machine in slowly raveling or squeezing ground	1.0
Poor practice with closed face in raveling ground	2
Poor practice with closed face machine in poor (fast raveling) ground	3
Poor practice with little face control in running ground	4.0 or more

The volume loss method is a semi-empirical method based partially on theoretical grounds. The method introduces, although indirectly, the basic parameters of excavation into the analysis (these include mechanical parameters of a medium, technological effects of excavation, excavation lining etc) using 2 comprehensive parameters (coefficient k for determination of inflection point and a percentage of volume loss VL). These parameters uniquely define the shape of subsidence trough and are determined empirically from years of experience as mentioned in Figure 2.1

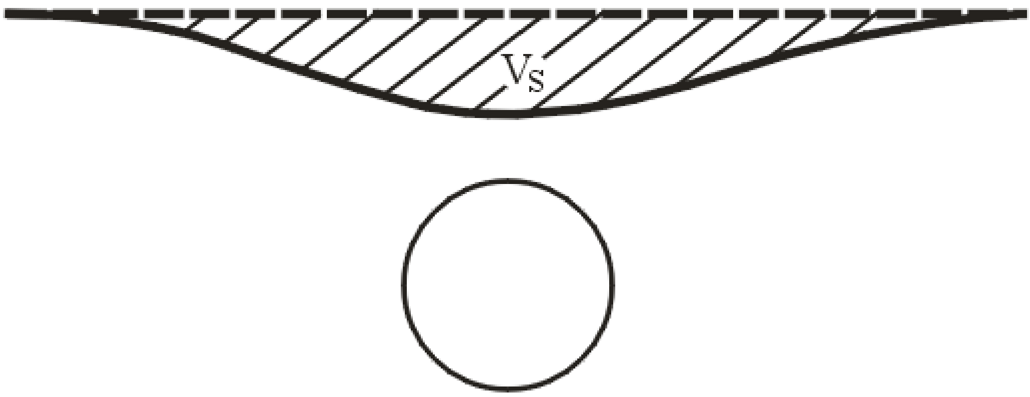


Figure 2.1: Settlement expressed in terms volumes

The maximum settlement Smax, and location of inflection point Linf are provided by the following expressions in Equation 3.1 and Equation 3.2 :

$$L_{\inf} = k.Z \tag{3.1}$$

$$S_{\max} = \frac{A.V.L}{100} \cdot \frac{1}{\sqrt{2.\pi}.L_{\inf}} \tag{3.2}$$

where:
A - excavation area
Z - depth of center point of excavation
k - coefficient to calculate inflection point (material constant)
VL - percentage of volume loss

The roof deformation u_a follows below as mentioned in Equation 3.3:

$$u_a = \frac{2.r - \sqrt{4.r^2 - \frac{4.r^2.VL}{100}}}{2} \tag{3.3}$$

where:
r - excavation radius
VL - percentage of volume loss

2.3.2 Recommended values of parameters for volume loss analysis

Data needed for the determination of subsidence trough using the volume loss method as mentioned in Table 2.2 and Table 2.3:

Table 2.2 : Coefficient to calculate inflection point k

Soil or rock	k
cohesionless soil	0,3
normally consolidated clay	0,5
over consolidated clay	0,6-0,7
clay slate	0,6-0,8
quartzite	0,8-0,9

Table 2.3 : Percentage of volume loss

Technology	VL
TBM	0,5-1
Sequential excavation method(NATM)	0,8-1,5

Several relationships were also derived to determine the value of lost volume VL based on stability ratio N defined by Broms and Bennermarkem as mentioned in Equation 3.4:

$$N = \frac{\sigma_v . \sigma_t}{S_n} \tag{3.4}$$

where:

σ_v - overall stress along excavation axis

σ_t - excavation lining resistance (if lining is installed)

S_n - undrained stiffness of clay

For N < 2 the soil/rock in the vicinity of excavation is assumed elastic and stable, for N < 2,4 local plastic zones begin to develop in the vicinity of excavation, for N < 4,6 a large plastic zone develops around excavation and for N = 6 the loss of stability of tunnel face occurs. Figure 2.2 shows the dependence of stability ration and lost volume VL.

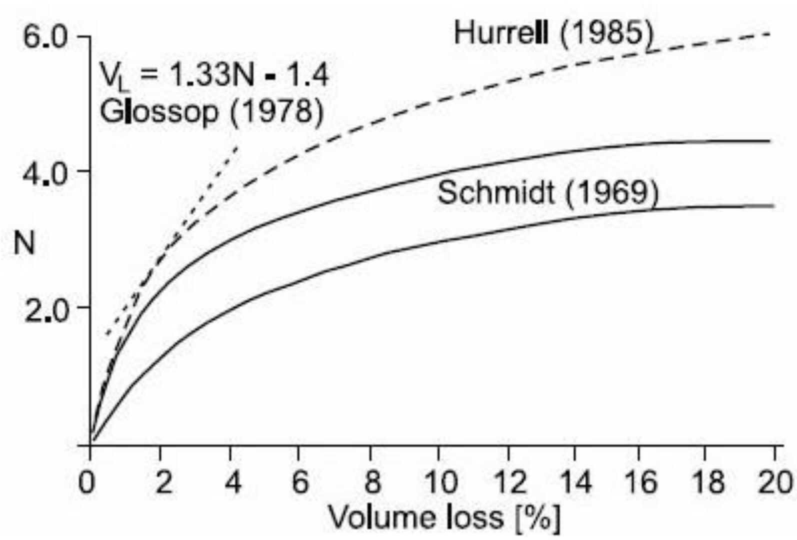


Figure 2.2: The stability ration and lost volume

2.4 Analytical Solutions

Analytical solutions for ground-support interaction for a tunnel in soil are available in the literature. The solutions are based on two dimensional, plane strain, linear elasticity assumptions in which the lining is assumed to be placed deep and in contact with the ground (no gap), i.e., the solutions do not allow for a gap to occur between the support system and ground.

Early analytical solutions by Burns and Richard (1964), Dar and Bates (1974), and Hoeg (1968) were derived for the overpressure loading, while solutions by Morgan (1961), Muir Wood (1975), Curtis (1976), Rankin, Ghaboussi and Hendron (1978), and Einstein et. al. (1980) were for excavation loading. Solutions are available for the full slip and no slip conditions at the ground-lining interface.

2.4.1 Classical Theory

Convergence analysis of an excavation and calculation of the maximum settlement in a homogeneous body are the same for all classical theories. The subsidence trough analyses then differ depending on the assumed theory (Peck, Fazekas, Limanov).

When calculating settlement, first, determine the radial loading of a circular excavation as mentioned in Equation 3.5:

$$p = \sigma_z \frac{1+K_r}{2} \tag{3.5}$$

where:

σ_z - geostatic stress in center of excavation

K_r - coefficient of pressure at rest of cohesive soil

The roof u_a and the bottom u_b deformations of excavation follow from as mentioned in Equation 3.6 and Equation 3.7:

$$u_a = (1+v).\frac{p}{E}.r.\frac{Z+(1-2.v).r}{Z+r} \tag{3.6}$$

$$u_b = -(1+v).\frac{p}{E}.r.\frac{Z+(1-2.v).r}{Z+r} \tag{3.7}$$

where:

Z - depth of center point of excavation

r - excavation radius

E - modulus of elasticity of rock/soil in vicinity of excavation

ν - Poisson's number of rock/soil in vicinity of excavation

The maximum terrain settlement and the length of subsidence trough are determined as follows as mentioned in Equation 3.8 and Equation 3.9:

$$S_{\max} = (1-v^2).\frac{p}{E}.r.\frac{4.r^2.Z}{Z^2-r^2} \tag{3.8}$$

$$L = 2.\sqrt{Z^2-r^2} \tag{3.9}$$

where:

Z - depth of center point of excavation

r - excavation radius

E - modulus of elasticity of rock/soil in vicinity of excavation

ν - Poisson's number of rock/soil in vicinity of excavation

When the tunnel roof displacement is prescribed the maximum settlement is provided by the following expression as mentioned Equation 3.10:

$$S_{\max} = 4.u_a \cdot \frac{Z.(1-v)}{(Z+r).(Z+r+2.v.r)} \tag{3.10}$$

where:

Z - depth of center point of excavation

r - excavation radius

u_a - tunnel roof displacement

ν - Poisson's number of rock/soil in vicinity of excavation

When determining a settlement of layered subsoil, it is first calculated the settlement at the interface between the first layer above excavation and other layers of overburden S_{int} and determines the length of subsidence trough along layers interfaces. In this case the approach complies with the one used for a homogeneous soil as mentioned in Figure 2.3

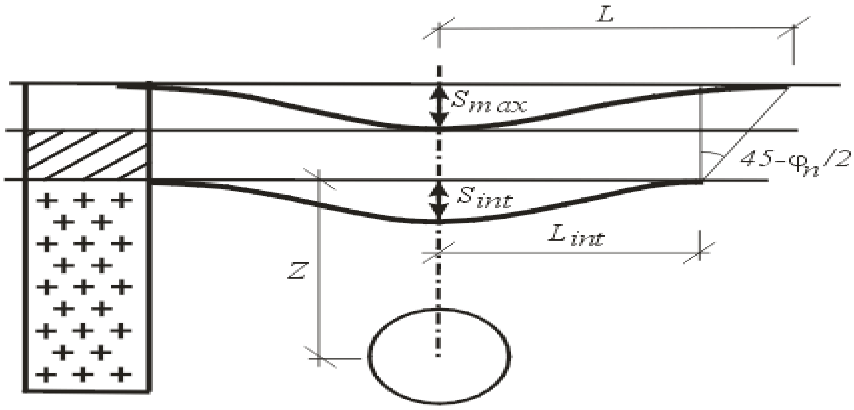


Figure 2.3: Analysis of settlement for layered subsoil

The next computation differs depending on the selected analysis theory:

Limanov described the horizontal displacement above excavation with the help of lost area F as mentioned in Equation 3.11:

$$S_{\max} = \frac{L}{F} \tag{3.11}$$

where:

L - length of subsidence trough

F - volume loss of soil per 1 m run determined from below as mentioned in Equation 3.12:

$$F = S_{\text{int}}.\pi.\frac{L_{\text{int}}}{2} \tag{3.12}$$

where:

L_{int} - length of subsidence trough along interfaces above excavation

S_{int} - settlement of respective interface

Fazekas described the horizontal displacement above excavation using the following expression as mentioned in Equation 3.13:

$$S_{\max} = S_{\text{int}}.\frac{L_{\text{int}}}{L} \tag{3.13}$$

where:

L - length of subsidence trough

L_{int} - length of subsidence trough along interfaces above excavation

S_{int} - settlement of respective interface

Peck described the horizontal displacement above excavation using the following expression as mentioned in Equation 3.14:

$$S_{max} = S_{int}.\frac{L_{int}}{L_{inf}} \tag{3.14}$$

where:

L_{int} - length of subsidence trough along interfaces above excavation

S_{int} - settlement of respective interface

L_{inf} - distance of inflection point of subsidence trough from excavation axis at terrain surface

2.4.2 Curve based on Gauss

A number of studies carried out both in the USA and Great Britain proved that the transverse shape of subsidence trough can be well approximated using the Gauss function. This assumption then allows us to determine the horizontal displacement at a distance x from the vertical axis of symmetry as mentioned in Equation 3.15:

$$S_i = S_{max}.e^{\left(\frac{-x_i^2}{2.L_{inf}^2}\right)} \tag{3.15}$$

where:

S_i - settlement at point with coordinate x_i

S_{max} - maximum terrain settlement

L_{inf} - distance of inflection point

2.4.3 Curve based on Aversin

Aversin derived, based on visual inspection and measurements of underground structures in Russia, the following expression mentioned in Equation 3.16 for the shape of subsidence trough:

$$S_i = S_{max}.\left(1 - \frac{x_i}{L}\right)^4.e^{\left(\frac{4.x_i}{L}\right)} \tag{3.16}$$

where:

S_i - settlement at point with coordinate x_i

S_{max} - maximum terrain settlement

L - reach of subsidence trough

2.4.4 Coefficient of calculation of inflection point

When the classical methods are used the inputted coefficient k_{inf} allows the determination of the inflection point location based on $L_{inf}=L/k_{inf}$. In this case the coefficient k_{inf} represents a very important input parameter strongly influencing the shape and slope of subsidence trough. Its value depends on the average soil or rock, respectively, in overburden, literature offers the values of kinf in the range 2,1 - 4,0.

Based on a series of FEM calculations the following values are recommended in Table 2.4:

Table 2. 4: The coefficient for calculation of inflection point

Soil or rock	k_{inf}
gravel soil G1-G3	3,5
sand and gravel soil S1-S5,G4,G5, rocks R5-R6	3,0
fine-grained soil F1-F4	2,5
fine-grained soil F5-F8	2,1

2.4.5 Subsidence through with several excavations

The principal of superposition is used when calculating the settlement caused by structured or multiple excavations. Based on input parameters, first it is determined subsidence troughs and horizontal displacements for individual excavations. The overall subsidence trough is determined subsequently.

Other variables, horizontal strain and gradient of subsidence trough, are post-processed from the overall subsidence trough.

2.4.6 Analysis of subsidence through at a depth

A linear interpolation between the maximal value of the settlement S_{max} at a terrain surface and the displacement of roof excavation ua is used to calculate the maximum settlement S at a depth h below the terrain surface in a homogeneous body as mentioned in Figure 2.4:

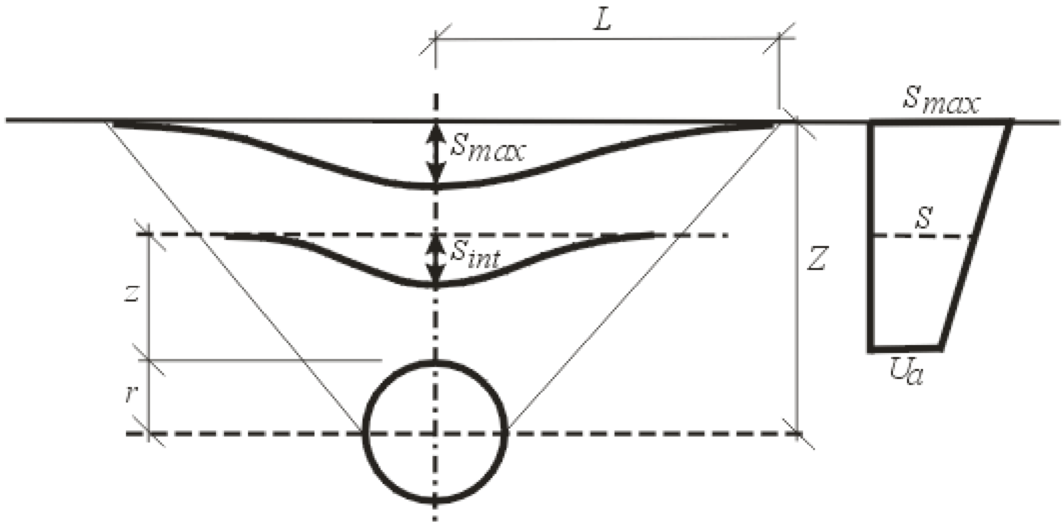


Figure 2.4: Analysis of subsidence trough at a depth

The width of subsidence trough at an overburden l is provided in Equation 3.17 as:

$$l = \frac{(L-r).(z+r)}{Z} + r \tag{3.17}$$

where:

L - length of subsidence trough at terrain surface

r - excavation radius

Z - depth of center point

z - analysis depth

The values l and S are then used to determine the shape of subsidence trough in overburden above an excavation.

2.5 Numerical Methods

Application of the analytical solutions is restricted when the variation of stress magnitude is significant with depth from the tunnel crown to the invert, such that assumptions made in the analytical solutions are not valid. Then, numerical method can be used to simulate support-ground interactions.

Numerical modeling has been driven by a perceived need from the tunneling industry in recent times. It has led to large, clumsy and complex numerical models. Properly performed numerical modeling will lead engineers to think about why they are building it, why build one model rather than another, and how the design can be improved and performed effectively.

Starting with the 1960's the last fifty years have lead to a significant development and advance in the application of numerical methods to tunnelling. Whereas in the beginning of its development, numerical analysis as a design tool was often criticized, nowadays the increase of computer capacity has caused a revolution within the field of tunnelling.

There are no significant tunnelling projects any more, which are carried out without the support of full numerical analyses. No doubt, simlified methods still play an important role and they can not be omitted, as they reflect both tunnelling tradition and experience. But the days are gone in which tunnel design was based on experience, intuition and analytical solutions of simple continuum models alone. Todays tunnelling engineers are provided with a wide range of various modern numerical tools: Finite Element Method, Finite Difference Method, Boundary Element Method, Discrete Element Method, etc. Cumbersome data input and viewing of calculation results may soon be remembered as a thing of the past, as modern user-friendly data pre-and post-processing tools are being developed. Automatic mesh generation and colored output graphs make such calculations even more attractive to the engineer. Thanks to powerful computer capacity and user friendly software, numerical analyses that once took weeks are being performed within a few days and in future within a few hours. The advantages of numerical analysis are obvious. Both complex material behavior and boundary conditions can be taken into account, whilst parameteric studies to improve the design can be easily carried out. The proper use of numerical tools requires sufficient background knowledge, not only in geotechnical engineering, but also a good basic understanding of the numerical method itself. Unlike analytical solutions, results from numerical analyses often can be hardly verified and only good engineering judgement may estimate whether or not results can be believed as plausible. On the other hand, the need for a high specialization in numerical analysis is often a communication hinderance between designers and analytical modelers.

The most relevant numerical method for tunnelling applications is the Finite Element Method (FEM). The method has been presented throughout the literature and detailed descriptions are available e.g. by Zienkeiewicz and Taylor (1991) or Bathe (1982). In the following a contribution will be given to the modelling of tunnels with the help of the FEM.

2.5.1 Basic terms and modelling aspects of FEM tunnel analysis

In order to provide a better comprehension, some basic terms and modelling aspects of FEM tunnel analysis will be introduced in Figure 2.5:

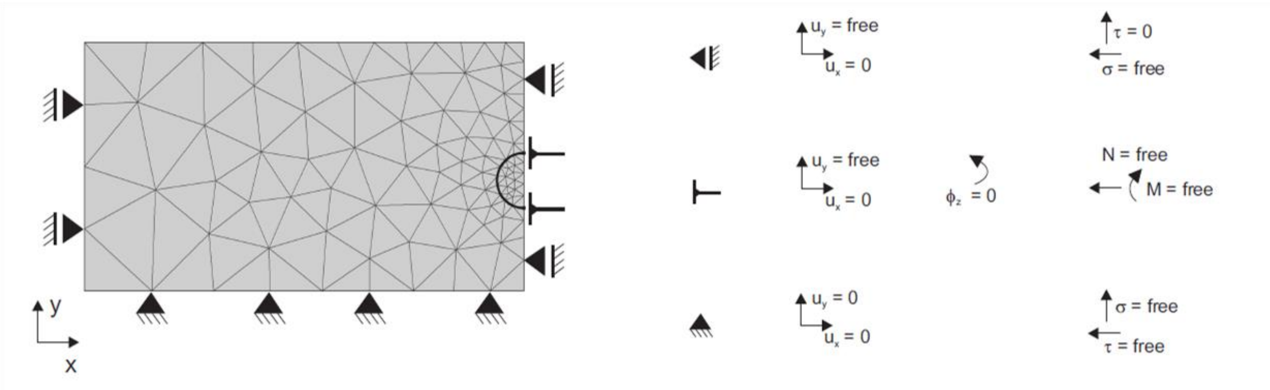


Figure 2.5: Boundary conditions of bottom, surface and vertical boundaries of symmetrical half

2.5.2 Boundary conditions

The Finite Element Method is used to solve initial and Boundary Value Problems. For the vertical boundaries at the nodes, the vertical displacement u_y is left free and the horizontal displacement u_h is restrained, allowing only for a normal stress and no shear stress τ. The condition that the vertical shear stress must be zero also satisfies the formulation of the statical symmetry at the right vertical mesh boundary. For the tunnel shell elements, the additional condition applies that the rotation ϕ_z must be zero, allowing also for a bending moment to take place.

The bottom mesh boundary has total fixities restraining both horizontal and vertical displacements. Hence normal stresses as well as shear stresses may occur. The fact that both vertical and horizontal displacements are restrained is related to the specific application of the FEM to geotechnical problems. For deeper ground layers in reality one tends to find a considerable increase of the stiffness, and therefore deformation will hardly occur. Moreover, when approaching stiff and strong rocklike materials at more shallow depth, it is logical not to extend the deformation analysis into such layers. Considering the bottom boundary to represent such a stiff ground layer, it is appropriate to restrain both vertical and horizontal displacements. The upper horizontal boundary has no fixities at all and is left free to displace.

2.5.3 Steady-state settlement trough

One of the basic terms used for three-dimensional tunnel analysis is the statement that for a representative prediction of settlements, one should refer to the steady-state of settlements. The sufficient FE-mesh dimensions for the steady-state solution shall be used.

2.5.4 Tolerated error

When using elastoplastic constitutive models in a FE-deformation analysis, the equations to be solved become non-linear. In order to solve such nonlinear equations an iterative solution procedure is needed. After each iteration the equilibrium error is computed. The criterion whether the equilibrium error remains within acceptable bounds is linked to the definition of the tolerated error. When adopting a high value of the tolerated error, the convergence of the iterative procedure may become faster, but at the same time the accuracy of computational results is reduced. On the contrary, when the tolerated error is chosen too low, computer times may become excessive.

The influence of the tolerated error on the results of FE-tunnel analysis has been investigated. It was found that surface settlements requirer a relatively tight tolerated error in order to give accurate solutions. On the contrary it was observed that structural forces are less affected by the accuracy check, requiring only a relatively relaxed tolerated error.

2.5.5 FE-mesh dimensions

Both longitudinal distributions of surface settlements and structural forces require a large number of excavation phases to arrive at a representative steady-state solution, being no more influenced by the mesh boundaries. Similar to the influence of boundary conditions on the development of longitudinal surface settlements, the transverse settlement trough may as well be affected by the model boundaries. When involving too small mesh widths, such that the computed transverse settlement trough does not fit into the selected geometry, the results will be significantly affected by the displacement boundary conditions. Because the vertical boundaries are left free to displace in the vertical direction, the choice of insufficient mesh widths will cause too large settlements. To avoid the influence of boundary conditions on the results of FE-analysis sufficient mesh dimensions have to be considered.

For the choice of sufficient mesh dimensions Meissner (1996) states that the stresses at the model boundaries should not be influenced by the tunnel excavation, recommending to use (4 - 5) × D from the tunnel center line to the vertical mesh boundaries and (2 - 3)× D from the tunnel center point to the bottom boundary. Bliehm (2001) considers a different criterion relating the choice of mesh dimensions to the magnitude of strains. According to his recommendation the dimensions should be chosen such that the strains perpendicular to the mesh boundaries do not exceed 0:005% after construction of the tunnel.

2.5.6 The influence of the mesh coarseness

If properly formulated and implemented, the FE-solution converges to the true solution when the number of degrees of freedom is increased. In such 3D analyses the consume of computer resources can increase rapidly and one would thus like to reduce the number of nodes to a minimum keeping computational results within a certain margin of accuracy. Therefore parametric studies of the mesh coarseness are required.

For 2D surface settlements it was observed Moller (2006) that they require a relatively fine local mesh coarseness, whereas 2D structural forces are little affected, requiring only a relatively coarse local mesh. On the contrary it will be shown that 3D structural forces are most sensitive to a variation of the number of nodes per round length, requiring relatively fine 3D meshes with a high number of elements.

2.5.7 The initial stress

When comparing tunnel analysis to problems of structural engineering, one main difference is the fact that a significant part of the structure, the ground is not in a stress free situation before the tunnel is built. In tunnelling the ground is already subjected to relatively high initial stresses whereas in structural engineering, stresses for which the structure has to be designed are only developing as a result of (external) loads. In structural engineering a stress free situation may be considered as a starting point and the final stresses may be obtained as a result of numerical analysis. On the contrary, in tunnelling (and also in geotechnical engineering problems in general), initial stresses of considerable magnitude have to be considered as a starting point. Surface settlements and structural forces to be obtained from numerical analyses are by a large portion determined by the anisotropy of the initial stress field.

In order to account for the relatively high influence of initial ground stresses, a reasonable estimate of both their magnitude and orientation is needed. The natural initial stress distribution in the ground is mostly unknown and it is influenced by quite a number of factors. Tectonic movements, thermic effects, creep or weathering are only a few such factors and it often is very difficult to evaluate or even measure initial stresses. Thus, for assessing initial stresses some reasonable assumptions and/or approximations are needed.

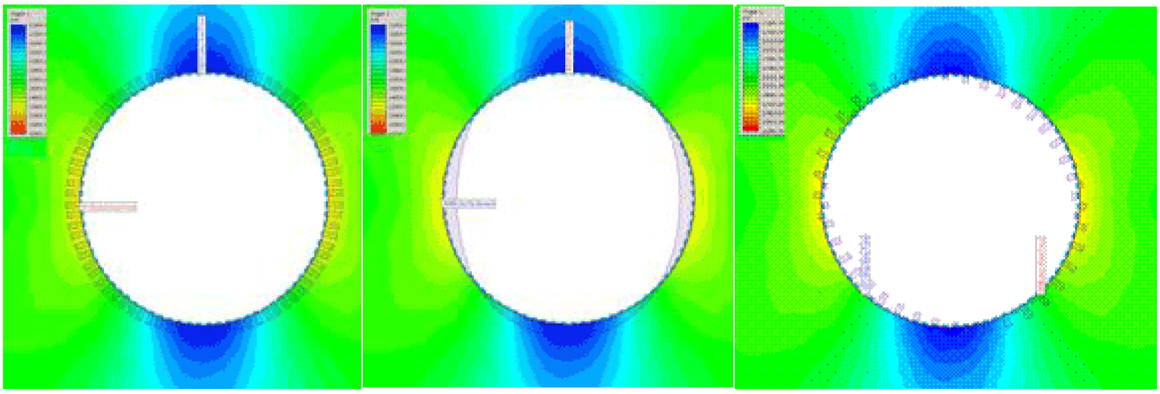
2.5.8 An outline of the steps for performing a numerical analysis

An outline of the steps recommended for performing a numerical analysis for tunneling is as follow:

- Step 1: Define the objective of the numerical analysis
- Step 2: Select 2D or 3D approach and appropriate numerical software
- Step 3: Create a conceptual drawing of the analysis layout
- Step 4: Create geometry and finite element meshes
- Step 5: Select and apply boundary condition, initial condition and external loading
- Step 6: Select and apply constitutive model and material properties

- Step 7: Perform the simulation for the proposed construction sequence
- Step 8: Check / verify the results
- Step 9: Interpret the results

For the analysis of tunneling in soil, continuum analysis is generally accepted, where the domain can reasonably be assumed to be a homogeneous media. The continuum analysis includes Finite Element Method (FEM), Finite Difference Method (FDM), and Boundary Element Method (BEM) which indicates the section loads like Figure 2.6



(a) (b) (c)

Figure 2.6: Loads on a Concrete Lining Calculated by Finite Element Analysis: (a) Axial Force, (b) Bending Moment, (c) Shear Force

3. MARMARAY PROJECT BC-1, TBM-1 TUNNELS

3.1 A Brief Summary of Marmaray Project BC-1, TBM-1 Tunnels

The Marmaray Project is a modern fast rail-track transportation scheme of 76.3 km connecting the European and Asian sides of the Istanbul City underneath the Bosphorus at a depth of 60 m. The objective of the Project is to establish a transportation capacity for about 150.000 passengers per hour in both directions and to reduce the present total travelling time of 185 minutes by nearly half.

13.6 km of the total length of 76.3 km is a new underground route within the scope of the Engineering-Procurement-Construction (EPC) Contract BC1, namely Railways Bosphorus Tube Crossing-Tunnels and Stations, which is constructed at the undertaking of the Joint Venture of the Contractors TAISEI from Japan, GAMA and NUROL, two leading worldwide known Turkish Construction Companies. The BC1 Project, designed for a lifetime of more than 100 years, consists of a twin route of 1.4 km immersed tube tunnel, 9.7 km TBM tunnels, 2.0 km NATM tunnels, 0.5 km of cut-and-cover stations (no 2), a tunnel station (no 1) and surface stations (no 2).

3.2 TBM-1 Tunnels

The total route of the BC1 Contract is shown in the following longitudinal profile in Figure 3.1, where the twin TBM-1 tunnels, namely track T2 and T1, are located along a length of 2430 m between the Yedikule and Yenikapı sites, a highly overcrowded urban area of the Istanbul City on the European side:

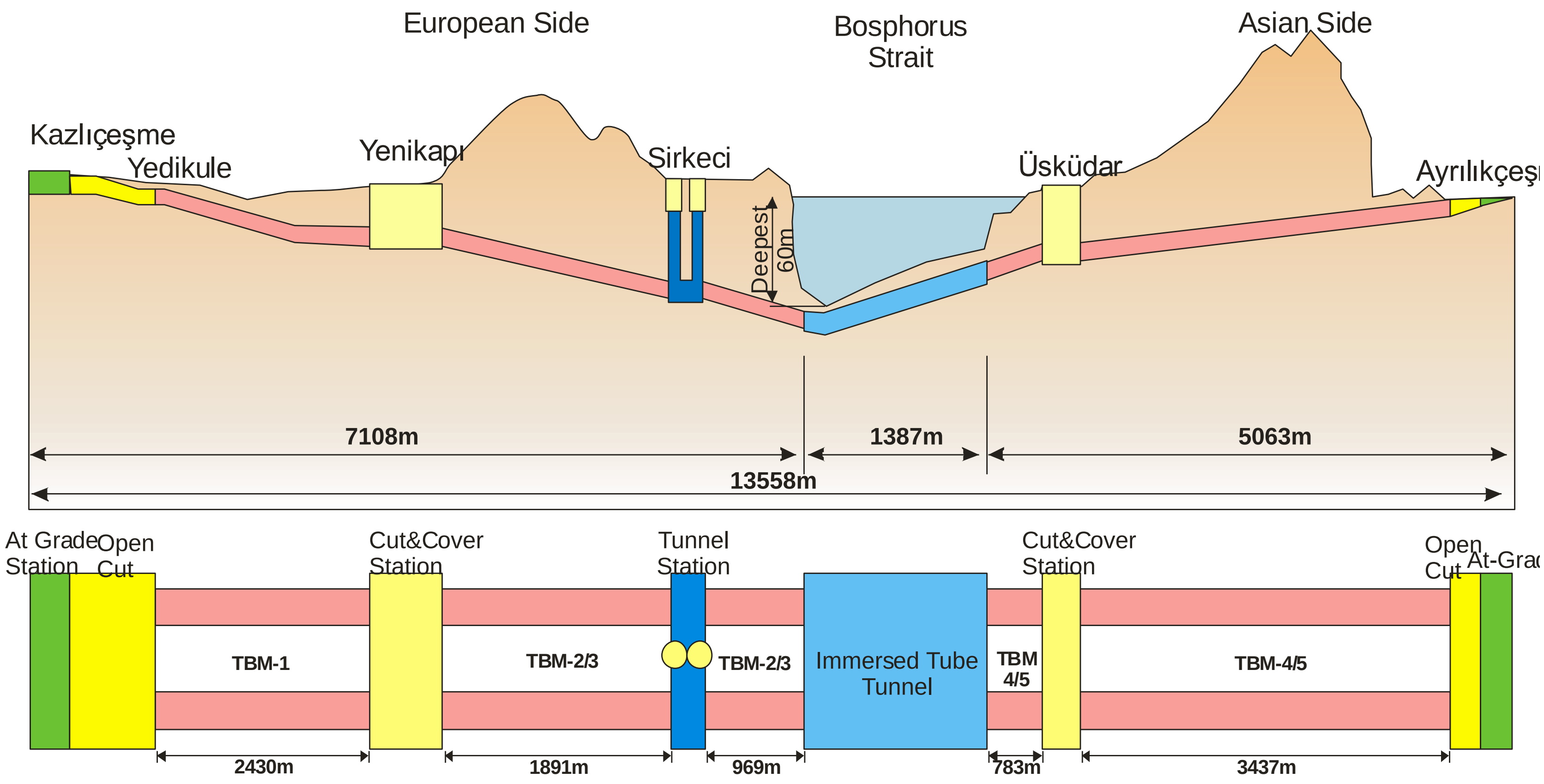


Figure 3.1: Project Route

The urban tunneling was performed under a vulnerable built environment consisting of many old and ruined buildings, protection of which against the tunnel induced deformations is a legal and contractual liability on the Contractor. In this context, the design was revised, even after commencement of tunneling, by shifting the tunnel alignment outside of the settlement area as much as possible so that the interface with the built environment could be decreased to the greatest possible extent; the number of buildings within the potential influence zone of the TBM-1 tunnels was decreased from 550 to 158. Plan view and longitudinal profile of the revised tunnel alignment is shown herein below in Figure 3.2:

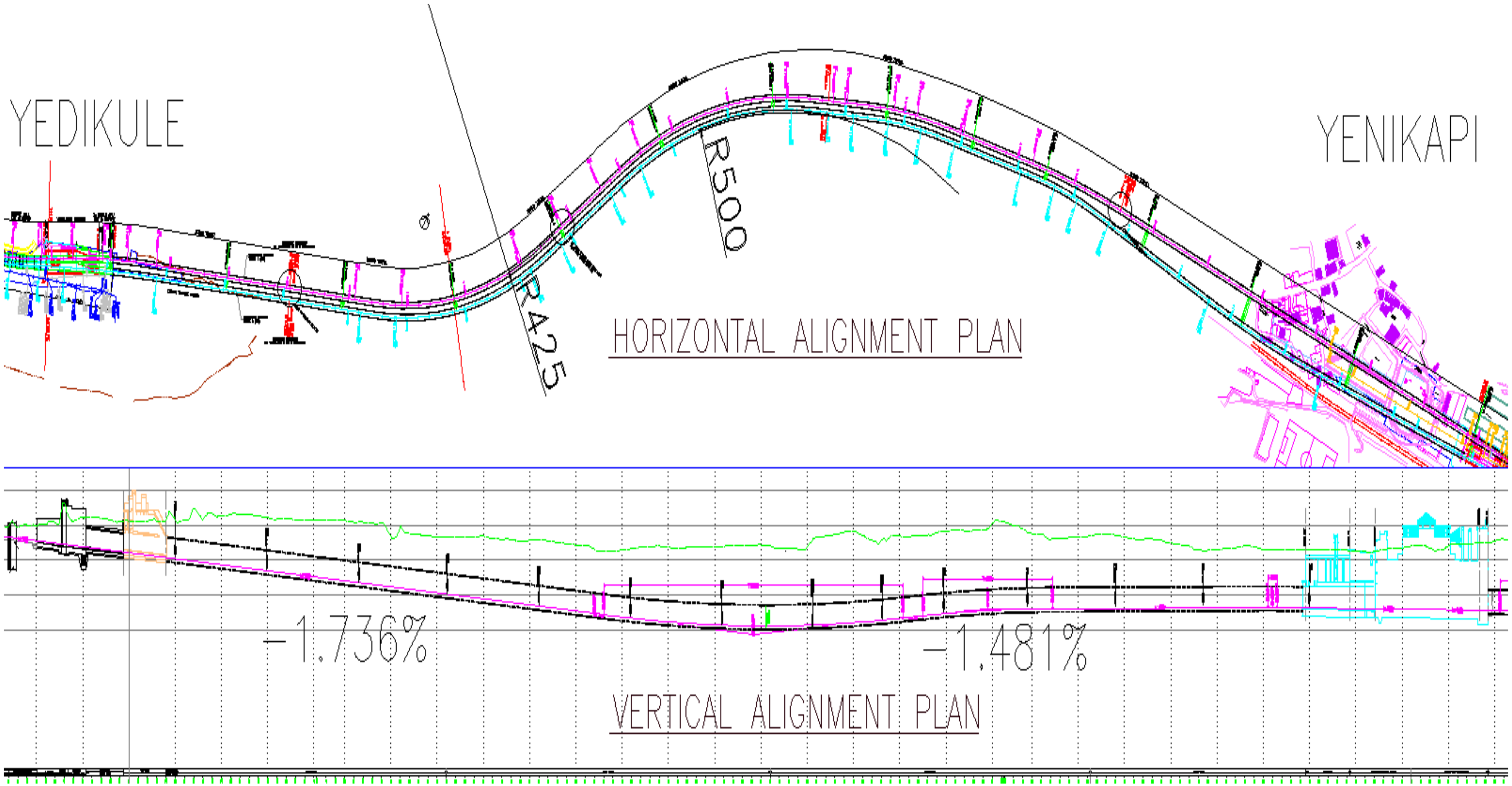


Figure 3.2: Tunnel Alignment

The tunnels, bored at an excavation diameter of 8 m, are approximately 14 m apart from each other from axis to axis and connected by cross connections at about every 200m distance. The tunnels are lined with precast concrete segment rings of 1.5 m length and 320 mm thickness. The peripheral voids between the segments and the ground are backfilled with the first stage contact grouting. A sump pit is provided to each tunnel for dewatering. A typical cross section of the tunnels through a cross passage and the sump pit is shown herein below in Figure 3.3:

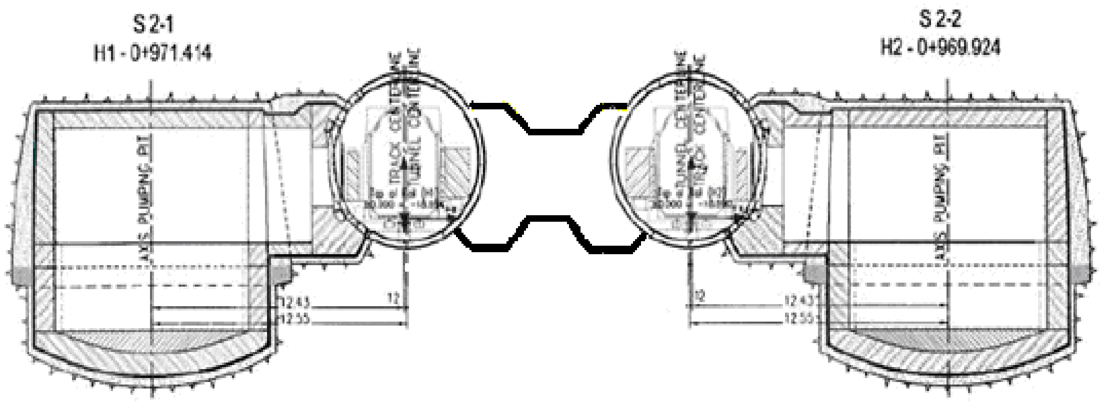


Figure 3.3: Tunnel cross section

3.3 Geology and Ground Conditions

The tunnel route consists, from younger to older, of manmade fill (F), actual marine sediments (SM3/SC3, Kuşdili formation), marl-limestone-clay (LMST/CH2/MRL, Bakırköy formation), green stiff to very stiff plastic clay (CH3, Güngören formation) with grey-beige silty fine sand layers (SM2) as mentioned in Figure 3.4. The geology at the last 300 m of the route is discontinuous and complex, likely of an old land slide or depositional process, where excessive settlements did occur during tunneling.

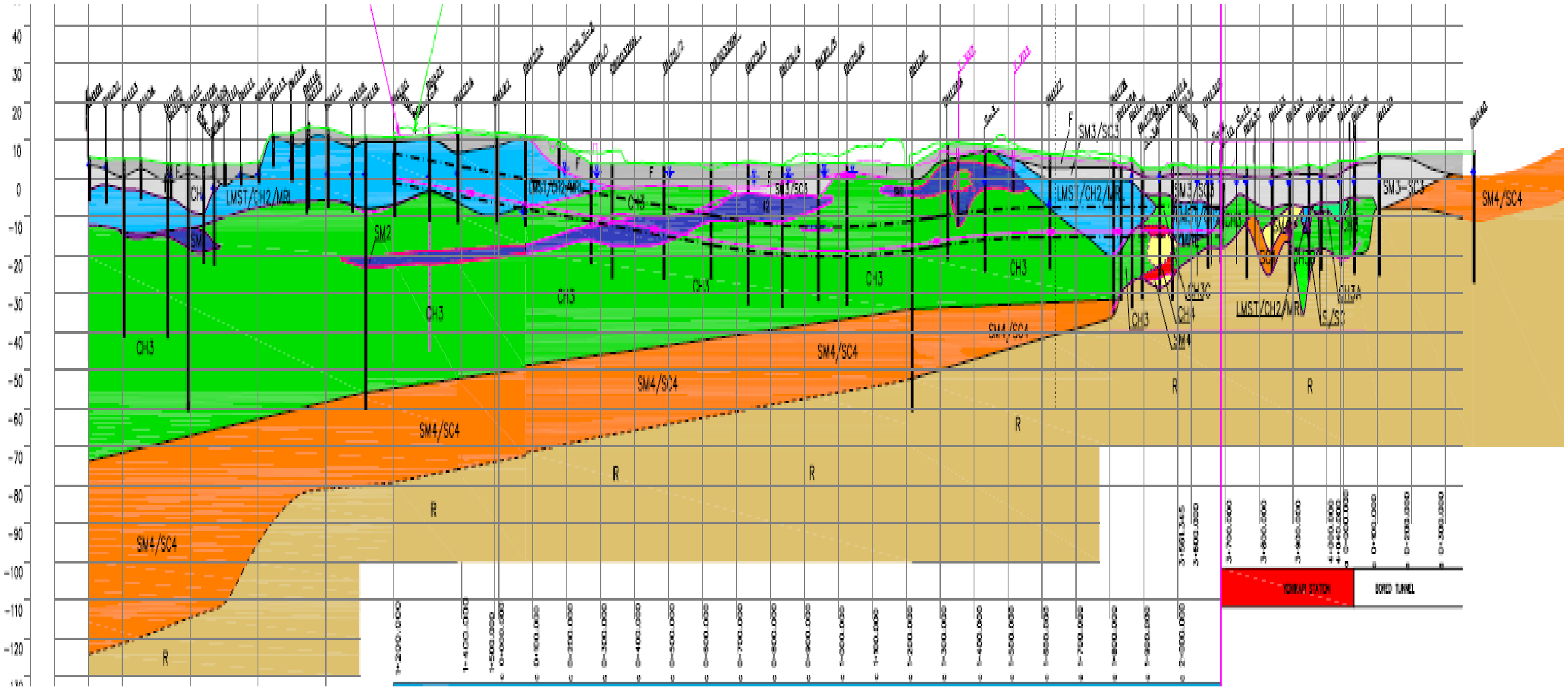


Figure 3.4: Geological profile

The overburden above the tunnel varies from 8 to 15 m whilst the groundwater is maximum 20 m above the tunnel invert level. The tunnel passes through the marl-limestone-clay layer at both ends, except the last 300 m. In between, the tunnel proceeds through the impermeable green stiff clay, which is intersected however by a water bearing sandy layer lying from 1+810 to 3+460 at a thickness of 5 to 10 m respectively. The interferences of this sandy layer with the tunnel route resulted in the following geological incidents:

i. At the intersection of the invert of the Tunnel T2 at 1+860, a water ingress of about 15 l/sec occurred into the tunnel together with sandy material,

ii. At the crown of the Tunnel T2 along the reach of 2+210-2+260, high settlements occurred during tunneling.

3.4 TBM Equipment

A LOVAT RME314 Series 21900 EBP Type Mixed Face TBM equipment was used for boring of the TBM-1 Tunnels as mentioned in Figure 3.5:

Figure 3.5: Tunnel Boring Machine – TBM-1

The basic machine properties are briefly summarized herein below as mentioned in Table 3.1:

Table 3.1 : TBM-1 properties

No	Item	Description
1	Machine Type	Lovat RME314 Series 21900/EPB Type Mixed Face EPB (Closed – Pressurized)
2	Excavation Type	8 cm/min max. penet. rate Screw conveyor
3	Cut Dia.	Ø7,994 mm
4	Bore Dia.	Ø7,969 mm
5	Shield Dia.	Ø7,956 mm
6	Cutting Head Power System	7x300 kW Water Cooled Electric Motors coupled to Variable Disp. Pumps, Total Power available to cutting head: 2,100 kW
7	Cutting Head Drive System	11 nos Hydraulic Motors, Variable Disp.Type, complete with Gear Reducers, cutting head operation bi-directional with full hydraulic power and variable speed
8	Max.Torque	2,045 t.m @ 1.00 RPM @ 248 bar
9	Min.Torque	1,023 t.m @ 2.00 RPM @ 248 bar
10	Peak Starting Torque	2,556 t.m @ 310 bar
11	Propulsion System	25 cylinders of each 255 tons, 6,375 ton
12	Articulation Angle	2° in any direction

3.5 Precast Segments

The segmental linings consist of 6 equal size precast concrete segments and 1 key segment connecting altogether to form rings as shown in Figure 3.6 below. The thickness of the segments is 320 mm.

The basic concrete properties of the precast segments are as follows:

i. Characteristic cylinder compressive strength : f_{ck} = 50 N/mm2

ii. Mean value of the tensile strength : f_{ctm} = 4.1 N/mm2

iii. Value of secant modulus of elasticity : Ecm = 37 kN/mm2

iv. W/C ratio less than 0.35

v. Minimum cement content: 275 kg/m3.

Figure 3.6: Segment Stock Area

3.6 Tunneling under the EPB Method

The tunnels were driven by an Earth Pressure Balance (EPB) Tunnel Boring Machine, namely TBM-1, which is able to operate in a closed-face-mode, semi-closed and open mode. An EPB-shield can provide continuous support of the tunnel face under the closed-face-mode, which is essentially required for the control of tunnel induced deformations.

The TBM was launched from the Yedikule shaft that tunneling proceeded in the direction of Yenikapı site. After completion of excavation of T2, the TBM was disassembled at the Yenikapı site and brought back to the Yedikule shaft for boring of the tunnel T1. The first 100 m of the tunnels was bored as an initial excavation, for which the thrust is taken from the steel push frame temporarily erected at the Yedikule shaft. The TBM operates in two phases; tunnel boring and tunnel lining as explained herein below:

Tunnel boring proceeds through the drive of the cutting-head in successive stages of 1.5 m lengths. As the cutting-head rotates, propulsion cylinders push the TBM forward causing the cutting tools on the cutting-head to excavate the ground from the tunnel face. Excavated material (muck) passes through the openings in the cutting head into the cutting-head chamber. The screw conveyor extracts muck from the forward shell and dumps it onto a trailing conveyor, which in turn discharges onto the continuous conveyor, which operates for transportation of the muck outside of the tunnel as shown in Figure 3.7:

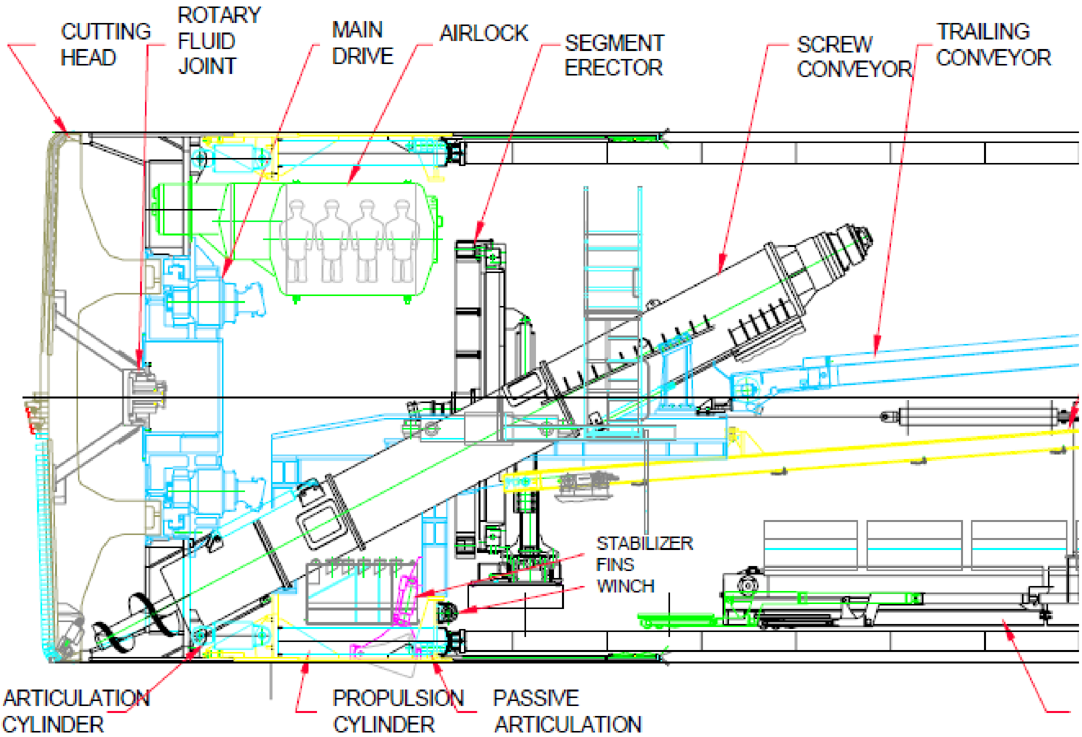


Figure 3.7: Main parts of TBM

3.7 Tunnel Lining

Tunnel lining is performed in two stages; segment erection and segment fixing. Segment erection is performed inside the trailing shield by means of a segment erector, which places the pre-cast concrete segments into position around the tunnel. Segment fixing is performed in two parts, bolting and grouting. Bolting proceeds during segment erection. A backfill grouting is performed by a special mix of the peripheral voids between the ground and segment whilst the tunnel boring advances for the next ring. Grout passes through the trailing shield to the tunnel annulus.

3.8 Earth Pressure Balance (EPB) Method

While tunneling in unstable ground, it may be necessary to balance the pressure inside the cutting head chamber with that of the surrounding soil to provide a safe environment within the TBM and to minimize surface settlement above the tunnel. This can only be achieved when the screw conveyor is installed in position. As the TBM excavates and advances, material passes through the cutting-head openings into the sealed cutting-head chamber where it is temporarily retained to maintain pressure on the tunnel face, which is known as the earth pressure balance (EPB) method. The principle of the EPB method is illustrated in the following Figure 3.8:

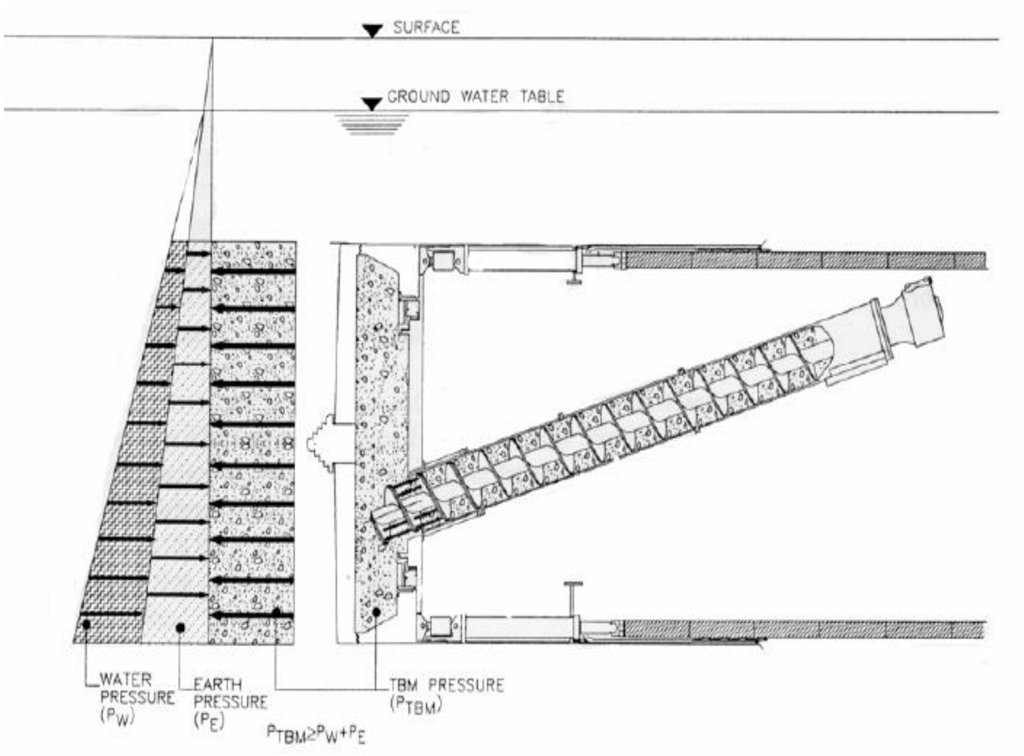


Figure 3.8: The principle of EPB Method

3.9 Occurrence of Settlement/Heave on the Ground

The closed or full EPB mode provides continuous pressure-balanced support to the tunnel face. The TBM is arranged with the screw conveyor for full EPB operation. Face pressure is maintained by balancing propulsion cylinder thrust with the rate of material extraction from the cutting-head chamber by the screw conveyor. The rotation speed of the screw conveyor determines the rate of excavation of the TBM. Therefore, the face stability in the EPB-shield is dependent upon the amount of soil removed by the screw conveyor being proportional to the amount of excavated soil entering into the mixing chamber. The screw conveyor rotation speed is controlled in proportion to the shield advancing speed in order to maintain the balance of these amounts. It is difficult however to precisely measure actual amounts of soil, in particular soil removed by the conveyor, for the following reasons :

i. Conveyor volumetric efficiency varies depending on the operational conditions and varying density of materials.

ii. Measurements of both shield advance and conveyor rotation speeds include tolerances.

iii. As the time elapses, the difference between the excavated soil and discharged soil amounts is expanded by errors in volumetric efficiency and measurement tolerance.

The unbalanced face pressure thereof causes settlement or heave as the following:

a. EPB lower than the ground pressure results in a volume loss of the ground which is observed as a settlement on the surface,

b. EPB greater than the ground pressure causes heave on the ground surface.Settlement Analysis of TBM-1 Tunnels by Empirical Methods

3.10 Settlement Analysis

It is assumed that the tunnel induced settlement is of a bell-shaped curve (Gaussian Error Function). The volume of the settlement trough is equal to the total volume of the “lost ground” (volume loss) during tunneling as mentioned in Figure 3.9:

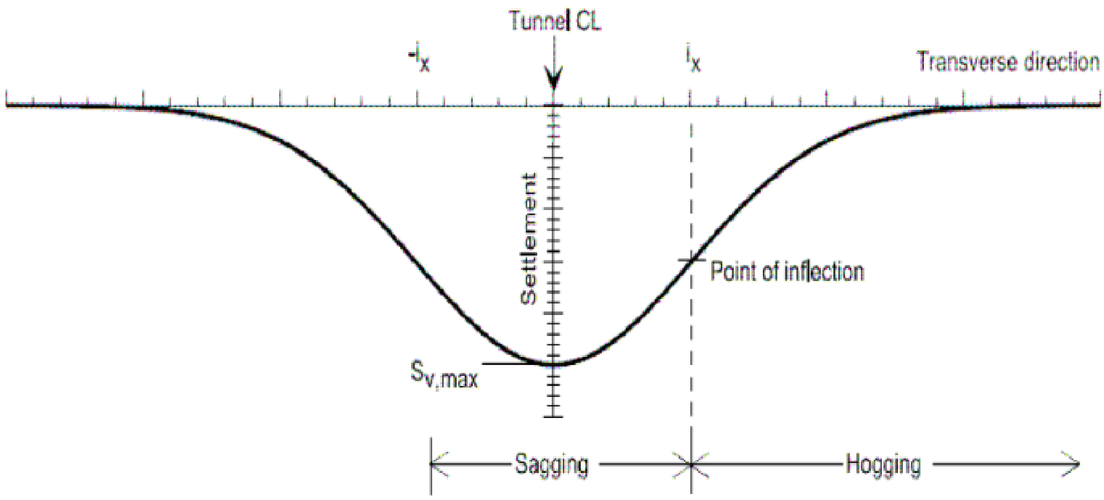


Figure 3.9: Bell-shaped settlement curve

The area encompassed by the settlement contours of 0.1 % inclination at each end of a settlement trough along the length of the tunnel is defined as the zone of potential influence, beyond which the influence on the structures is deemed to be negligible. The zone of potential influence of TBM-1 Tunnels, determined as above, corresponds to a strip-wise area of about 50X2430 m.

The settlement analysis is performed in accordance with the adapted method in order to calculate the tunnel induced ground deformations on the selected locations, i.e. foundation of the buildings:

i. Maximum vertical and differential settlements,

ii. Maximum horizontal and differential displacements,

iii. Maximum inclination (%),

iv. Horizontal strain and

v. Angular distortion (%).

The settlement analysis of the TBM-1 tunnels was performed on the basis of the parameters assumed by experience such as a volume loss of 1% and a soil coefficient of 0.4-0.7.

3.11 Predicted Settlements by Empirical Method

The predicted maximum surface settlements have been reported by Marmaray Project Contract BC1 - TGN - BC1 - 80 - 00 - 00 - 00 - 2483 # 00 - 75 A1 Bored Tunnels Between Yedikule and Yenikapi Risk Assesment and Settlement Analysis VO75 Addendum 1 Section From Km c0+800 (km2+312)to Km 3+680 are as mentioned in Table 3.2:

Table 3.2 : Empirical Settlement

1+200	32 mm
1+540	22 mm
1+680	23mm
2+060	30 mm
2+400	22 mm
2+710	26mm
2+920	30mm

For the predicted settlements, an empirical approach is used. Within these methods it is assumed that the tunnel settlement trough is bell shaped (Gaussian error function) and that the volume of the settlement trough is equal to the total volume of "lost ground" during tunnelling ("volume loss"). The volume loss normally is defined as a percentage of the excavation diameter of the tunnel. The calculated settlements troughs for adjacent tunnels are assumed to be additive.

Surface settlement

$$S, v(X) = Sv,max -e z Ix z \qquad (3.18)$$

Sy,max maximum settlement a tunnel axis [m]
x distance to tunnel axis in transverse direction [m]
ix parameter for den inflection point in transverse direction [m].

3.12 Settlement Analysis of TBM-1 Tunnels by Analytical Solutions

No analytical solution have been carried out for TBM-1 Tunnels. Analytical solutions have not been preferred as they are not practical nor reliable when compared with Emprical and Numerical Methods.

3.13 Settlement Analysis of TBM-1 Tunnels by Numerical Methods

In numerical calculations and analysis for ground surface settlement during tunnel excavation, the three dimensional FEM analysis software Plaxis 3D Tunnel 2.4 is used.

Plaxis software is based on the finite element method and intended for 2-Dimensional and 3-Dimensional geotechnical analysis of deformation and stability of soil&rock structures, as well as groundwater and heat flow, in geo-engineering applications such as excavation, foundations, embankments and tunnels.

Plaxis 3D Tunnel Software which has been used in this study offers improved and extended options to create circular and non-circular tunnels composed of arcs, straight lines and corners. Plates and interfaces may be added to model the tunnel lining and the interaction with the surrounding soil. Fully isoparametric elements are used to model the curved boundaries within the mesh. Different practical methods are implemented to analyse the deformations that occur due to the construction of the tunnel.

In Plaxis 3D Tunnel Software it is possible to add a thick and massive lining that is composed of volume elements rather than plates. To simulate volume loss during the construction of the Bored Tunnel, a contraction has been applied.

The ground parameters for FEM analysis have been used as follows in Table 3.3:

Table 3.3 : Yedikule to Yenikapı St. (STA 1+200 to STA 2+400)

Soil Classification	Unit Weight (kN/m3)	Modulus of elasticity (kN/m2)	Poissons's ratio
Fill	18	8000	0.30
Fat Clay (CH-2U)	19	24200	0.40
Fat Clay (CH-3U)	18	18900	0.40
Fat Clay (CH-3L)	18	27000	0.40
Silty Sand (SM-2)	19	49500	0.30

The (USCS) Unified Soil Classification System has been used for the classification of the soils as mentioned in Table 3.4:

Table 3.4 : Unified Soil Classification System

First Letter		Second Letter	
Letter	Definition	Letter	Definition
G	Gravel	P	Poorly Graded
S	Sand	W	Well Graded
M	Silt	H	High Plasticity
C	Clay	L	Low Plasticity
O	Organic		

The numbering after the USCS Code (2U, 3L) has been only mentioned to differentiate the soil types of the project regardless of any classification intent.

Mohr-Coulomb Material Model is assumed to be used as related model parameters in Figure 3.10:

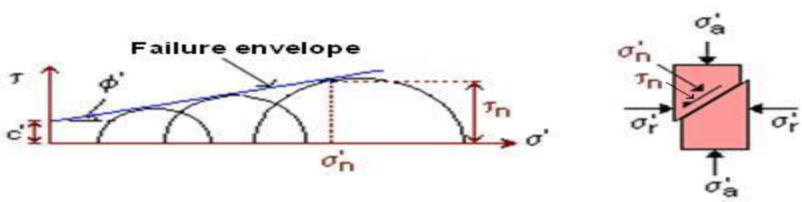


Figure 3.10: Mohr-Coulomb Failure Criterion

$$\sigma \equiv E \times \varepsilon \tag{3.18}$$

where:

σ - Strength

E – Elasticity Modulus

ϵ - Displacement

The FEM calculation briefly computes the Elasticity Moduli and Displacements while the cohesion and internal friction angle are predefined constants.

The model boundary conditions were used as follows in Figure 3.11:

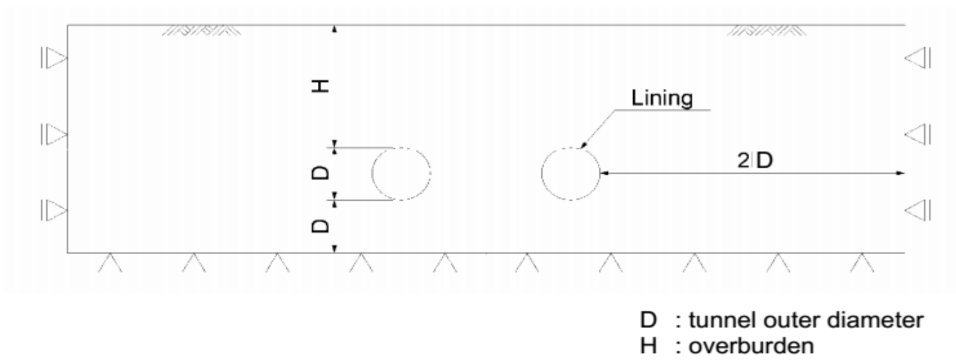


Figure 3.11: Boundary Conditions

The Planes (Advances) in the Models are summarized as follows in Table 3.5:

Table 3.5 : Planes (Advances) in the Models

Plane	Definition	Excavation &Support
Front Plane	25 m behind the Excavated Section (to keep a boundary between excavated and not excavated ground)	0
Plane A	1st Excavation	0~1,5 m
Plane C	3rd Excavation	3~4,5 m
Plane H	8th Excavation	10,5~12 m
Plane O	15th Excavation	21~22,5 m
Plane T	21th (Last) Excavation	30~31,5 m
Rear Plane	25 m ahead of the Last Excavated Section(to keep a boundary between excavated and not excavated ground)	56,5 m

The initial excavation and support stages have been modelled in below sequence in Table 3.6:

Table 3.6 : Stages carried out in Front Plane, Plane A, Plane C, Plane H

Stage	Definition
1	The ground inside the TBM Shell has been deactivated to simulate the excavation as long as 12 meters which is the TBM Shield Machine Length
2	The TBM Shell has been activated to simulate the TBM Steel Shield as long as 12 meters. (0~12 meters)
3	The Tunnel support pressure has been applied to the tunnel face at the end of above mentioned 12 meters.
4	The forces from hydraulic jacks (600 kN/m2) advancing the TBM have been applied to the clusters representing the tunnel lining.
5	The conicity of the TBM has been modelled using tunnel contractions ranging 0.5% to 0.1%

The following excavation and support stages have been modelled in below sequence in Table 3.7:

Table 3.7 : Stages carried out in Plane O, Plane T and Rear Plane

Stage	Definition
6	All the above sequences have been applied to the next 1,50 meters of excavation (between 12~13,5) and precast concrete segmental lining have been defined for the initially excavated ring (0~1,5m)
7	Above sequence have been repeated until 31,5 meters length of ground excavated and 19,5 meters length of precast concrete segmental lining have been placed.

Outputs for stages 1 to 7 for 1+200 section have been exspressed in Figure A.1 to A.9.

Outputs for stages 1 to 7 for 2+060 section have been exspressed in Figure A.1 to A.9.

Outputs for stages 1 to 7 for 2+400 section have been exspressed in Figure A.1 to A.9.

The surface settlements through excavation steps have been exspressed in Figure 3.12 and tunnel displacements at the top of the tunnel through excavations steps have been exspressed in Figure 3.13

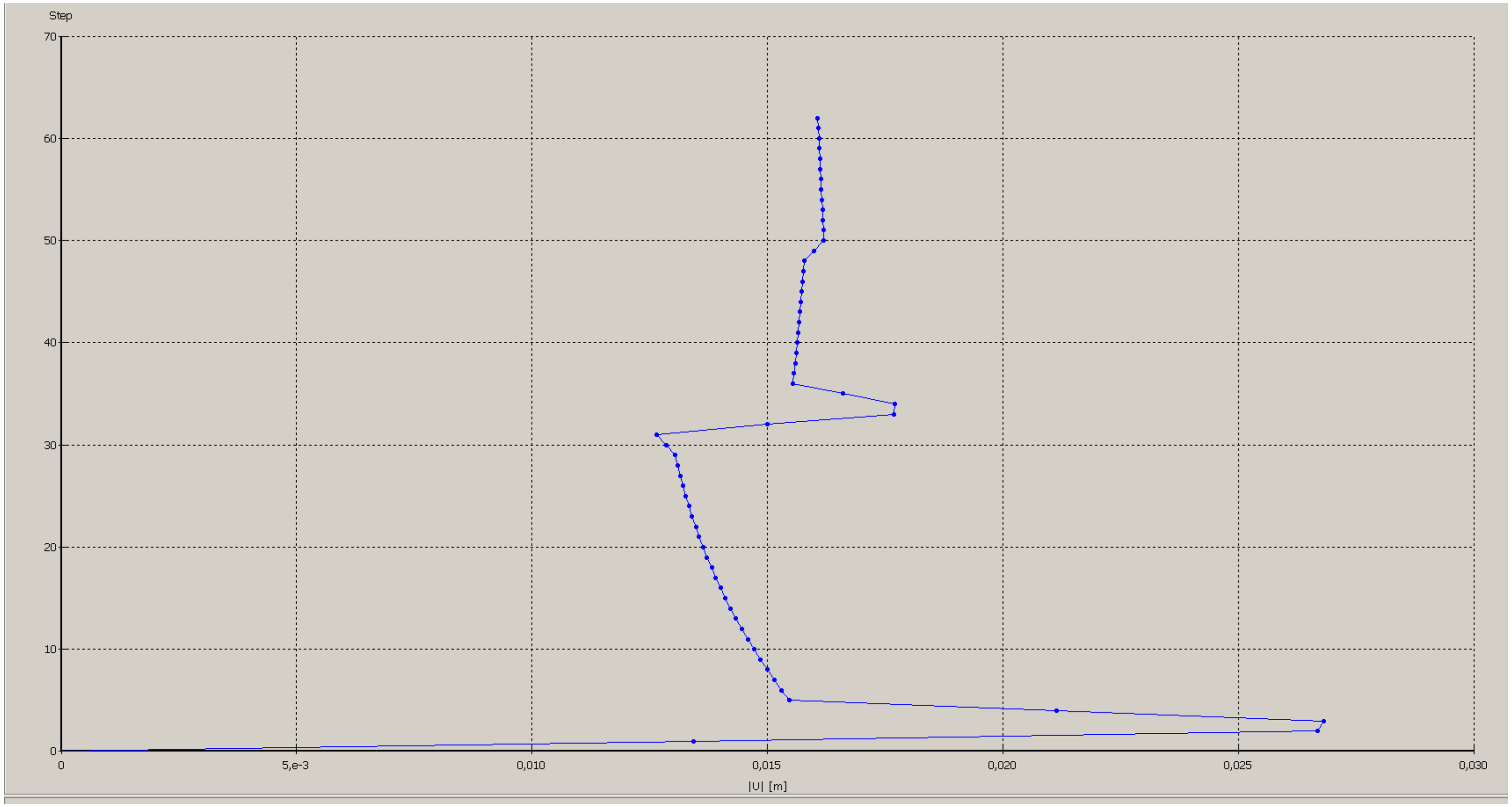


Figure 3.12: Surface Settlements Through Excavation Steps (0 start – 70 end)

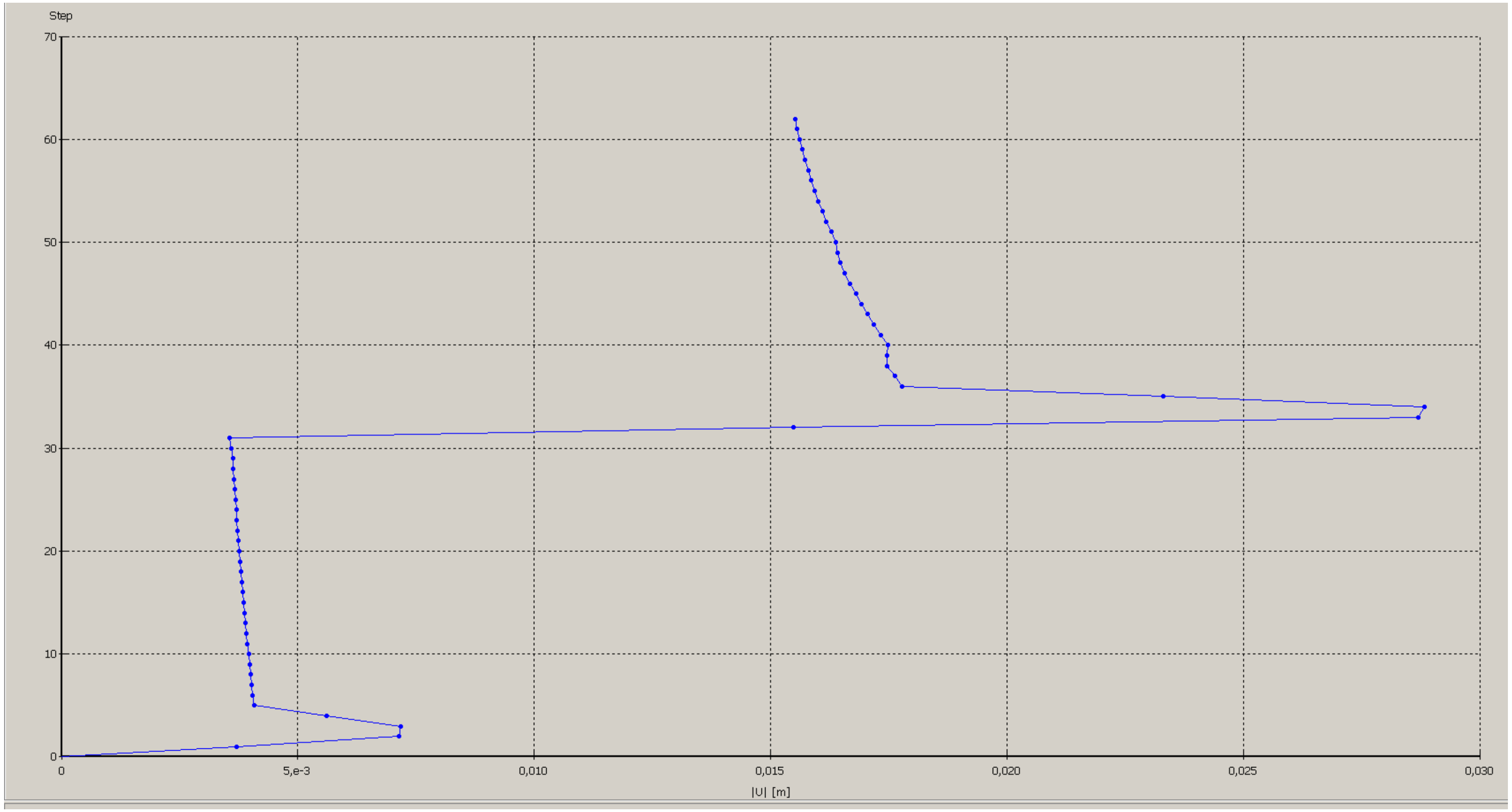


Figure 3.13: Tunnel Displacements at Top of the Tunnel Through Excavation Steps (0 start – 70 end)

Maximum Ground settlements are exspressed in Table 3.8:

Table 3.8: Analysis results for maximum ground settlement at each cross section

STA	1+200	2+060	2+400
Settlement (mm)	16	13	14
Ground slope(%)	0.18	0.10	0.11

3.14 Actual Site Settlement Measurements of TBM-1 Tunnels

3.14.1 Deformation monitoring scheme

For the tunnels, a typical instrumentation plan in accordance with the expected deformation behaviour is illustrated in the following Figure 3.14:

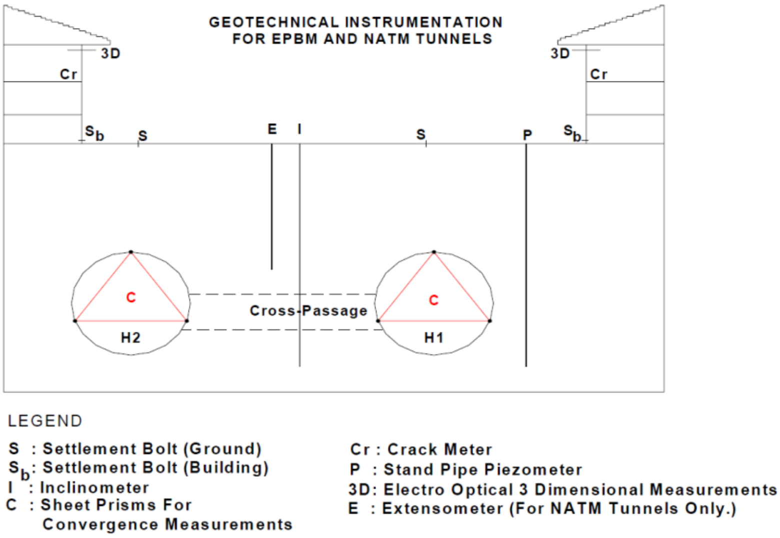


Figure 3.14: Typical geotechnical instrumentation for tunnels

Ground settlements are monitored all along the tunnel route. Additional points are formed at special points such as building locations, existing underpasses etc. A schedule of the geotechnical instruments used in the Project is shown in the following Table 3.9:

Table 3.9: Geotechnical instruments used in the Project

No	Instrument	Monitoring Item	Measuring frequency	Nos
1	Ground sett. bolt	Ground settlement	D-W-M	585
2	Building sett. bolt	Ground settlement	H-D-W-M	1956
3	3D bolt	Disp.	D-W	293
4	Inclinometer	Disp.	D-W	83
5	Extansometer	Disp.	D-W	32
6	Piezometer	Water level	D	79
7	Piezometer	Water pressure	D	18
8	Conv. bolt	Conv.	D	473
	H: Hourly	D: Daily	W: Weekly	M: Monthly

Geotechnical monitoring results are compiled in monthly reports together with a technical assessment of the measured behaviour. Monitoring data are continuously compared with pre-determined threshold values for action wherever necessary.

4. CORRELATIONS

4.1 Predicted vs. Actual Settlements

The predicted vs. actual maximum settlements (in mm) measured during tunneling are shown in the following Figure 4.1:

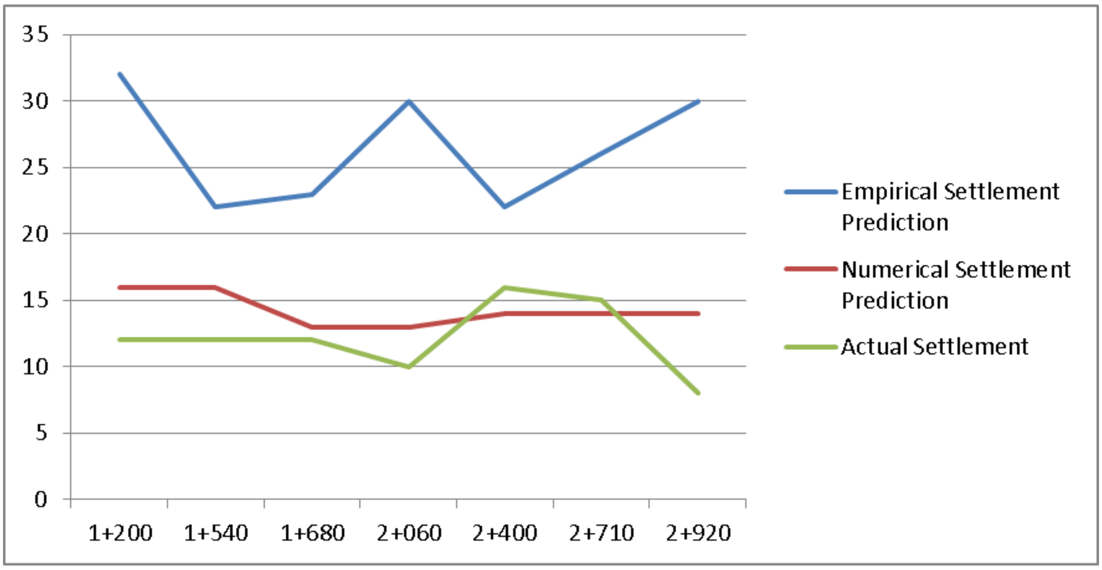


Figure 4.1: The predicted vs. actual settlements

The actual settlements are much less than the predicted maximum settlements with respect to the assumed volume loss of 1%.

The observed significant deviations of the actual settlements are indicated in the following Table 4.1:

Table 4.1: Deviations of empirical calculations with the observed settlements

Chainage	heave (+) / sett.	Eff. Area	Notes
Track-2			
1+310	+20 mm	local	heave, face pressure adjustments
1+350	40 mm	local	thickness of the fill increases
1+400	+20 to 40 mm	local	irregular heave and settlement
1+860	15 to 20 mm	local	intersection of water bearing sand layer at tunnel invert, water ingress
2+210-2+260	>20 mm	region	sand layer on the tunnel crown, water ingress
2+530	>20 mm	local	face pressure drops
2+660	>30 mm	region	sand layer above tunnel, water level drops by 1m
Track 1			
2+440-2+600	27 to 90mm	region	
2+640	78mm	local	

The local high settlements/heaves are rather irregular and attributable therefore either to abrupt variations in local geological/ground conditions and/or operational errors with adjustment of the face/grout pressure.

On the other hand, the water bearing sandy layer at the section from 1+810 to 3+460 and the last 300 m with the heterogeneous geological conditions are the main cause of the excessive settlements occurring respectively in those places.

The settlements incurred with tunneling of T1 seem to have significantly been attenuated along the last 200 m of the route, as compared to those of the T2 tunnel, which is mainly owing to the systematical ground treatment by jet grouting prior to excavation of the Turnback Tunnel (TBT) located between the TBM-1 tunnels for maneuvering of the commuter trains.

Numerical calculations and observed settlements varied ±20 mm maximum and generally estimated 10 mm more settlement than predicted. It should be also noted that mumerical model estimations were much more reliable than emprical results despite all the rough modelling conditions explained in Section 4.13

4.2 Volume Loss Measurements

The settlement prediction was made on the basis of an assumed volume loss of 1%. The actual volume loss occurring during tunneling was calculated through the actual settlements measured at 15 volume loss sections. A comparison of the predicted and actual volume loss values is shown herein below Table 4.2:

Table 4.2: Volume loss after excavation

No	Section	Chain.	Predic.	Tunnel T2	Tunnel T1-
1	Calib. Section	1+322	1	0.16 heave	0.08
2	CP2	1+406	1	0.50 heave	0.10
3	CP3	1+606	1	0.10	0.15
4	CP4	1+801	1	0.77	0.77
5	Calib. Section	1+913	1	0.34	
6	CP5	2+005	1	0.21	0.37
7	CP6	2+202	1	0.10	0.52
8	CP7	2+395	1	0.12	0.50
9	Sump Pit	2+485	1	0.17	1.04
10	CP8	2+587	1	0.78	1.33
11	CP9	2+735	1	0.29	0.98
12	CP10	2+857	1	0.12	1.36
13	CP11	3+047	1	0.90	2.32

The calculated actual volume loss is generally lower than the assumed value of 1% except the last 300 m of the route, where the assumed threshold has been exceeded significantly as highlighted herein above most likely owing to the prevailing discontinuous and heterogenous geological conditions.

5. CONCLUSIONS AND RECOMMENDATIONS

Precision of the predicted settlements becomes a very crucial issue for urban tunneling in case of a vulnerable built environment. The tunnel induced deformations are of a combined function of the geology and ground conditions as well as the operational quality of TBM utilization for tunneling. The diversity of the involved parameters particularly under complex and heterogeneous soil matrix may intricate the prediction process. However, experience from similar cases may considerably help for estimation of the ground behavior within a reasonable tolerance.

The settlement data base of the present case offers a remarkable experience for prediction of the ground behavior. The predicted settlements on the basis of a 1% volume loss confirm to the pattern of those actually measured, except some deviations occurring while passing through the sandy layers and heterogeneous geology, where it is recommended to be on the safe side with the precautionary and protection measures to be applied on the vulnerable built environment.

The numerical calculations provided much more accurate results when compared to empiricial methods as it can be seen on Figure 4.1

Empirical methods tend to exaggerate the predicted settlements, thus resulting over design of tunnel and support structures.

Numerical calculations provide much more accurate results when compared with actual results. The accuracy may be also increased by increasing calculation sections and using improved material models.

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APPENDICES

APPENDIX A: FEM Calculations for 1+200

APPENDIX B: FEM Calculations for 2+060

APPENDIX C: FEM Calculations for 2+400

APPENDIX A

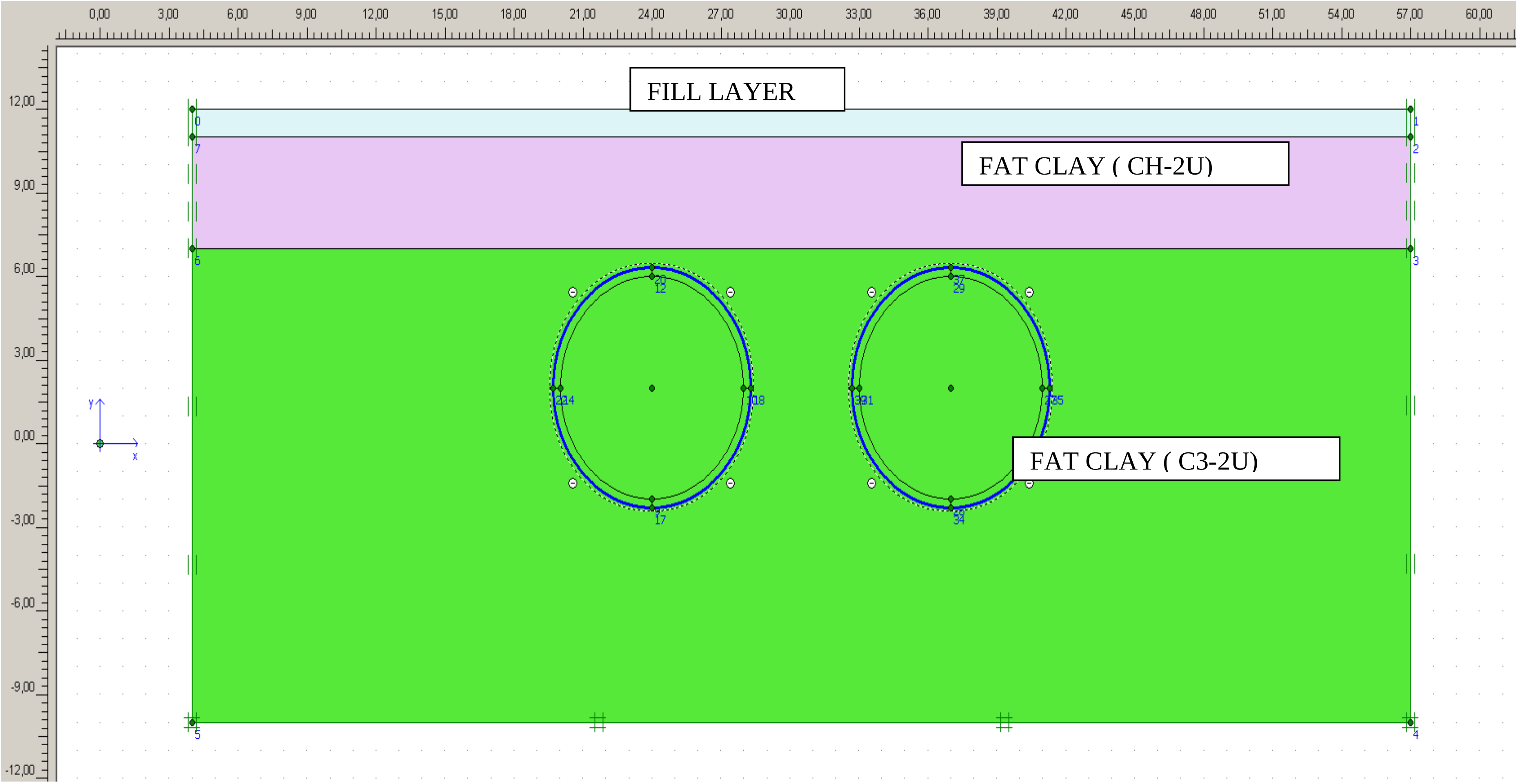

Figure A.1: 1+200 Model Layout

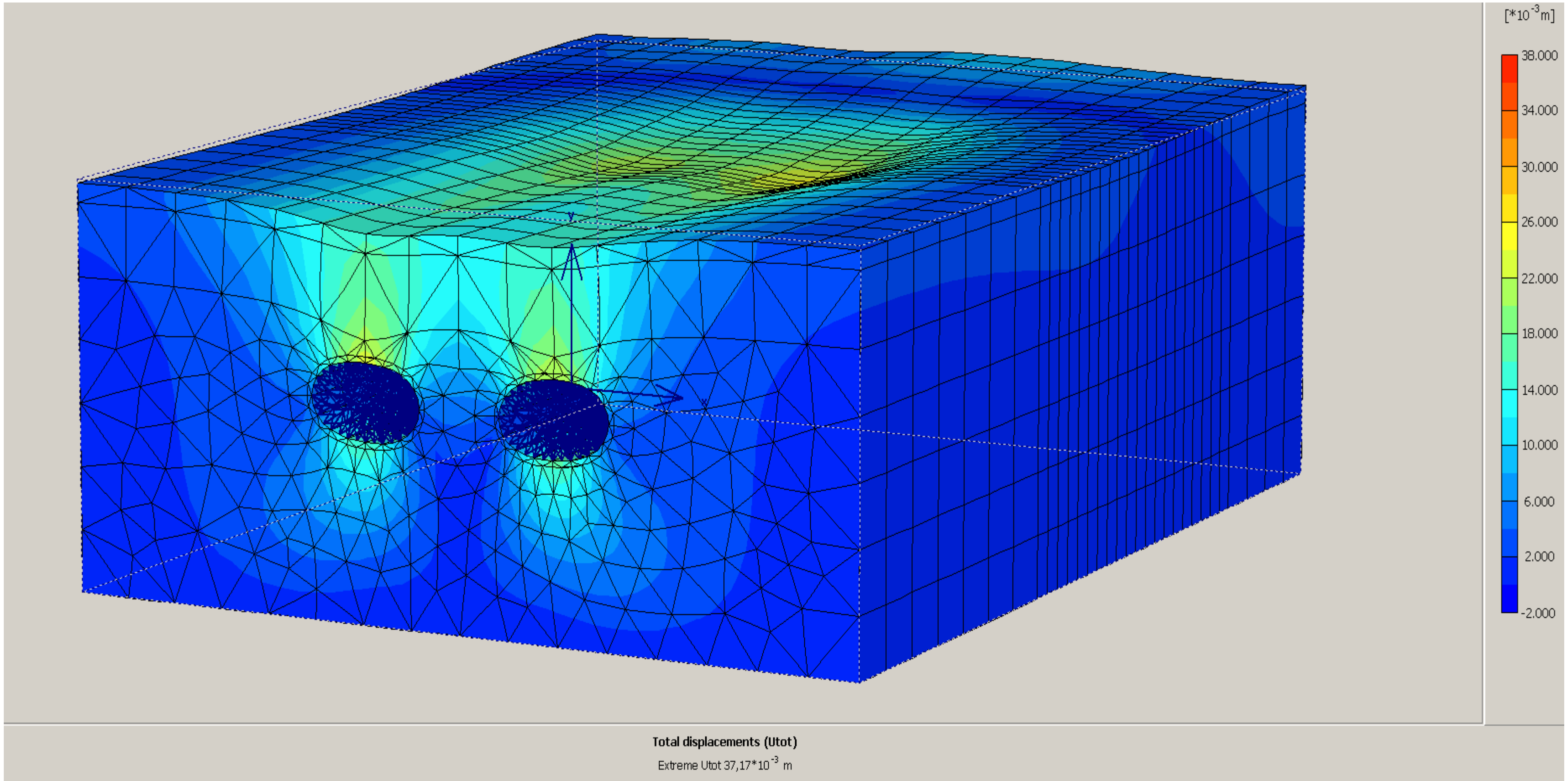

Figure A.2: 1+200 3D Total Displacements

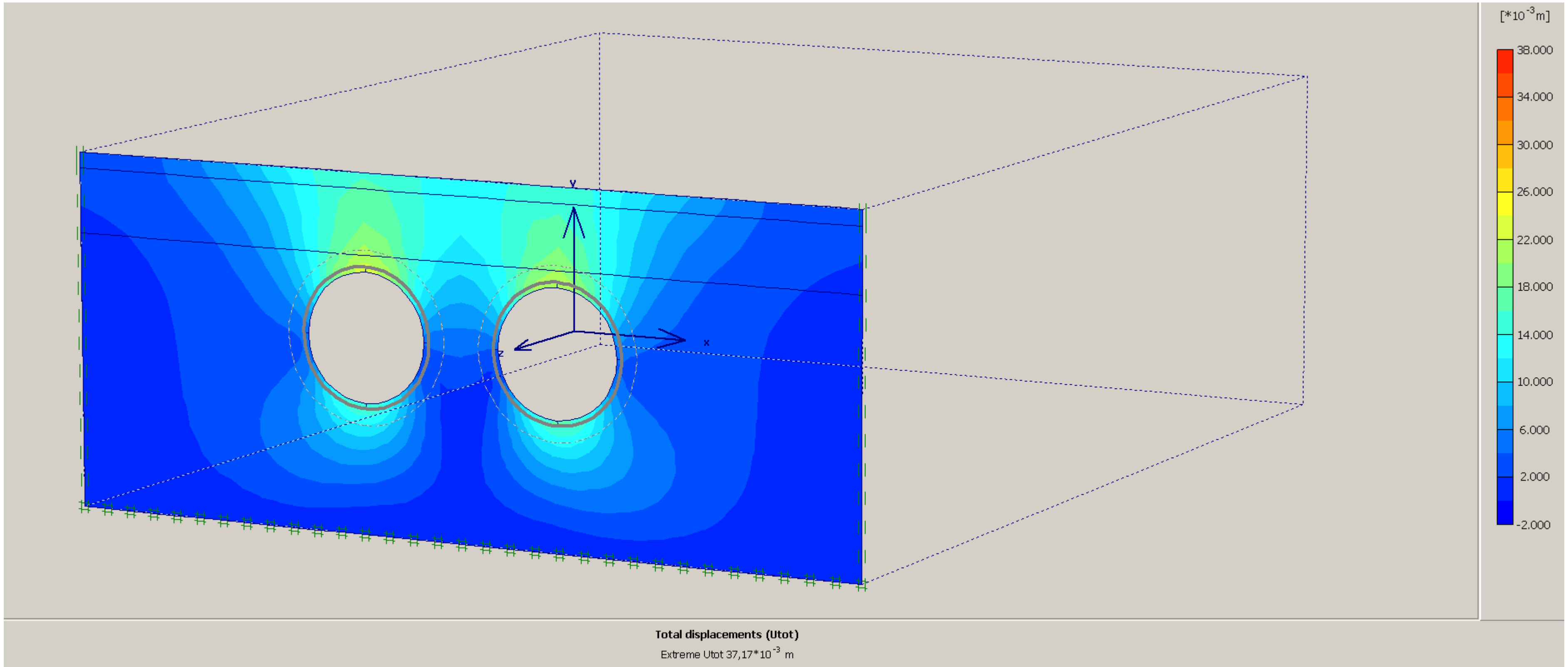

Figure A.3: 1+200 Front Plane Displacements

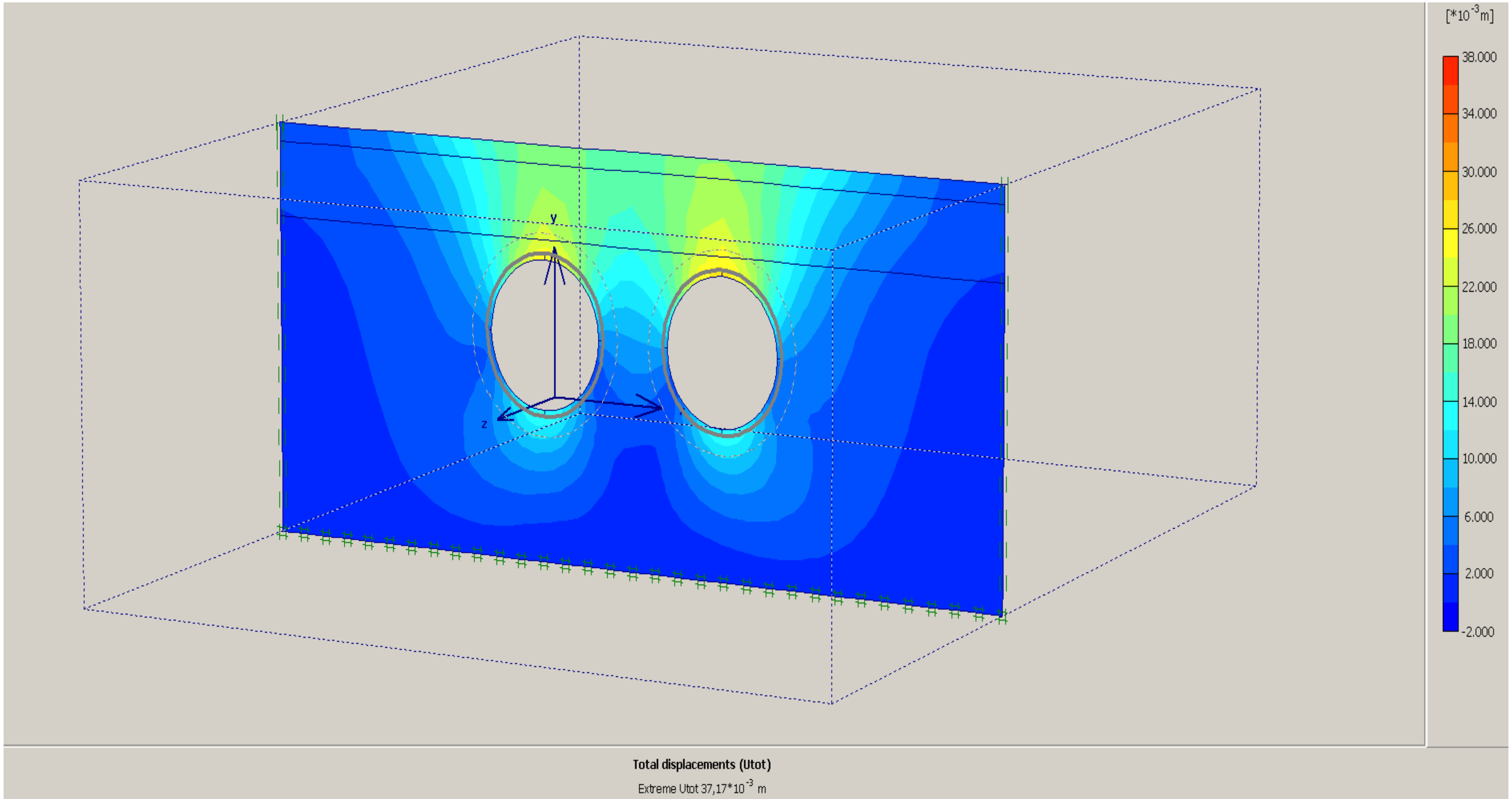

Figure A.4: 1+200 Plane A Displacements

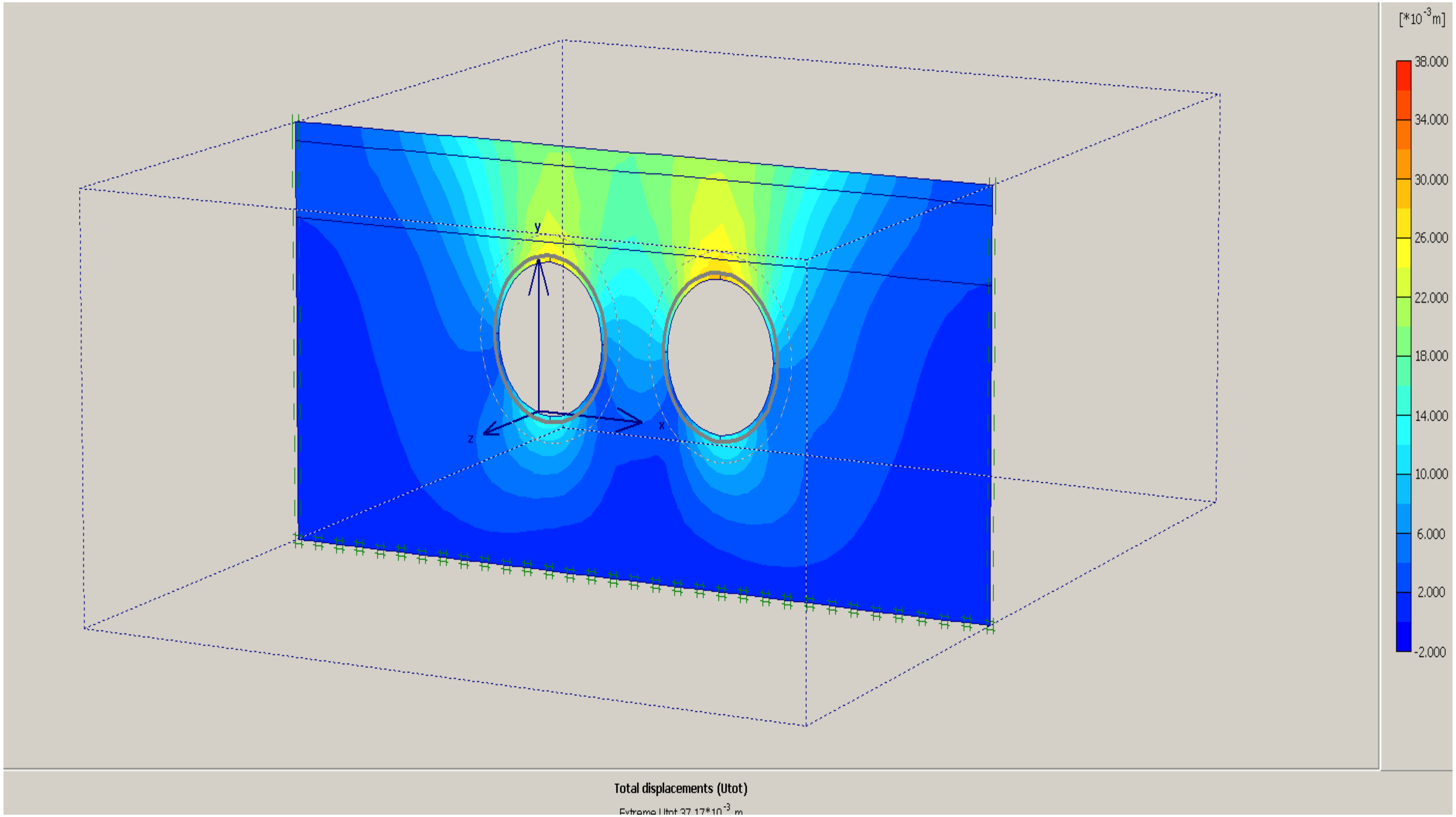

Figure A.5: 1+200 Plane C Displacements

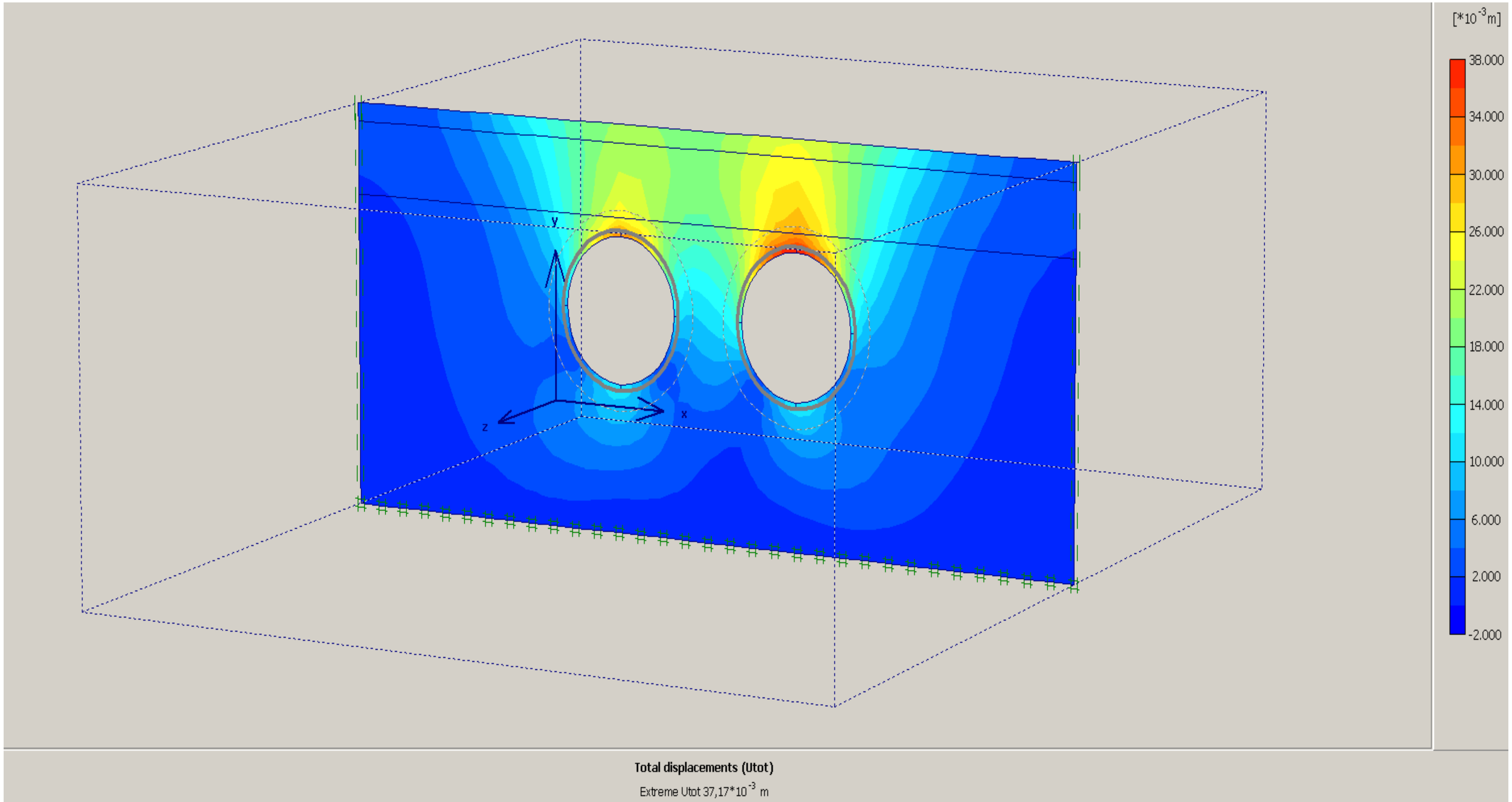

Figure A.6: 1+200 Plane H Displacements

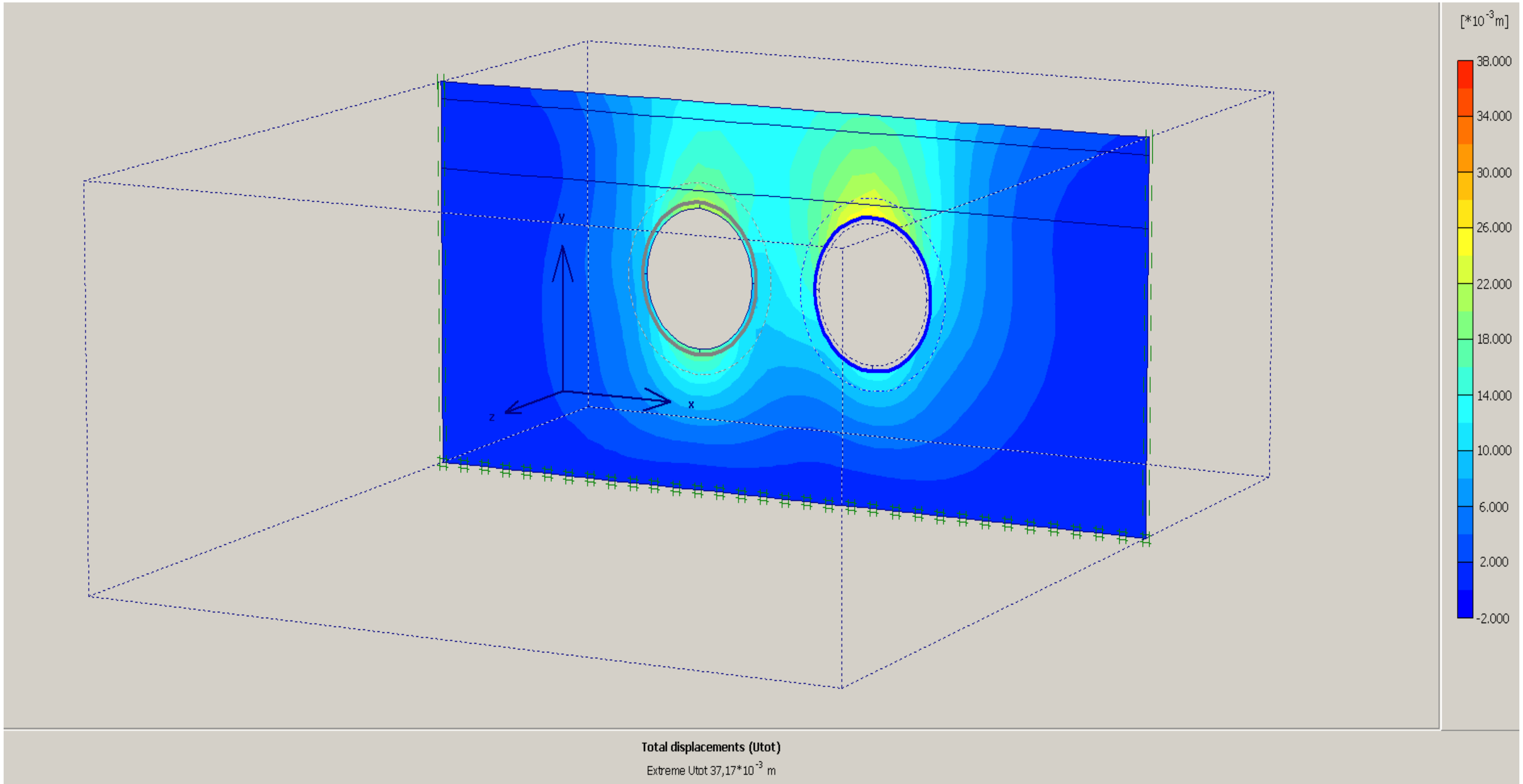

Figure A.7: 1+200 Plane O Displacements

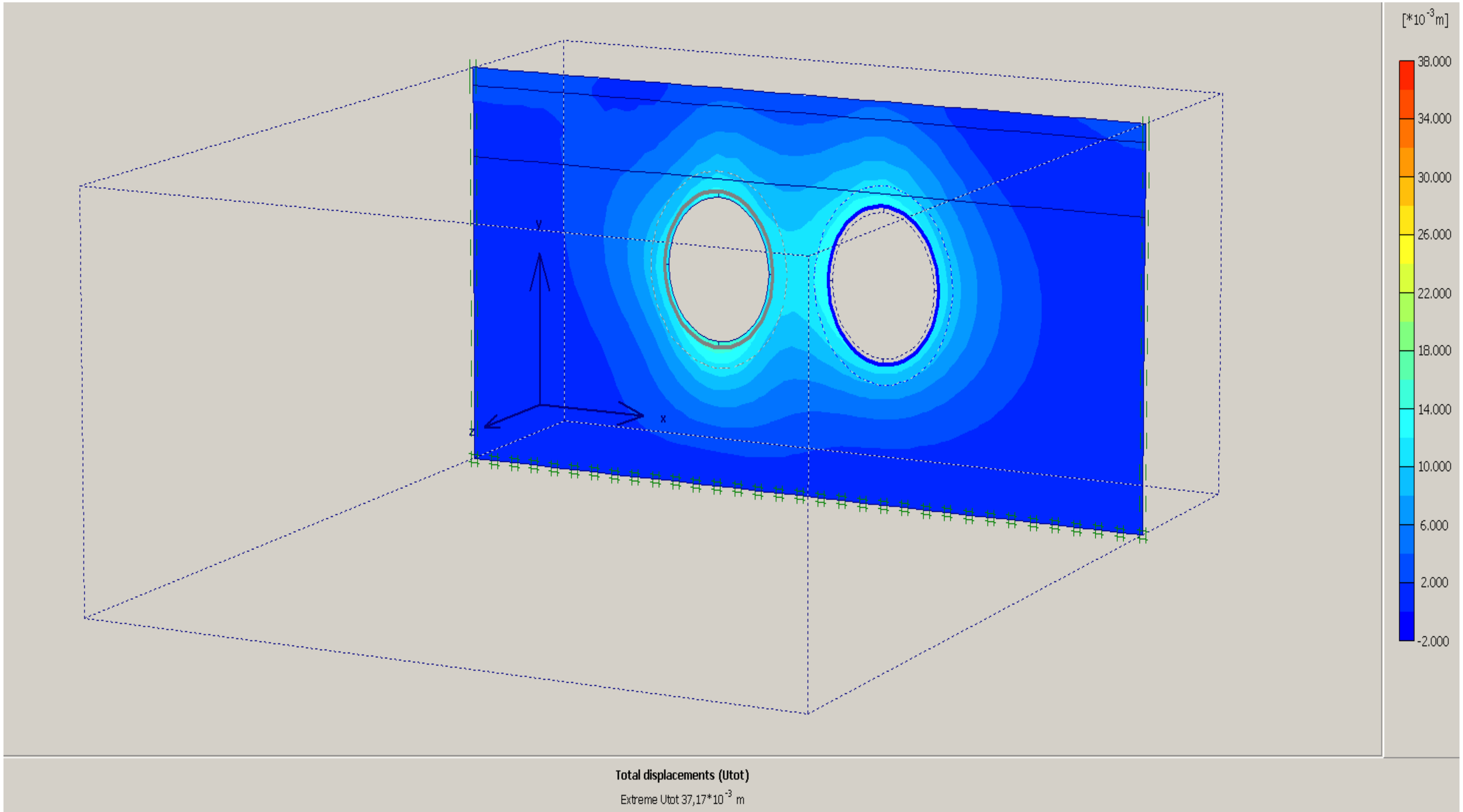

Figure A.8: 1+200 Plane T Displacements

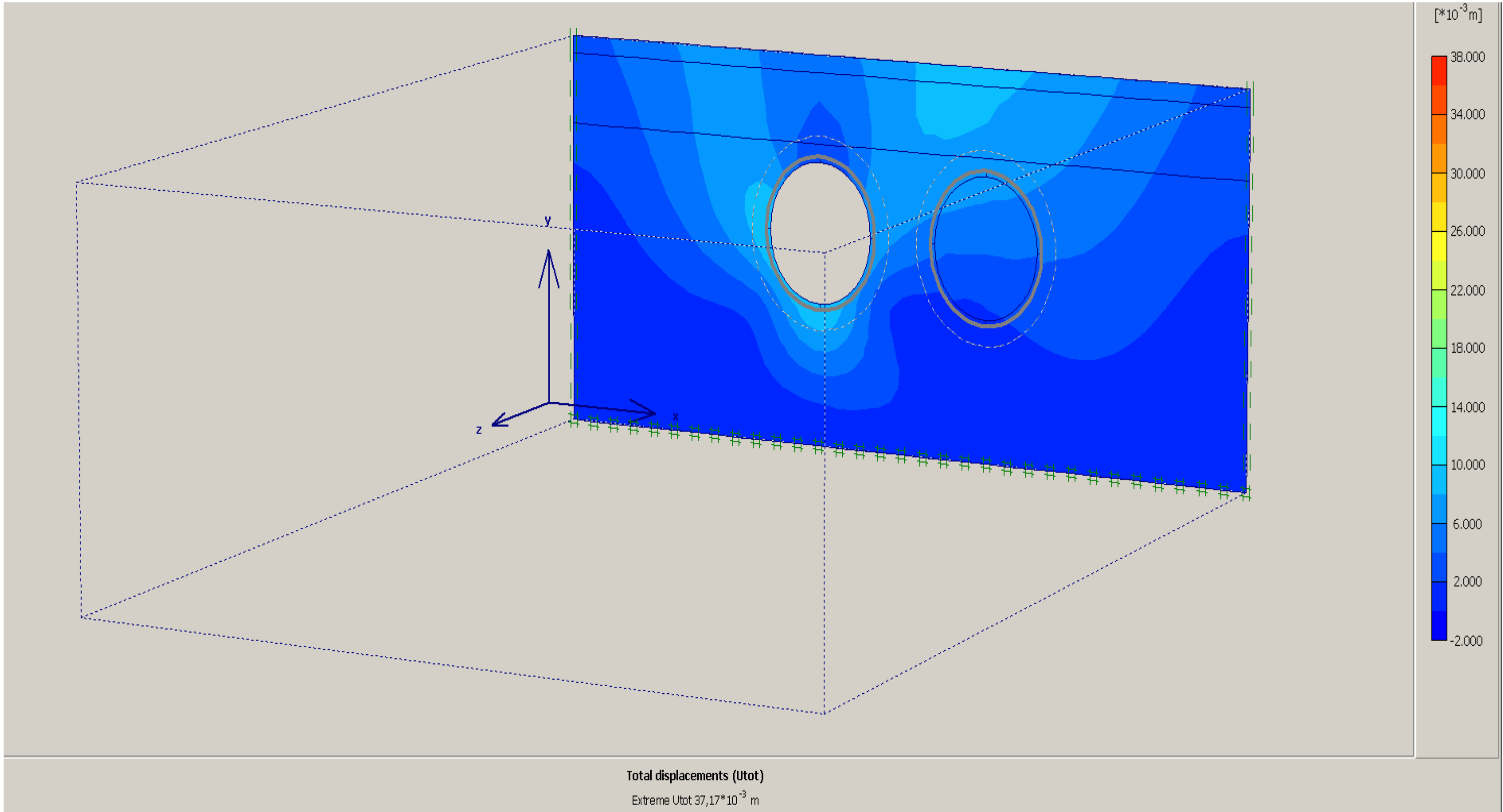

Figure A.9: 1+200 Rear Plane Displacements

APPENDIX B

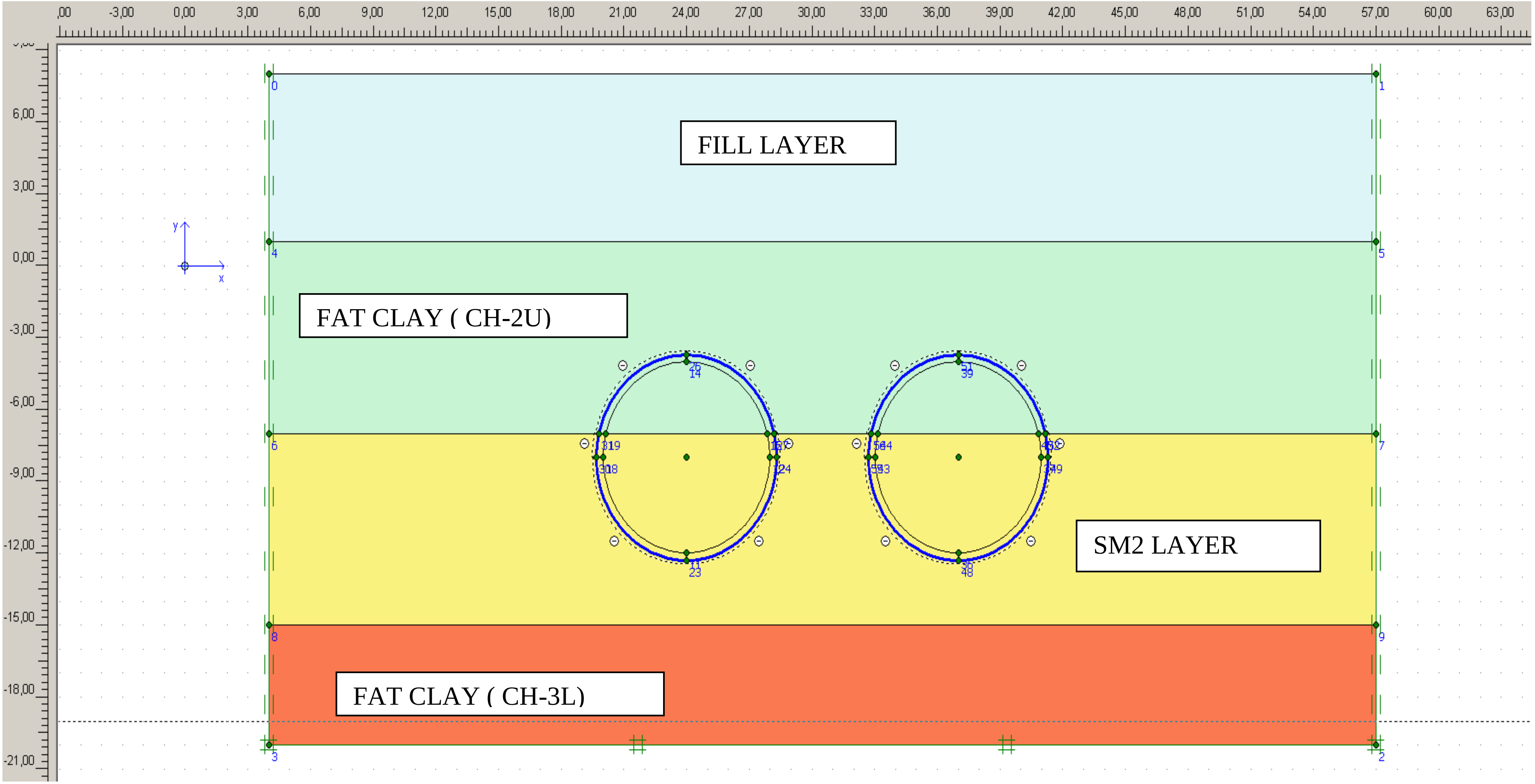

Figure B.1: 2+060 Model Layout

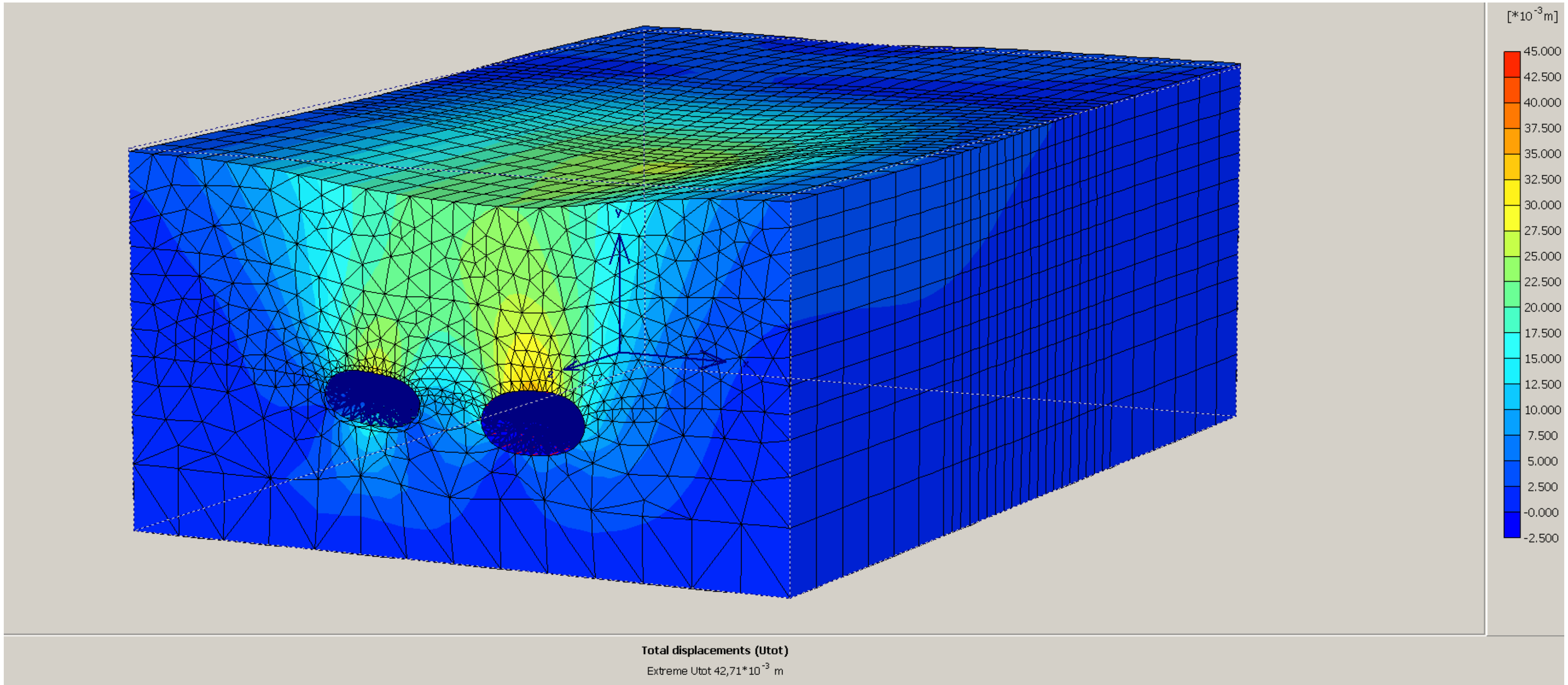

Figure B.2: 2+060 3D Total Displacements

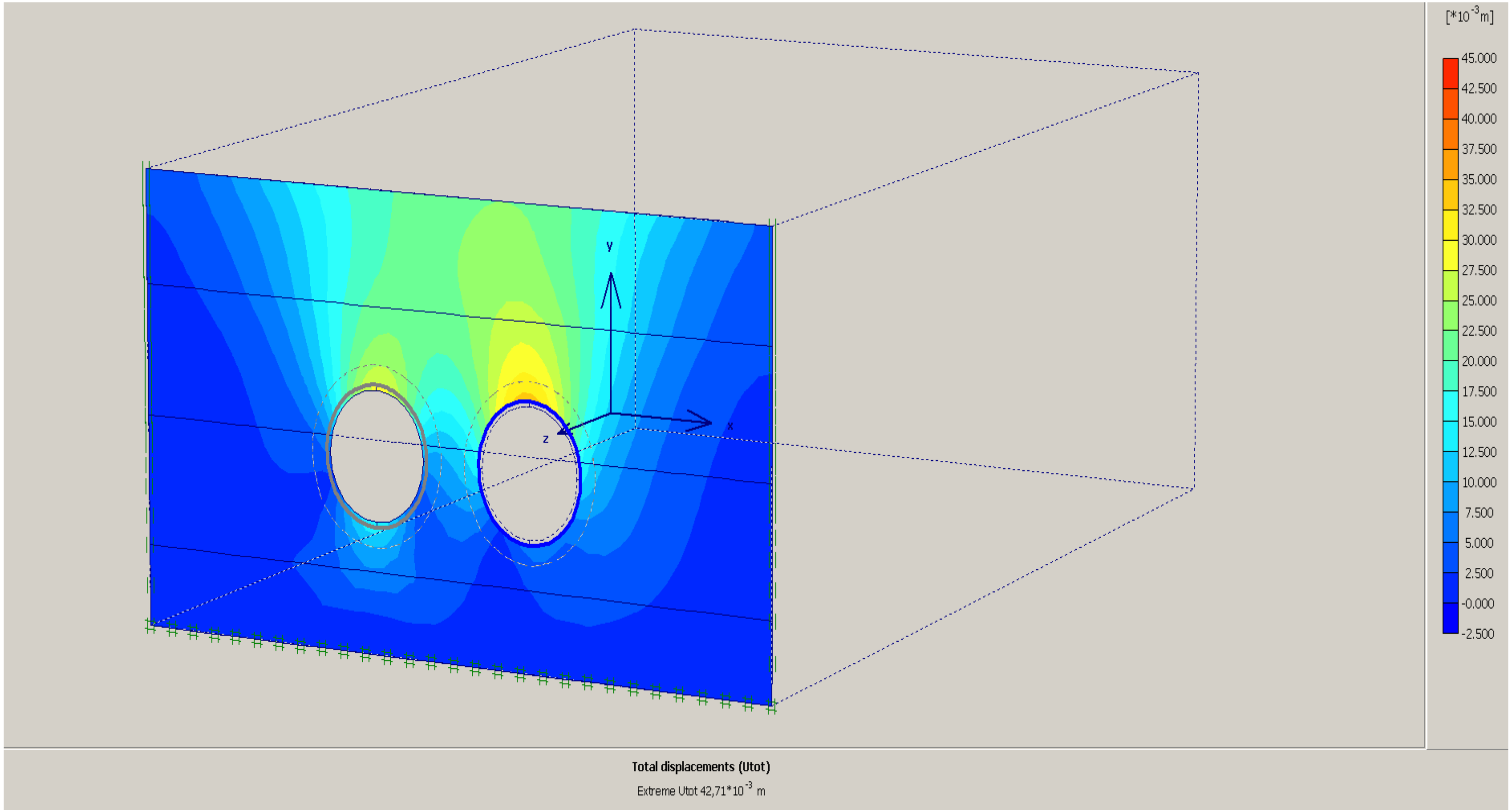

Figure B.3: 2+060 Front Plane Displacements

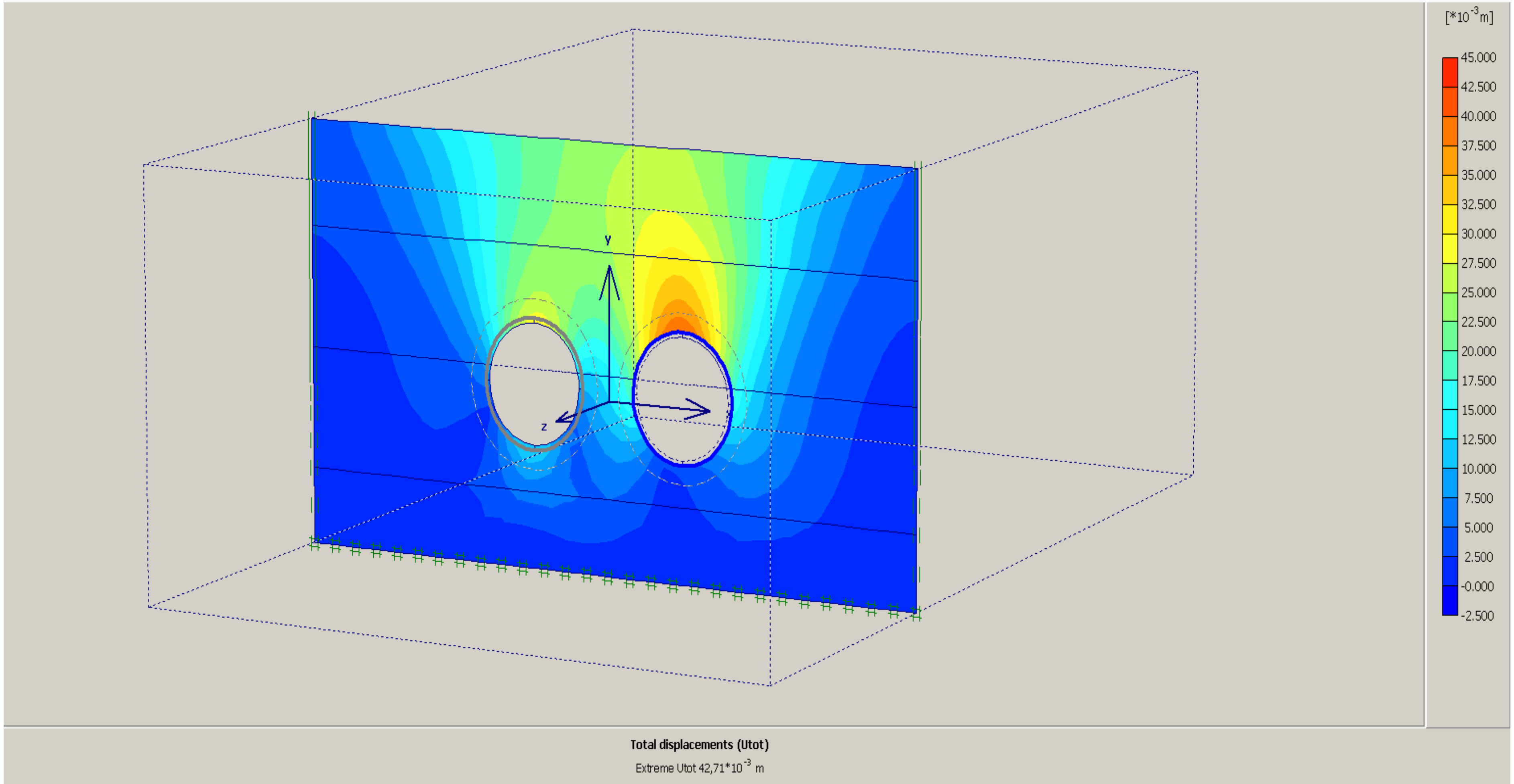

Figure B.4: 2+060 Plane A Displacements

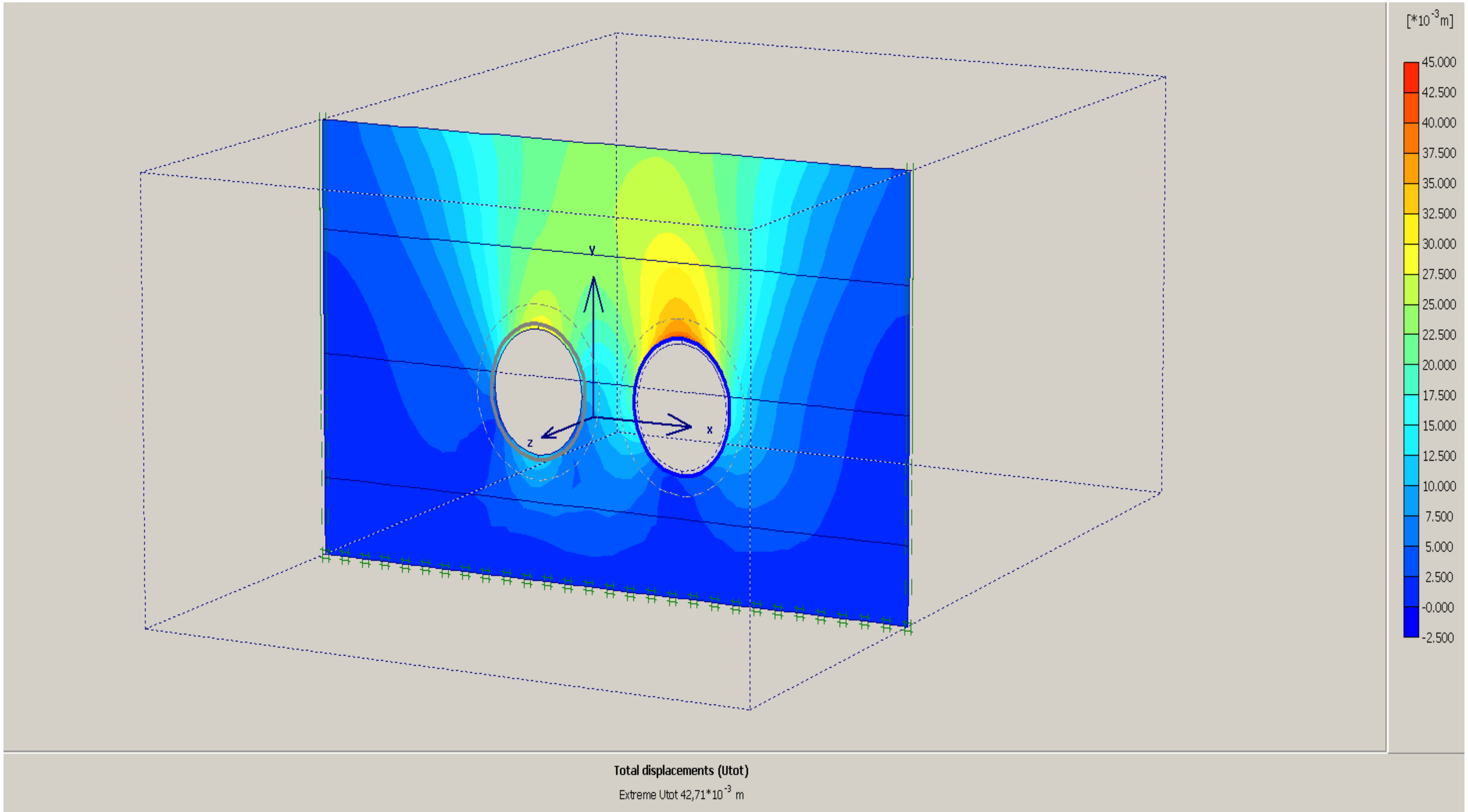

Figure B.5: 2+060 Plane C Displacements

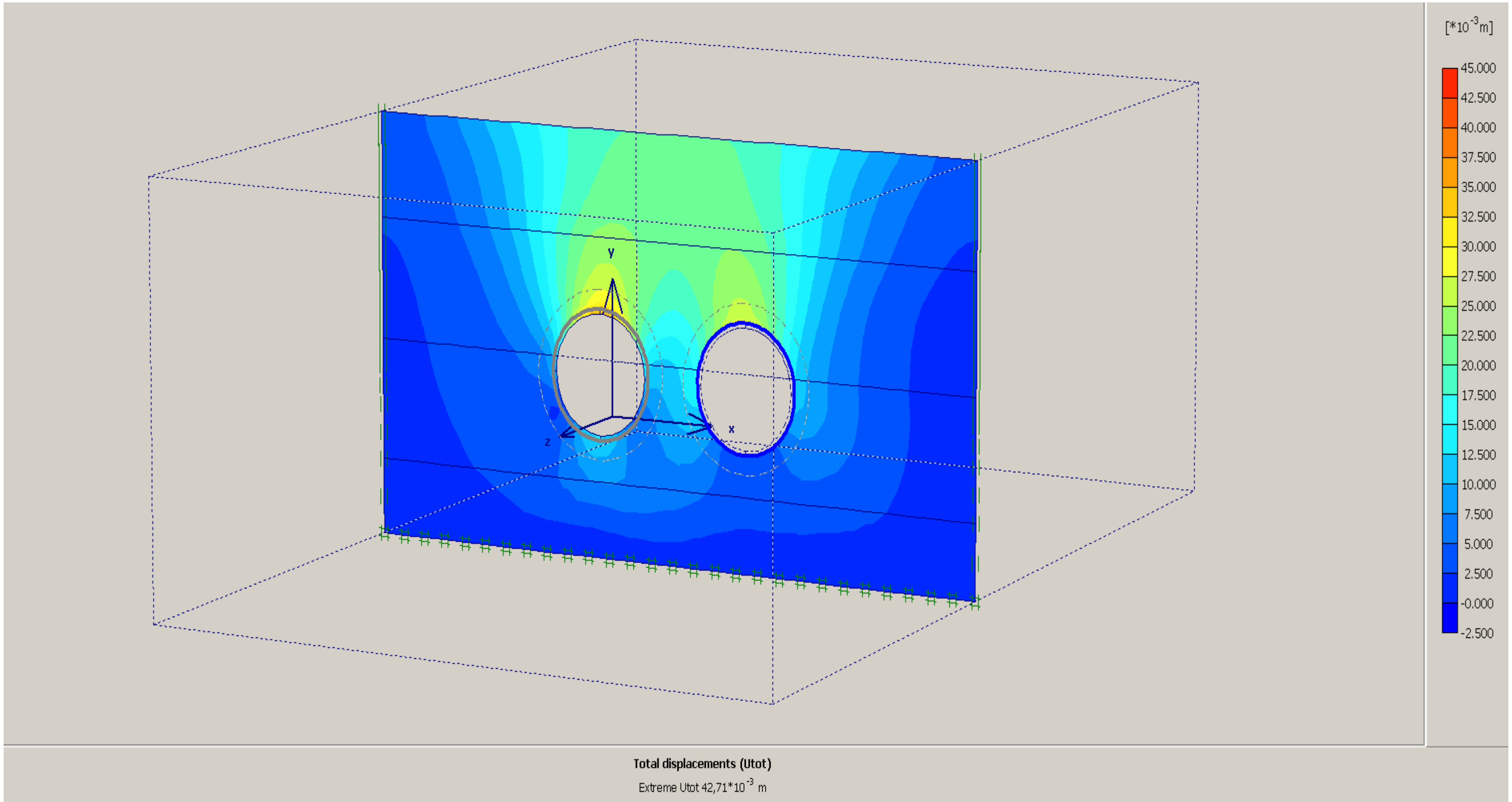

Figure B.6: 2+060 Plane H Displacements

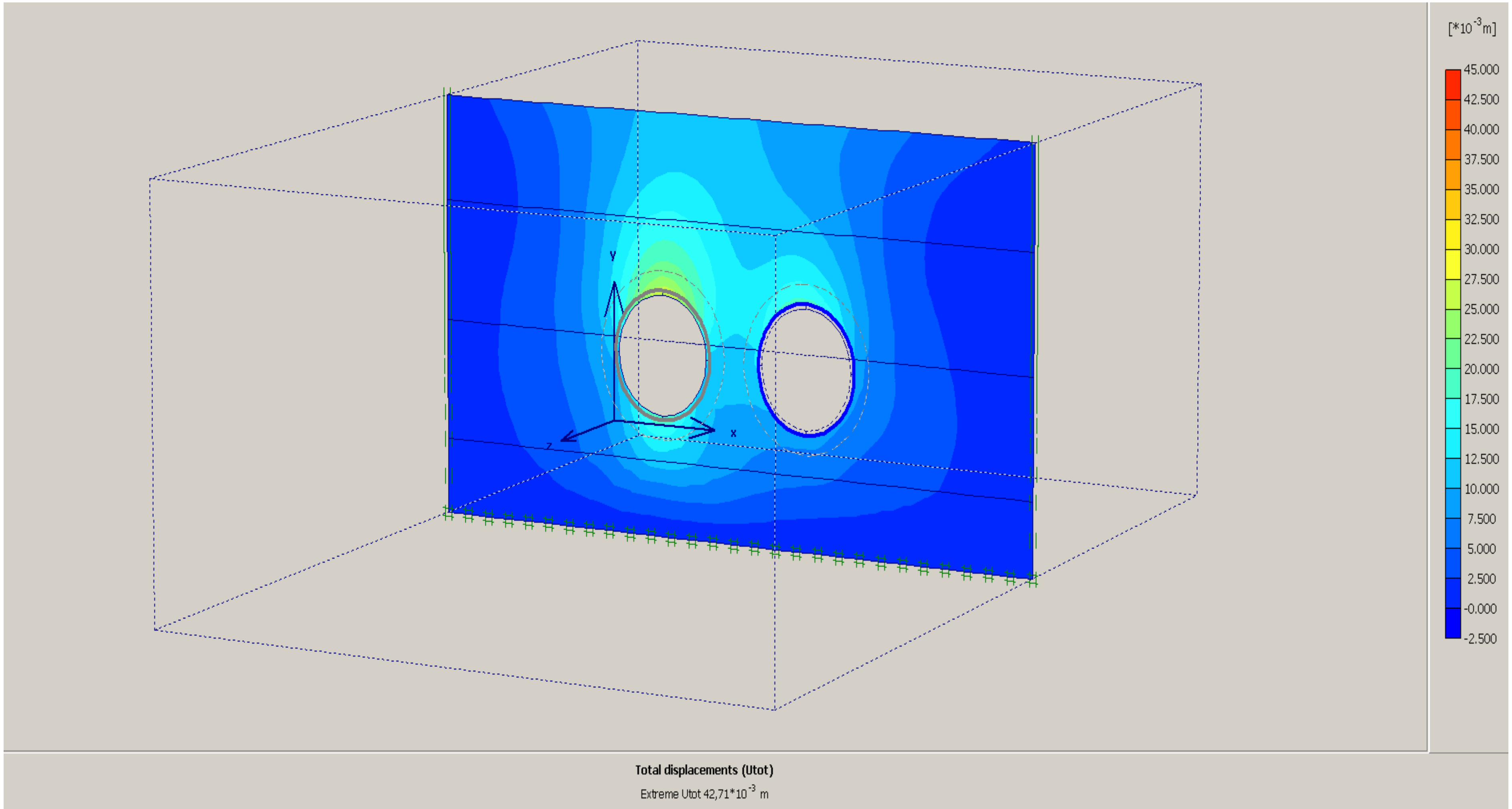

Figure B.7: 2+060 Plane O Displacements

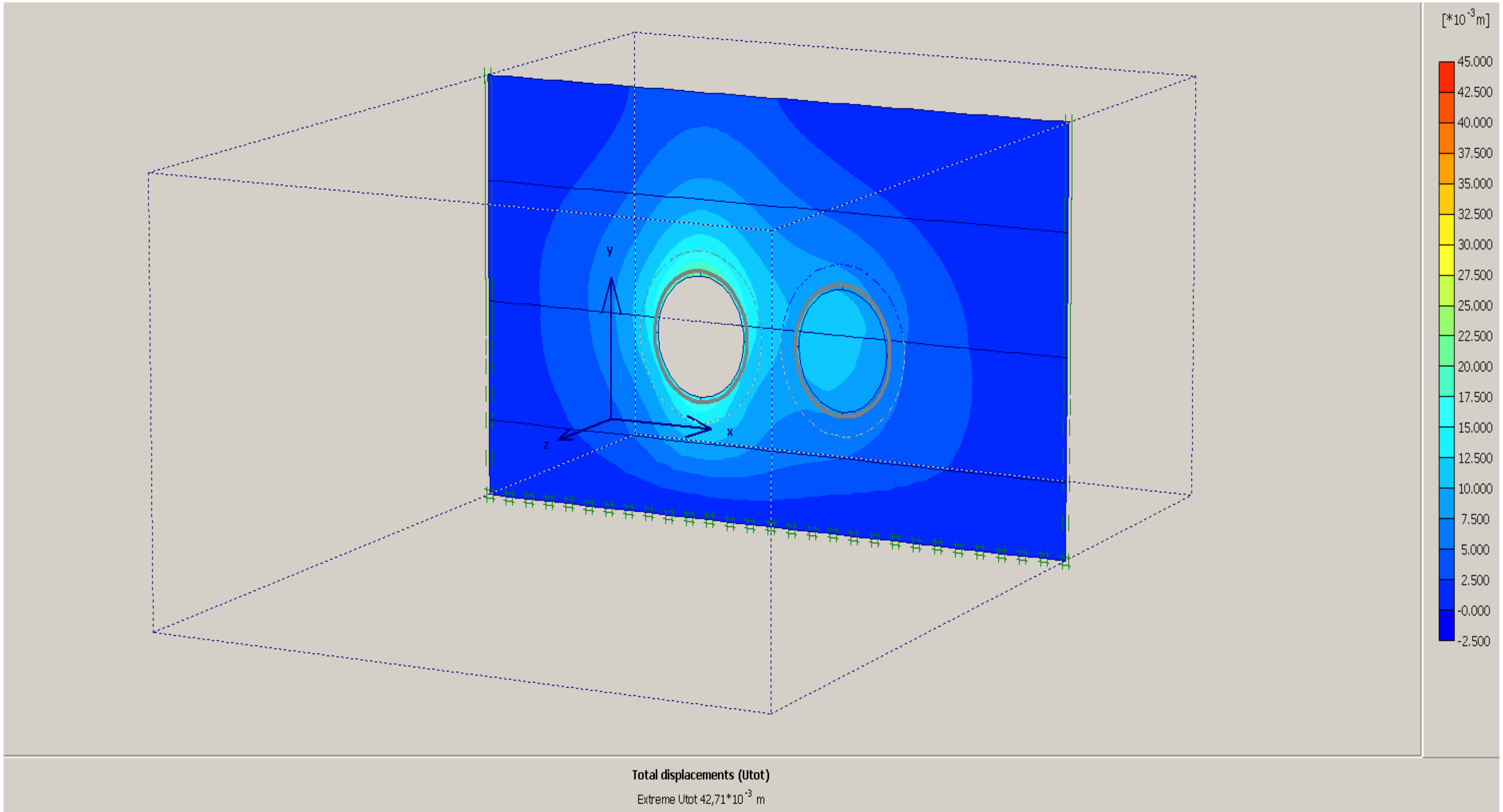

Figure B.8: 2+060 Plane T Displacements

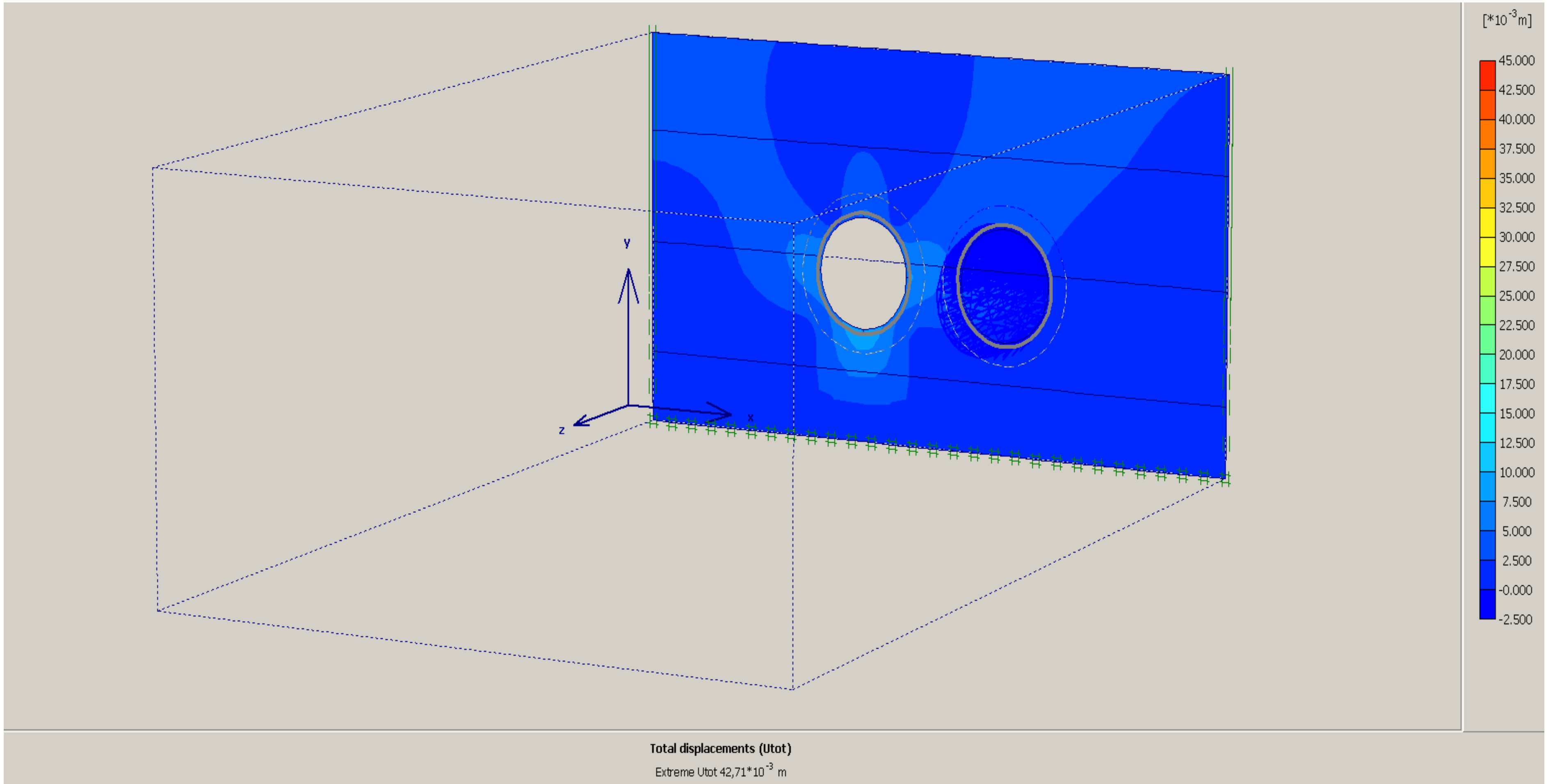

Figure B.9: 2+060 Rear Plane Displacements

APPENDIX C

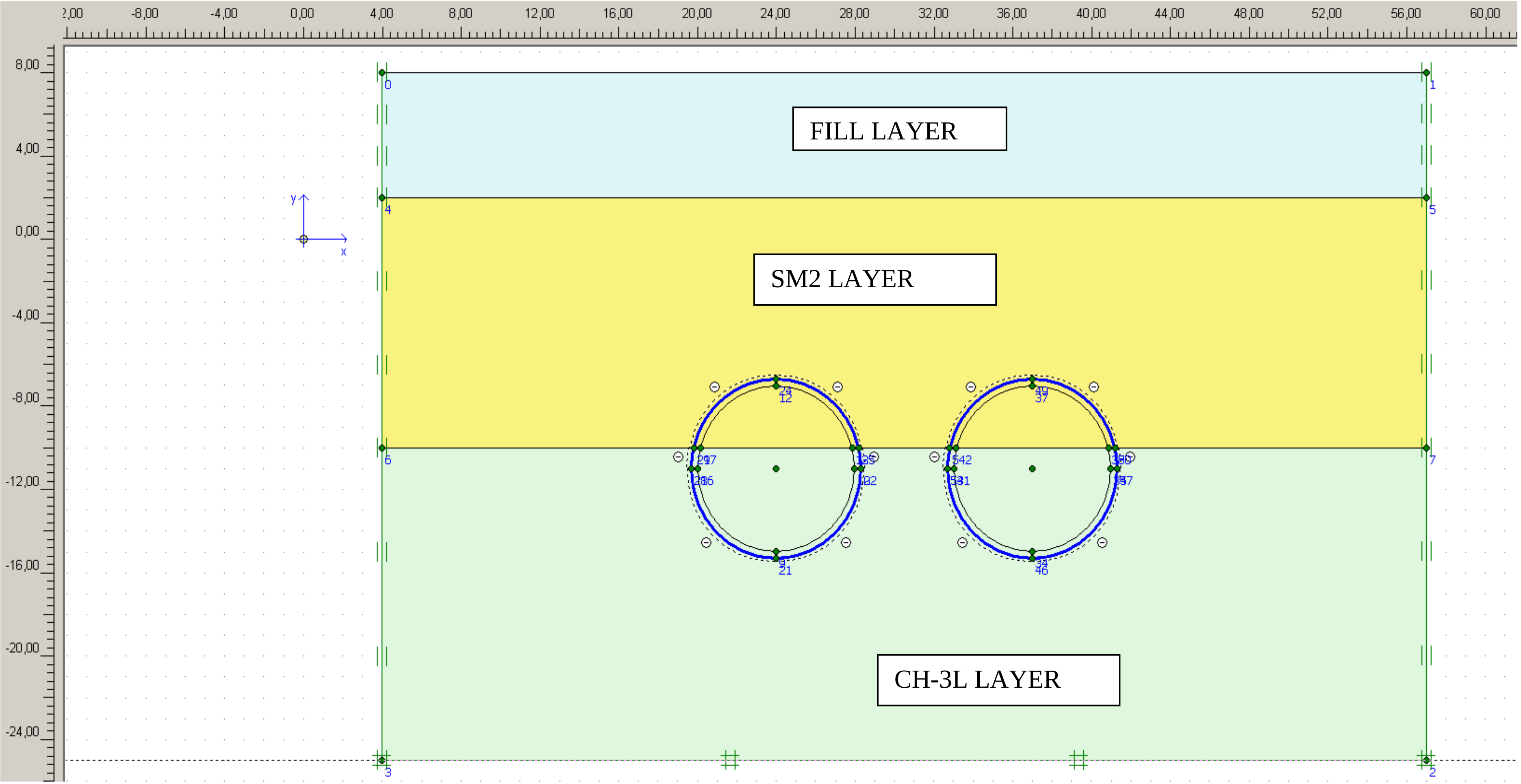

Figure C.1: 2+400 Model Layout

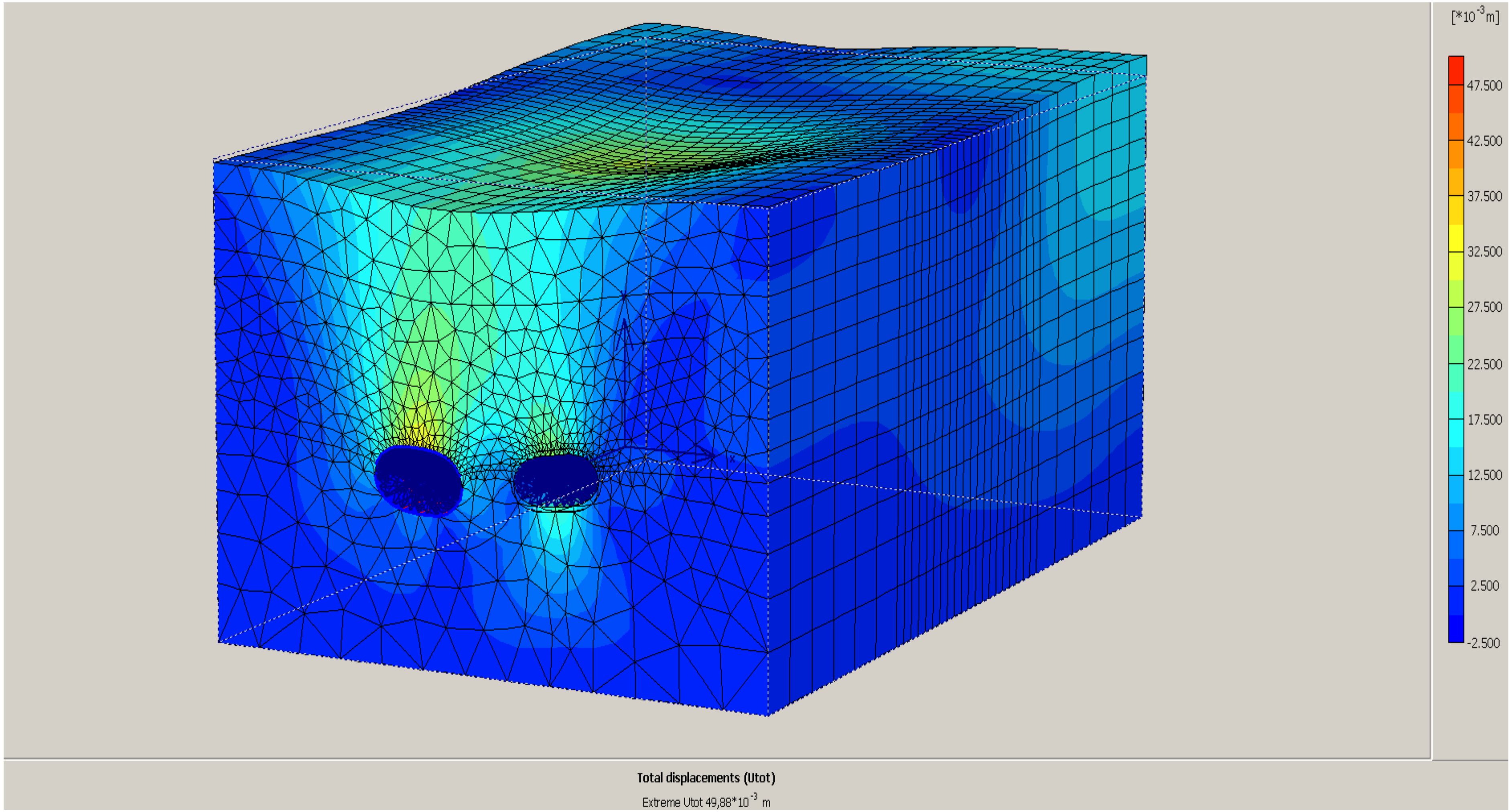

Figure C.2: 2+400 3D Total Displacements

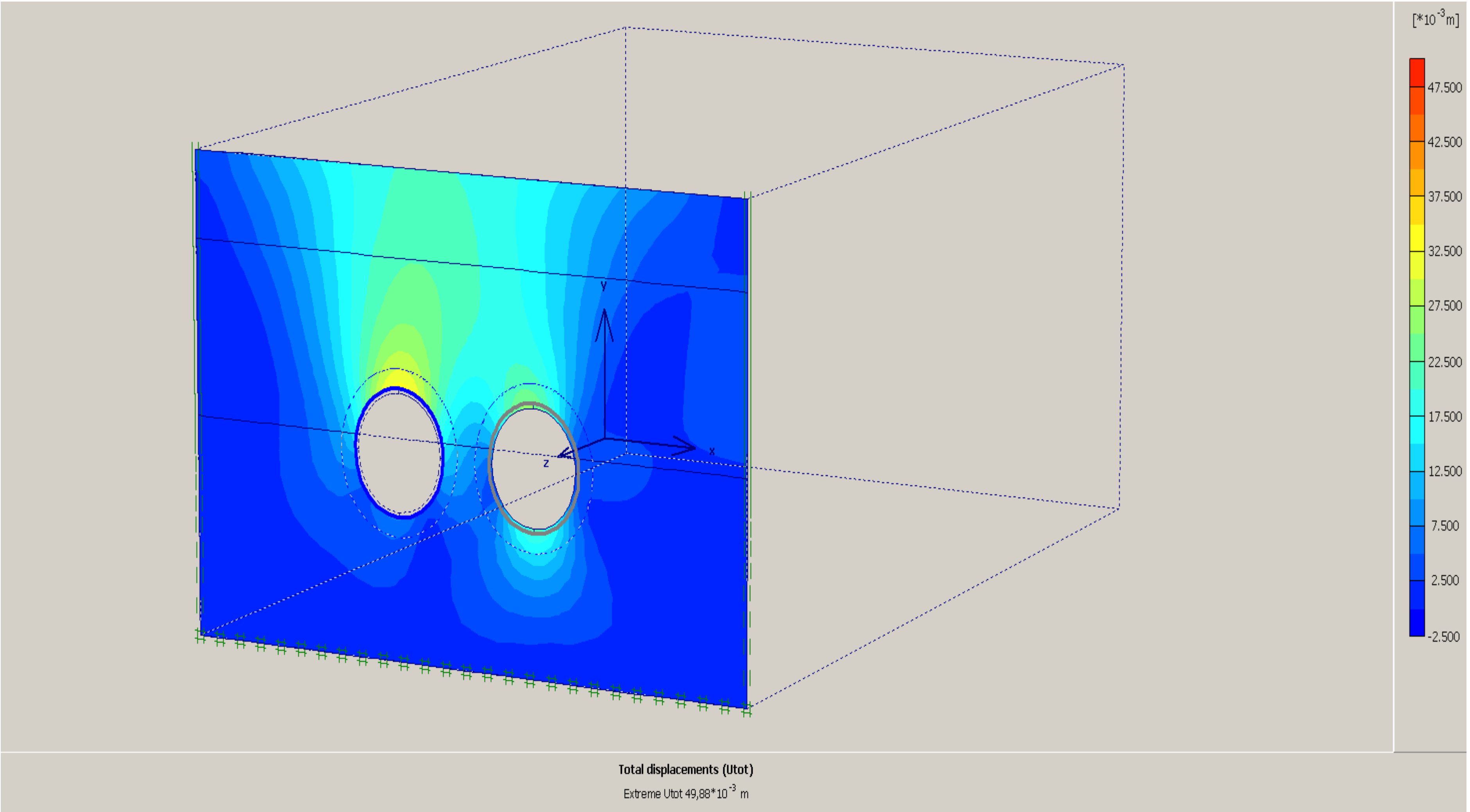

Figure C.3: 2+400 Front Plane

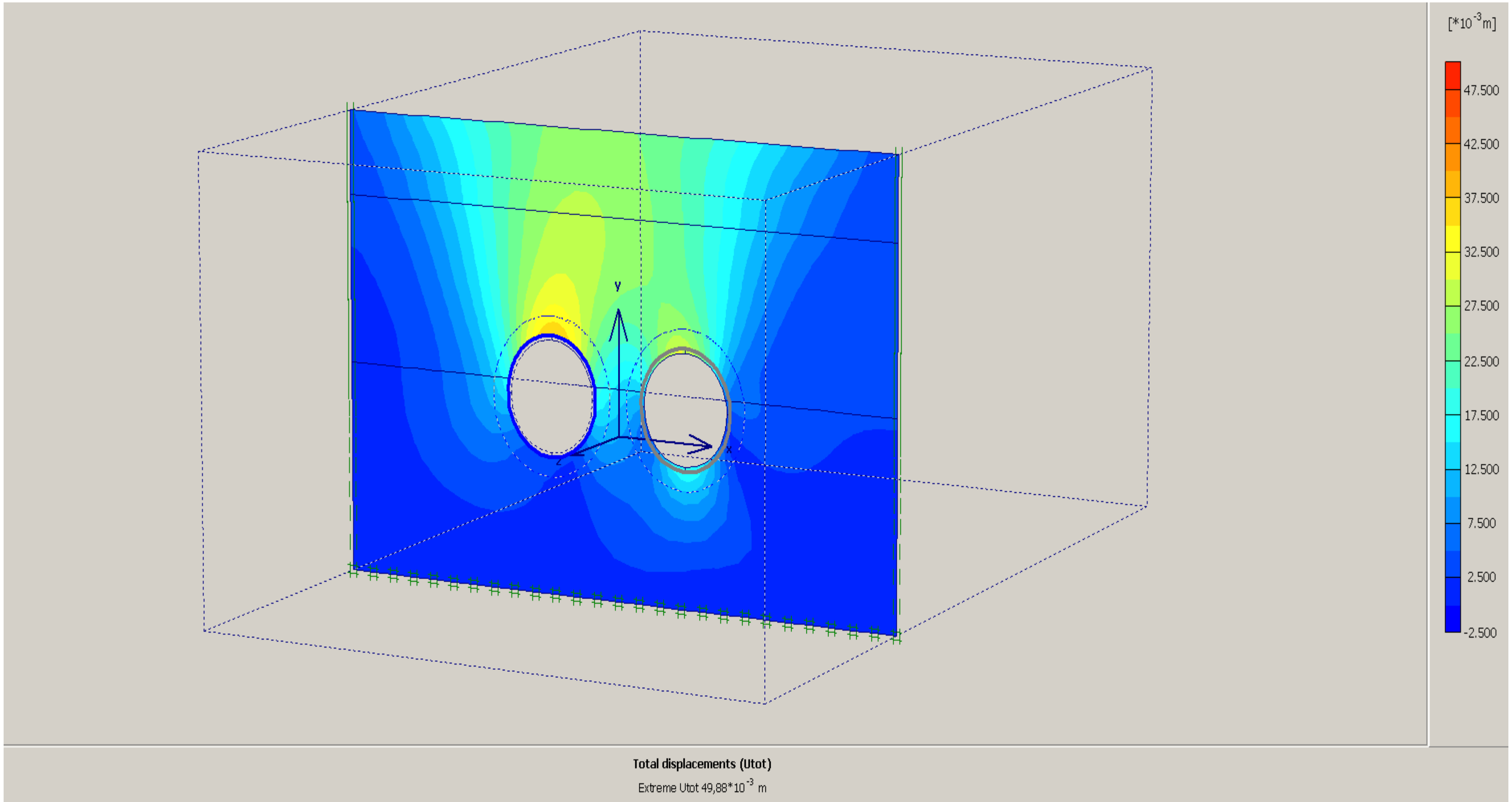

Figure C.4: 2+400 Plane A

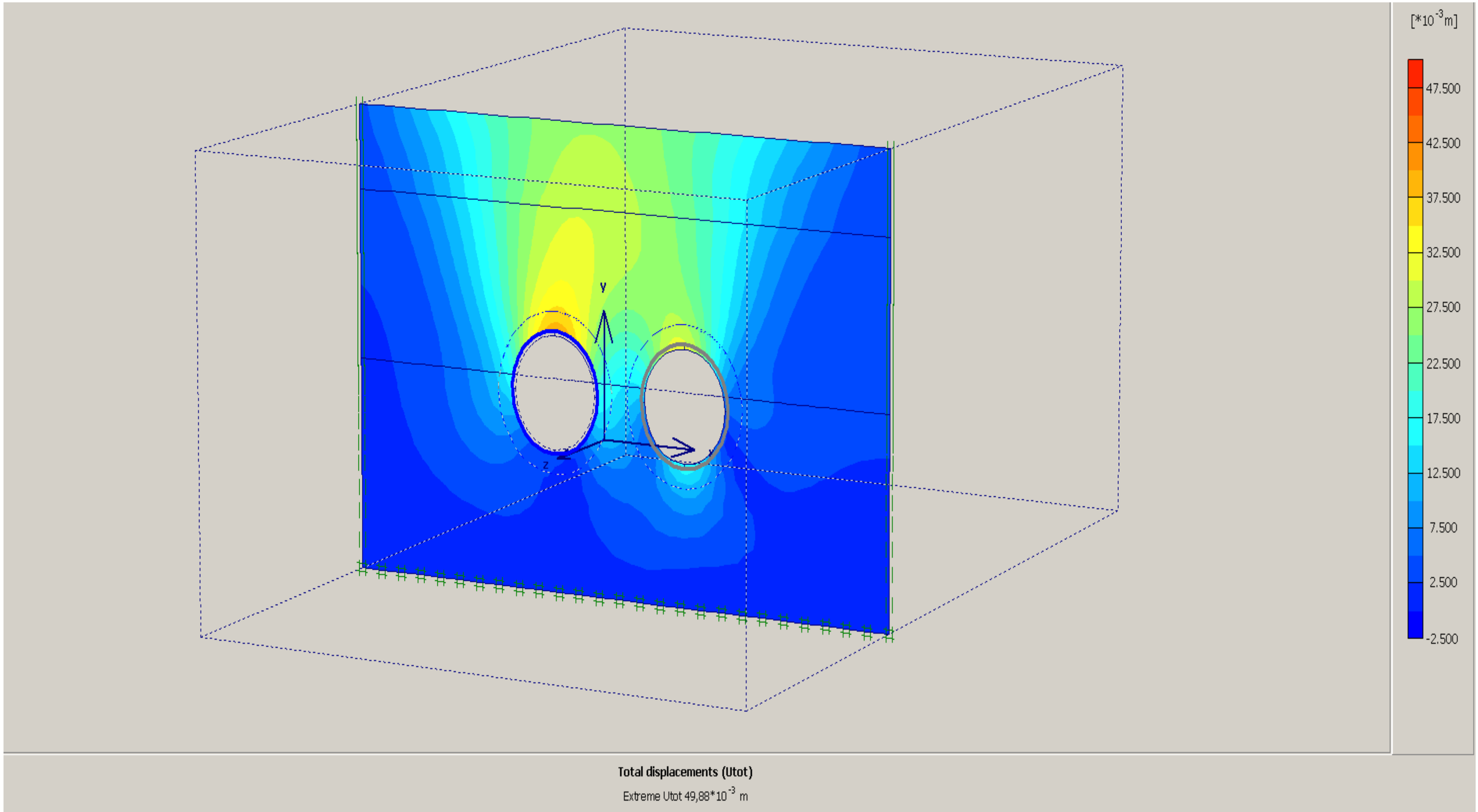

Figure C.5: 2+400 Plane C

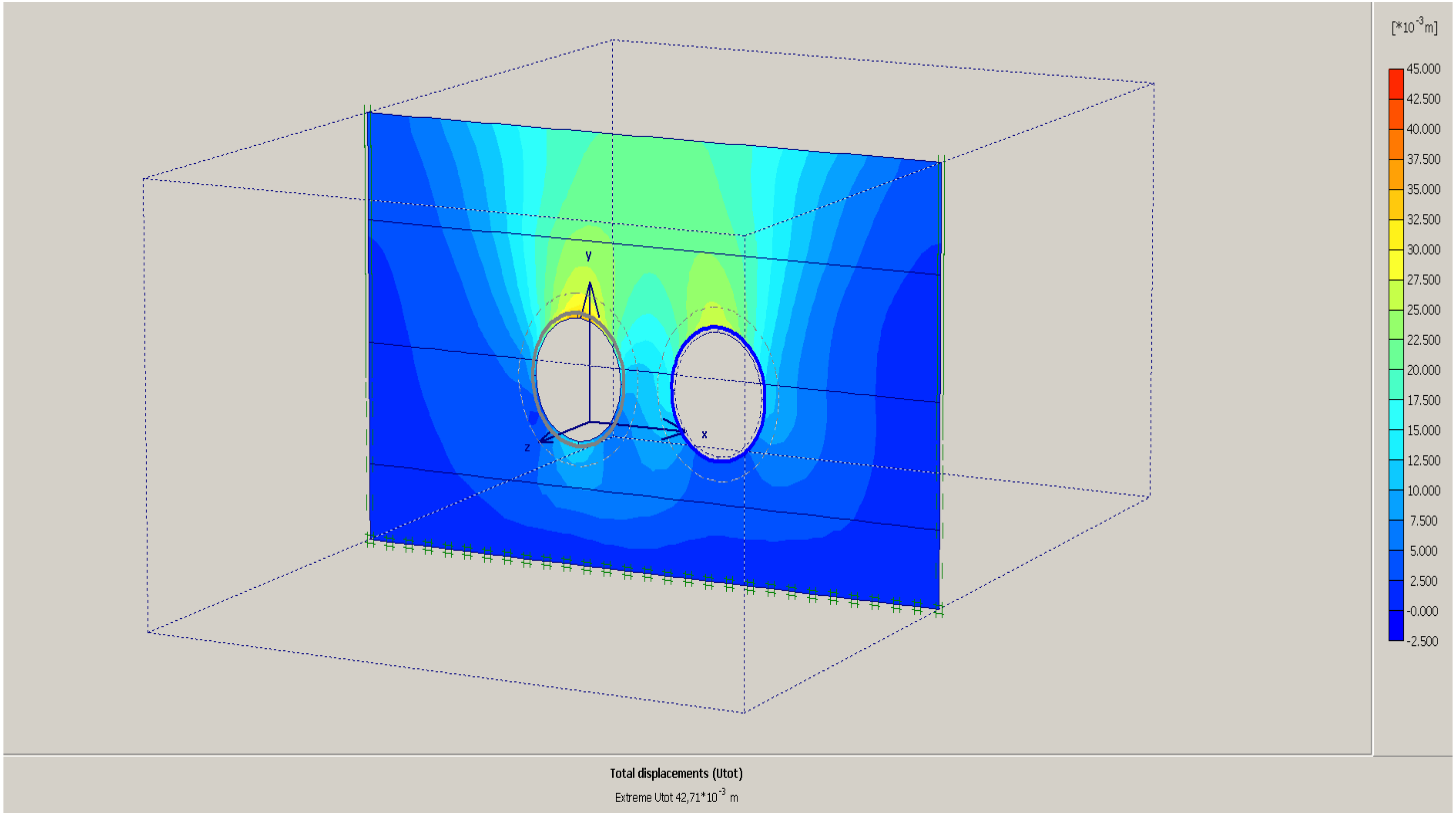

Figure C.6: 2+400 Plane H

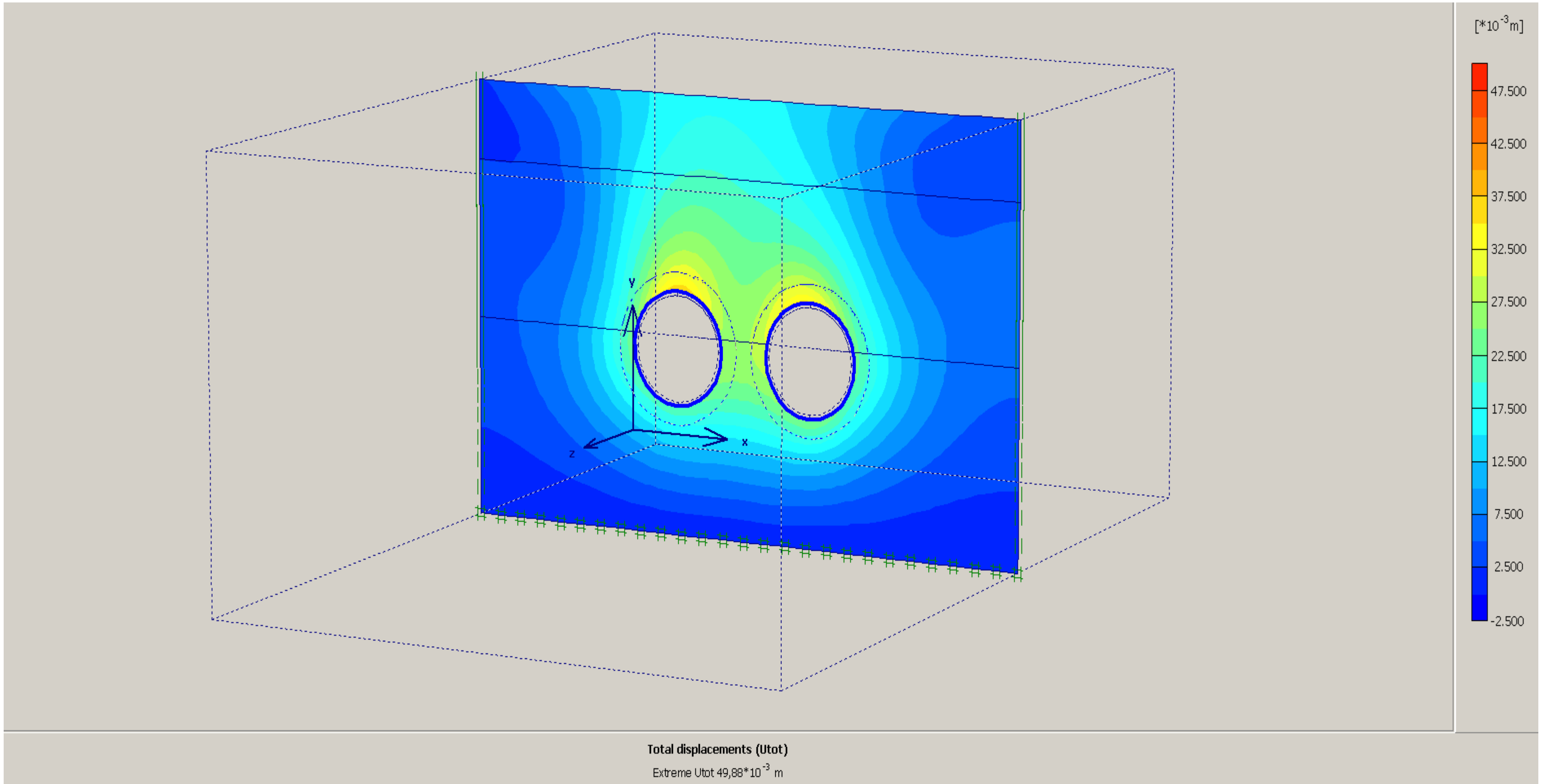

Figure C.7: 2+400 Plane O

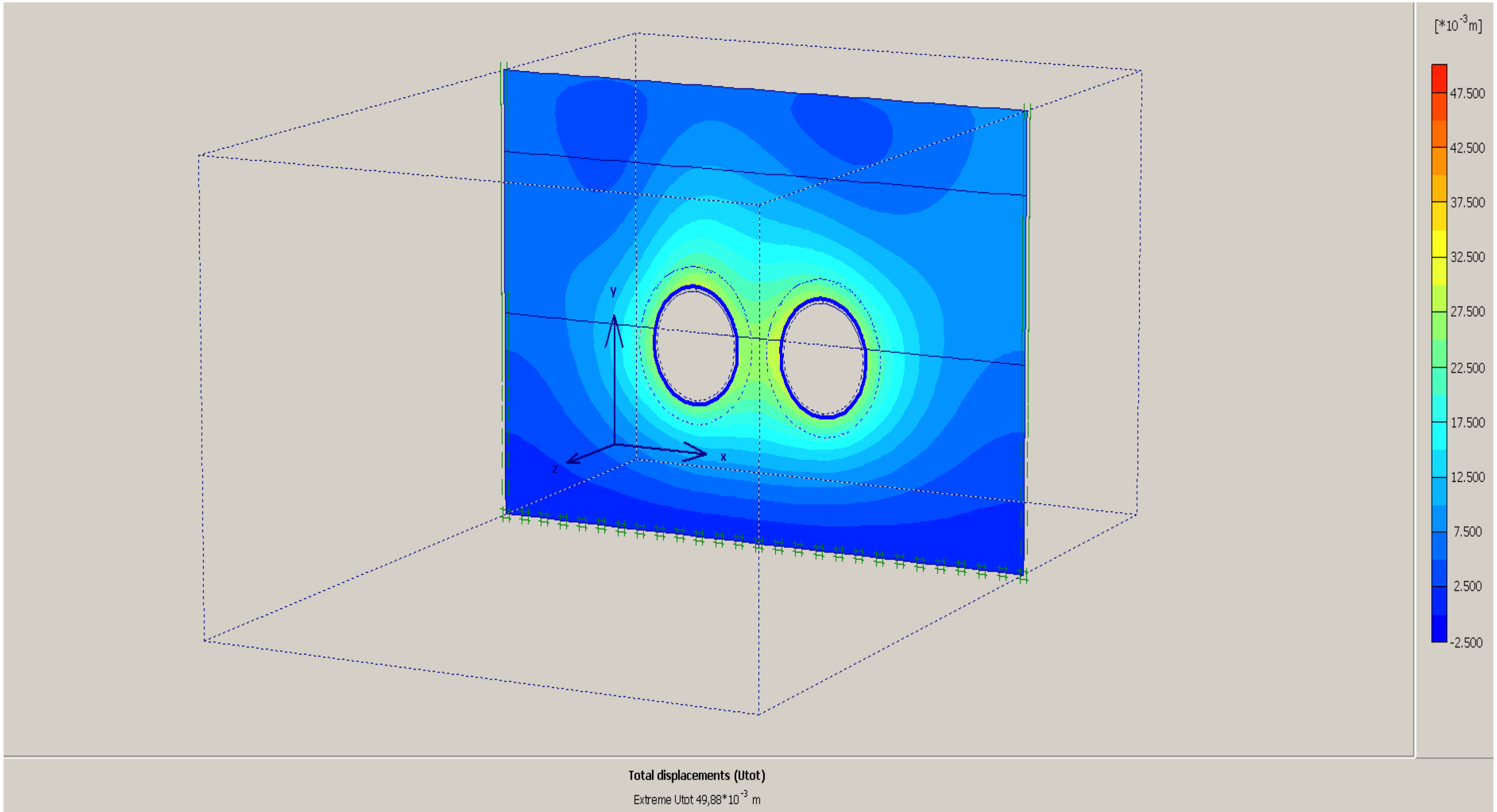

Figure C.8: 2+400 Plane T

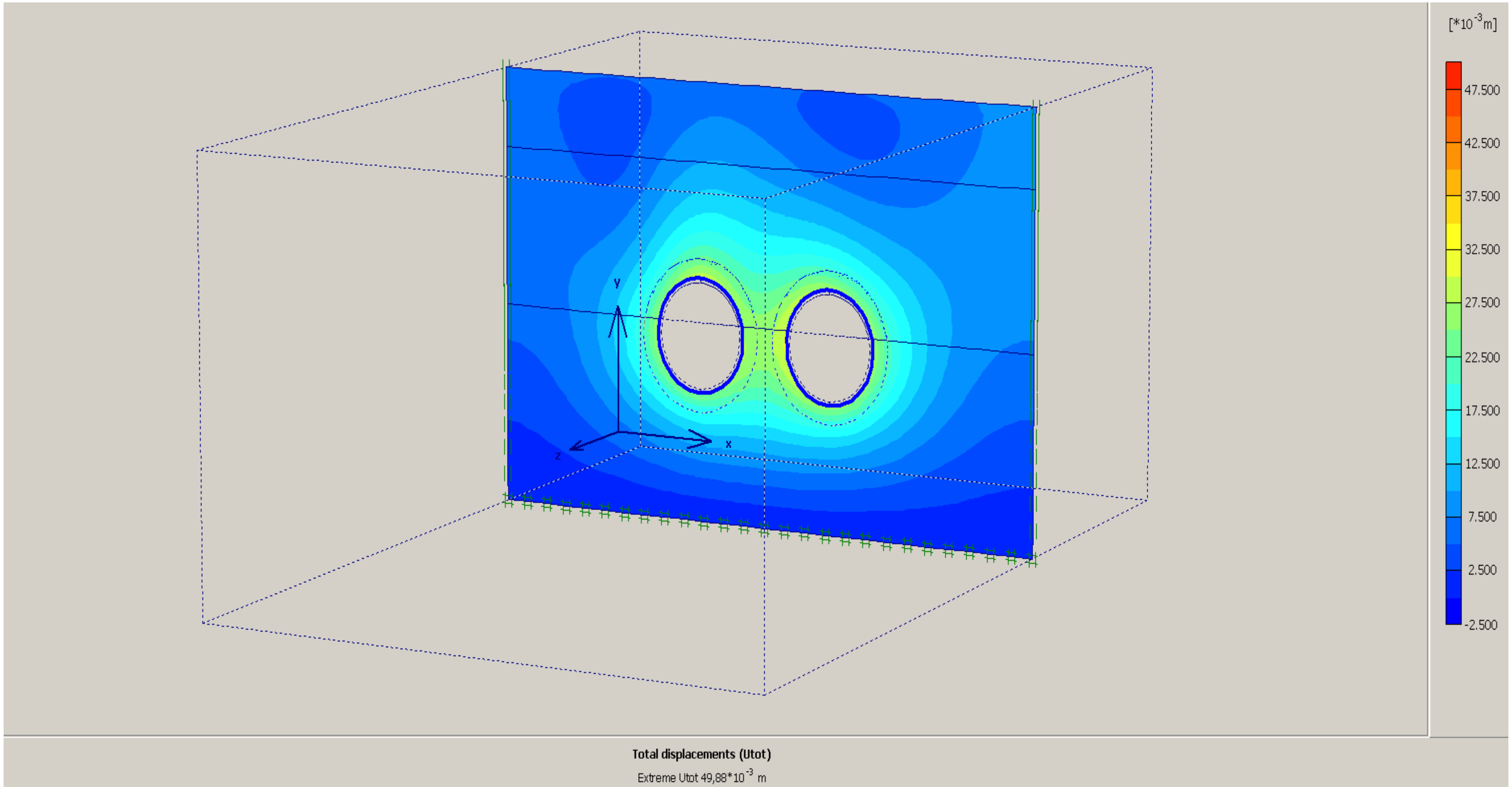

Figure C.9: 2+400 Rear Plan

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List of Publications and Patents: N/A

PUBLICATIONS/PRESENTATIONS ON THE THESIS

N/A