İSTANBUL TECHNICAL UNIVERSITY ★ INSTITUTE OF SCIENCE AND TECHNOLOGY

PERFORMANCE ANALYSIS OF EPB-TBM IN DIFFICULT GEOLOGICAL CONDITIONS FOR KADIKOY – KARTAL METRO PROJECT

M.Sc. Thesis by

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Programme : Mining Engineering

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JUNE 2011

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

KADIKÖY – KARTAL METROSUNDA ZOR JEOLOJİK KOŞULLARDA KULLANILAN EPB-TBM'in PERFORMANS ANALİZİ

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HAZİRAN 2011

To my family,

FOREWORD

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June 2011 Emre Avunduk

Mining Engineer

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ABBREVIATIONS

TBM	**:** Tunnel Boring Machine
EPB	**:** Earth Pressure Balance TBM
CL	**:** Clay
c	:Cohesion
cu	**:** Undrained shear strength
E	: Elasticity or Young's modulus
k	**:** Permeability
K,Ky	**:** Coefficient of horizontal effective stress
Ka,K0,Kp	**:** Coefficients of active, neutral and passive horizontal effective stress
γ	:Soil volumetric weight
γ′	**:** Effective soil volumetric weight
γ_{dry}, γ_{sat}	**:** Dry and saturated soil volumetric weight resp.
γ_w	**:** Water volumetric weight
ϕ	**:** Angle of internal friction,
a	:Soil arching relaxation length
B	: Reduced diameter or wedge width
˜c	: Hydraulic resistance
C	: Overburden
D	: Outer diameter of TBM
E	: Resultant earth force
G	**:** Effective weight
Gs	**:** Overburden force on wedge
Gw	**:** Effective weight of wedge
h	:Water level
ha	**:** Critical air infiltration height
H	**:** Thickness of soil layer
hw	**:** Murayama's model parameter
Ns ,Nγ	**:** Leca and Dormieux load factors
p_0	**:** Pore water pressure at rest
P	: Unsupported face length,
qs	**:** External load at soil surface
qw	**:** Effective load at wedge top
Q	: Wedge face force
Qa,Qb	**:** Continuity conditions force on top resp. bottom of wedge segment
ra, rd	**:** Murayama's model parameters
rp	**:** Equivalent pore radius
R	**:** Radius of TBM,
s	**:**Support pressure
s_{min}, s_{max}	**:** Minimal and maximal support pressure resp.
s′	**:** Effective support pressure, s − p
IPR	**:** Instantaneous penetration rate

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PERFORMANCE ANALYSIS OF EPB-TBM IN DIFFICULT GEOLOGICAL CONDITIONS FOR KADIKOY – KARTAL METRO PROJECT

SUMMARY

Many problems are encountered during the mechanical excavation of the tunnels due to the difficulties in the preplanning phase of the project. One of these problems is that uncertainties on the prediction of performance of tunnel boring machines (TBM).

The main purpose of this study is to analyze the effects of difficult ground conditions on an earth pressure balance (EPB) TBM. For this purpose, the alignment between Hasanpasa Crossroad – Otosan Land, around 500 m, of Kartal–Kadikoy Metro Project constructed by Anadoluray Joint Venture was selected as investigation area. The problematic locations and geological conditions were first defined. Face pressures were estimated for the areas where stability problems were expected. The problems encountered during the excavation were observed and recorded. Operational torque and thrust requirements, instantaneous penetration rates and face pressures were analyzed based on the data recorded by the TBM data logging system.

Studies indicated that the most important parameter affecting the performance of the EPB-TBM was the geological conditions. Performance of the EPB-TBM decreased due to difficult ground conditions and the main reasons for that was rock-soil mixed face, main rock-dyke contact zones, face stability problems and stoppages due to excessive disc cutter consumption and replacement.

KADIKÖY – KARTAL METROSU ZOR JEOLOJİK KOŞULLARDA KULLANILAN EPB-TBM'in PERFORMANS ANALİZİ

ÖZET

Zor zemin şartlarında mekanize tünel kazıları sırasında ilk planlama zorluklarına bağlı olarak, projenin yürütülmesi sırasında sık sık sorunlarla karşılaşılmaktadır. Bu problemlerden bir tanesi de tünel açma makinesinin (TBM) performansının tahminindeki belirsizliklerdir.

Bu çalışmada zor zemin şartlarının pasa basınçlı (EPB) TBM'in performansına etkilerinin araştırılması amaçlanmıştır. Bu amaçla Anadoluray tarafından yürütülen Kadıköy Kartal Metro Projesi inşaatının Hasanpaşa Kavşağı Otosan arazisi arasında kalan 500 metrelik bölümü çalışma alanı olarak belirlenmiştir. Arazi çalışmalarında öncelikle bölgenin jeolojisi ve tünel güzergâhının fiziksel durumu detaylı olarak araştırılıp TBM ile tünel kazısında sorun yaratacak bölgeler tespit edilmiştir. Stabilite problemleri yaşanabilecek bölgeler için TBM karşıt ayna basıncı hesaplamaları yapılmıştır. Güzergâhtan geçiş sırasında karşılaşılan problemler kaydedilmiştir ve TBM veri toplama sisteminde kaydedilen raporlar incelenip her bir ring için ölçülen tork, baskı (itme) kuvveti, net kazı hızı ve tünel aynasına gelen arazi basınçları irdelenmiştir.

Yapılan incelemeler sonucunda karşılaşılan problemlerin ve TBM performansını etkileyen parametrelerin başında jeolojik koşulların geldiği tespit edilmiştir. Elde edilen bulgular çerçevesinde zor zemin şartlarında TBM performansının düştüğü görülmüş ve bunun nedenlerinin başlıca kaya ile zeminin karışık olduğu tünel aynası koşulları, dayk-anakaya dokanak zonları, stabilite problemleri ve aşırı keski tüketimine bağlı duraklamalar olduğu tespit edilmiştir.

1. INTRODUCTION

Rapid increasing of human population in urban areas results in the need of new infrastructure systems such as metro tunnels, water tunnels and utility lines. The growing demand made by consumers on the (limited) available space, combined with higher demands on the quality of newly realized structures, forces decision-makers to make extended use of the third dimension, and in particular the available subsurface space. This need brings engineers to make new solutions to difficulties of construction in urban areas.

Too often the effectiveness of construction management is criticized for contractual problems that arise during excavation. In most instances the roots of such problems can be traced back to the initial planning. Unrealistic performance expectations leave a little opportunity the contractor to manage construction effectively.

Prediction of excavation rates is the most critical element in the planning of a TBM project. The critical path of tunnel project will normally be intimately linked to excavation activity and adherence to cost and schedule will implicitly require the TBM to mine as planned. For the project as a whole to be successful, the planning process must provide for a realistic evaluation of TBM performance expectations that reflects both the known and unknowns of the excavation process. For this purpose models are developed by researchers in order to estimate TBM performance by using geological data. These models fits well with the constant geological formations.

Today, there are few ground conditions where the TBM performance prediction models are not suitable. A certain and in some cases serious limitation is, however, represented by varying ground conditions, such as dykes, faults, weakness zones or even soils/soft rock, combined with hard rock. This is commonly referred to as mixed-face conditions. Previous experiments shown that penetration rate is not the only factor affecting the performance of TBM considering TBM as tunneling plant not only as machine.

In this study, TBM performance was analyzed in difficult ground conditions, especially mixed face conditions, to have a better understanding on both behavior of the ground and machine during construction. The Hasanpasa Crossroad–Otosan Land alignment of Kadikoy–Kartal Metro Project was selected for research area.

The excavation activities were observed in the field. Field performance of the TBM was analyzed based on the data recorded on the TBM data logging system. Analyzed TBM performance parameters included operational thrust and torque requirements, instantaneous penetration rates and disc cutter consumption rates. Stability of the face was analyzed and the required face pressures were estimated by using a few of the theoretical methods. These parameters were linked to the geological properties of the alignment such as rock-soil mixed ground at the face and weak contact zones between dykes and main rock.

2. MECHANIZED TUNNELING IN URBAN AREAS

2.1 Tunnels in Urban Areas and the Related Challenges

The world's cities today are closed networks of transportation systems, utilities, and residential and industrial buildings. Millions of people live and work in such major cities, often in restricted and congested spaces. And, according to various studies (Ray, 1998), the world's urban population is expected to rise significantly, such that in fifty or so years many cities of today will grow in size from small to medium, medium to large, and large to mega.

Such trends will constantly demand a proper allocation and redistribution of the limited urban space to the various urban functions, both existing and new. A good summary, on the challenges posed by the world population and consequently the planning needs, can be found in the keynote lecture by the current President of ITA presented to the International Seminar on Tunnels and Underground Works in Lisbon As already demonstrated by the development worldwide in the last century, the resolution of the constant conflict between the demand (for infrastructures and services) and the supply (limited urban space) has often led the planners, politicians, architects, and engineers to consider tapping a seemingly invisible resource: the underground space.

In fact, underground spaces have historically been created in urban areas and mainly used to host traffic ways (streets, subways, railways) and public-service utilities (water supply ducts, sewers). Nowadays, the underground space is created for storage, security, commerce, underground electric stations, and various other purposes. An exhaustive view of the reasons for going underground can be found in the well-known booklet published by ITA (2002), entitled "Why go underground?". In the same booklet it is stated that "whatever the type of underground structures in an urban environment, they all aim to free surface space for more noble human needs, improving the living conditions of our cities. In the case of interurban links, long-length tunnels are justified by saving time and reducing costs (shorter journeys and less energy consumption), maximizing safety and minimizing environmental impacts”.

An analysis of the increasing demand perspectives of underground structures worldwide was made by Assis (2003), including a focus on the methods for their construction. In terms of the large-scale development and use of the underground space in the future, the typical urban functions such as transportation (through infrastructures like metros, highways, motorways, railways), utilities (water-supply, sewage, telecommunication, heating), and safety (flood protection) make up a promising group of incentives to use underground space. Another strong reason for putting these typical urban functions underground is to reduce their visual impacts, limit the acoustic pollution, and preserve the surface environment. Furthermore, for the underground development of structures of long extent, tunnelling is a must, independent of the digging technique used.

According to Pelizza (1996) “going underground is not an obligation: it is a reasonable choice from among the various solutions, and one that is influenced by a multiplicity of social and economic factors, the perfecting of which should lead to an improved quality of life”.

“In an ever more populated world, the use of underground space will certainly be one of the most useful instruments for conserving – and, if possible, for improving – the quality of life that is compatible with the needs of human beings”. The ever-increasing need for tunnels in urban areas is, in turn, one of the most efficient prime movers for the development of tunnelling technologies and, in particular, for mechanized excavation. In the latter case, the continuous search for fast and safe solutions in any conditions has significantly increased the feasibility limits for the realization of tunnels, for example, see Figure for the biggest (for the moment) Earth Pressure Balance (EPB) shield in the world.

In fact, today it is possible to excavate tunnels rapidly under small overburden, in loose ground, and under a water table, minimizing at the same time the ground settlements, and not causing significant disturbances to the surface activities in an urban centre area, thanks to the great developments in mechanized excavation technologies achieved in the last 30 years.

Compared with the conventional excavation methods, the clear increase in the use of mechanized techniques in urban tunnelling is mostly due to the following advantages:

• The work environment is "factory" like, not the "mining" type, and characterized by higher levels of comfort and safety for the workers.

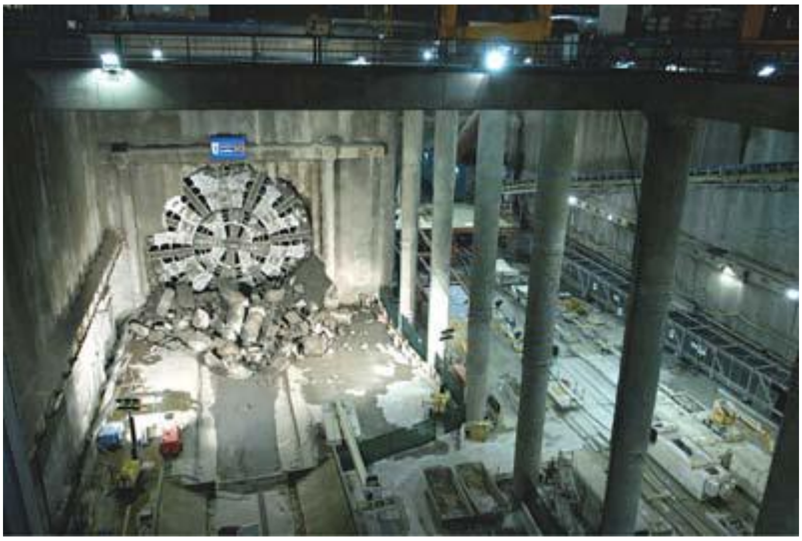


Figure 2.1.: The biggest (in 2007) EPB Shield: the 15.2-m diameter Herrenknecht S-300 for Calle 30 project, Madrid

• The rapidity and industrialization of the construction cycle with possible automation for all the working processes and activities: excavation, lining, transportation, mucking out and, consequently, the shortening of the construction period.

• The possibility to measure and keep under control the principal construction parameters like the quantity of excavated material, the support-pressure applied on the excavation face, the over-break, the ground movements around the tunnel periphery, and the surface settlements.

• The low noise levels, limited dust dispersion in the environment, and minimum disturbance to the water table.

• The use of pre-cast segments to line the tunnel, facilitating the control of the construction phases, and enhancing the quality of the finished work.

• Often, the overall cost is lower than that of the conventional method. Moreover, it is possible to state that, in some special cases, the desired infrastructures, such as the crossing of a railway line under important historical centers, could not even be conceived or built in urban areas, except through mechanized excavation, because of better control of a series of high level risks involved.

It is observed that, in cases other than those of micro-tunnels (with diameters of 2–3 m), which are increasingly common in cities for the installation of new subsurface utility networks, tunnel excavation by mechanized shields has been mainly used for construction of transport infrastructures including light-rail metro systems.

Applications of mechanized excavation to other transport systems, which are less frequent today but surely of great interest in prospect, concern the construction of road penetrations and by-passes in urban areas, obviously when they are sufficiently long to justify recourse to mechanized tunnelling. Some recent and significant projects in this category include the ring-road around Moscow involving a 2.2 km tunnel excavated using a 14.2-m diameter TBM, and the Madrid M-30 project involving a 3.6 km long, twin-bore tunnel excavated using two 15.2-m diameter TBMs, the largest shield machine in the world in 2007.

Even in the field of water supply structures, the excavation of large-diameter, collector tunnels using TBMs is also becoming quite common. For example, the 1.9 km long Ivry-Masséna tunnel (TIMA) in Paris, the largest and deepest rainwater accumulation tunnel in Europe, was excavated with a 7.9-m diameter TBM.

There are other important urban tunnels that perform multiple functions, such as the SMART system in Kuala Lumpur where a 13 km long tunnel, excavated using two 13.3 m-diameter TBMs, serves both as a road tunnel for traffic deviation and a storm-water diversion duct to mitigate the high risk to flooding in the centre of the city . It should be pointed out that the demand for mechanized excavation is also increasing for installation of gas-supply and waste-disposal pipelines in urban area.

In summary, there is an ever-increasing potential for application of mechanized tunnelling in urban areas because, in theory, any linear infrastructure that can be developed on surface can also be readily developed underground, perhaps also with reduced life-cycle costs [1, 2].

2.2. The Particular Challenges of Urban Tunnelling

The development of infrastructures and the related underground space in urban areas must, in particular, meet the requirements for sustainable development: the challenge for Owners, Planners, Designers and Constructors, is to build both for the future and for today in such a way as to disturb as little as possible the daily activities of the cities, guaranteeing at the same time the quality, safety, time, and cost targets of the development.

In comparison with tunnelling in open-space rural areas, tunnelling in urban areas has some major and peculiar characteristics and constraints as listed below.

• The layout is strictly related to the final use and to the functional aspects of the tunnel. Hence, in spite of the apparent "topographic freedom" of the 3-dimensional planning, many constraints intervene to limit the alignment location, resulting in frequent and often unavoidable, potential interferences with buildings at the surface, underground utilities, and other pre-existing underground structures.

• Urban tunnelling is generally carried out at a shallow depth for functional and cost reasons. This gives rise to a series of consequences in terms of geology, subsurface, and impacts.

• The sub-surface at shallow depth often consists of loose soils, alluvial deposits, or manmade fills. The poor quality of the ground is one of the key factors for the tunnel design and construction control.

• The immediate underground level of the sub-surface is reserved to the installation of underground utilities that have to be identified and assessed, in terms of the risk of potential damage caused by tunnelling-induced settlements, and subsequently diverted and relocated permanently, if needed.

• In many parts of the world, the cities have an important historical background. Hence, in the immediate underground level of the sub-surface, important archaeological features could be hidden; these have to be recognized and dealt with, especially when planning the tunnel accesses or the service shafts.

Urban tunnelling at shallow depth usually induces settlements at the surface, even under the most strictly controlled tunnel-driving operations. The magnitude of settlements is a function of many interrelated factors: the quality of the ground; the behavior of the ground when tunnelling; the control of the tunnel face and tunnel section stability during construction; the presence of underground water and the hydrogeological regime; etc.

• The response of buildings and utilities to tunnelling-induced settlements has to be rigorously assessed both in normal and anomalous conditions, i.e. considering a set of potential scenarios.

• To put the maximum effort in reducing, as much as reasonably possible, the occurrence of anomalous conditions (excessive settlements and/or collapses) is a 'must'.

• The high level of interaction with the life above the surface has to be analyzed and solved carefully with solutions that can be accepted by the public without causing major disturbances. This implies an appropriate plan for the temporary diversion of the traffic, an accurate planning of worksite areas, a particular attention to control of dust and noise emissions, and a special care for safety issues.

• An extensive and redundant geotechnical, structural, and environmental monitoring plan is required, which needs not only extra and direct money input but also additional human effort.

• Urban tunnelling is generally related to the implementation of strategic infrastructure projects, which have a high political relevance. The politicians and the financiers of the project, together with the public, will all demand for a certainty of the project budget in terms of cost and duration.

Finally, the public opinion can heavily influence the development of the project, because it is virtually in everybody's backyard. Hence, the public should be kept informed correctly and offered the possibility to voice its opinion and give input to the project. Further, every effort should be made to always guarantee the public safety, so that the project can be accepted by the public and the huge negative effects of a potentially adverse public opinion are minimized [1]

2.3 The Correct Approach to Success

Under normal conditions, the fundamental goal for the design and subsequent construction of a tunnel is to assure that the work is realized within the budget and constraints of time and cost, is stable and durable over a long time, and corresponds to the technical specifications and requirements of the Client. These objectives are really very important, but they are not comprehensive enough for tunnelling in a city. Indeed, in an urban environment, it is also necessary to take into account a set of completely distinct elements or factors that frequently influence the choice of the design and construction. The presence of these elements requires that particular attention be paid to the rules like:

• Disturb as little as possible the integrity of the ground surface and the built-up environment above. Take into account all existing structures and all underground services, such as the sewage system and superstructures.

• Respect the limits specified in the design for surface settlements, which is a function of the type of ground and the pre-existing conditions (or coefficients of vulnerability) as well as the construction technique to be used.

• Avoid absolutely the collapse of the tunnel face, which can cause property and/or personnel damage.

In fact, a potential tunnel collapse in a rural, non built-up area, a hazard which should be avoided whenever possible, can cause, at maximum, a stoppage of the works with a variable duration depending on the time required to recover the situation and implement the measures necessary to allow the restart of the tunnel. However, a collapse due to tunnelling in a densely populated urban area can have a very serious impact on public opinion and, in the extreme case, it may cause damage to properties and people or, even worse, when fatalities are involved it can lead to a complete blockage of the project for months or even years. Clearly, the risks related to such hazards need to be minimized, when it cannot be possible to avoid them totally, choosing alternative solutions. Elements are required at the design stage:

• Experience to define (i.e. identify and quantify) the project risks and to propose technical layouts and technological solutions, for the conception of the infrastructure, which are consistent with the necessity to reduce the risks.

• Particular approaches and methodology to systematically and consistently analyze and manage the project risks throughout the design process.

• Special thinking and innovative tools that can facilitate the decision-making process, for example, by providing an easy access to, and a timely availability of, all the collected monitoring-survey data and the investigation results to the involved specialists.

It is clear that the first response to risk is the selection of the appropriate construction method. Considering the huge technological improvements achieved in the last decades, the method of mechanized shield tunnelling can make construction feasible and, at the same time, minimize the undesirable interferences. However, mechanized shield tunnelling is not a risk-free technology, even though it is a modern and advanced technique, and the potential risks cannot be ignored.

There are also occasions when the risk analysis approach assumes a strategic importance for overcoming the difficult or unforeseen geological conditions. The need (and use) of the risk analysis approach is exemplified by the construction of the Porto Metro in Portugal. In this example, the potential to encounter fractures filled with water and loose materials in the class II granite could not be ignored and therefore, the correct approach for minimizing the risk of instability of the ground surface (even in granite) was to apply support pressure to the face.

In general terms, the logic for the risk analysis (regarding the tunnel construction in urban areas) would suggest that, even if the probability for a negative event (such as a fall or a chimney type of incident under a structure or under a street crossing) can be reduced to very low values using adequate measures, the resulting damage (including the potential loss of human life) can render the level of corresponding residual risk absolutely unacceptable. Therefore, it is often important to take additional, precautionary, mitigation measures such as consolidation of the foundation of a structure or a temporary closure of traffic in the affected area, or both. [1]

3. MECHANICAL EXCAVATION SYSTEMS

3.1. Historical Development and Future Challenges

Tunnelling developed rapidly during the industrialization at the start of the 19th century with the building of the railway network. In hard rock, this was by drilling and blasting The first stage of the developing mechanization of tunnelling therefore was the development of efficient drills for drilling holes for the explosive. There were also attempts to excavate the rock completely by machine.

The story of the development of the first tunnel boring machines contains, besides the technically successful driving of the Channel Tunnel exploratory tunnels by Beaumont machines, many attempts, which failed due to various problems. Either the technological limits of the available materials were not observed or the rock to be tunneled was not suitable for a TBM. The early applications were successful where the rock offered the ideal conditions for a TBM.

The first tunneling machines were not actually TBMs in the true sense. They did not work the entire face with their excavation tools. Rather the intention was to break out a groove around the wall of the tunnel. After this had been cut, the machine was withdrawn and the remaining core loosened with explosives or wedges. This was the basic principle of the machine designed and built in 1846by the Belgian engineer Henri- Joseph Maus for the Mount Cenis tunnel The machine worked with hammer drills chiseling deep annular grooves in the stone, dividing the face into four 2.060.5 m high stone blocks. Although this machine demonstrated its performance capability for two years in a test tunnel, it was not used for the construction of the Mount Cenis tunnel because of doubts about the drive equipment. The compressed air to power the drills was to be provided by water powered compressors at the portal and fed to the machine through pipes. Considering the 12,290 m length of the tunnel, Maus expected that only about 22 kW of the 75 kW generated would arrive at the machine. It also turned out that the material used at that time could not resist the wear during tunneling. The result would have been increased wear of the bits. Despite these problems Maus assumed an average advance rate of 7 m, or considering downtime for cutter change, 5 m per day.

The American Charles Wilson developed and built a tunnel boring machine as early as 1851, which he first patented in 1856(Fig.2.1.1.). The machine had all the characteristics of a modern TBM and can thus be classified as the first machine, which worked by boring the tunnel. The entire face was excavated using disc cutters, which Wilson had already developed in 1847 and applied for a patent for. The tools were arranged on a rotating cutter head and the thrust required for cutting was resisted by pressure sideways against the rock. In comparison with modern TBMs, the integration of a rotating mounting for the disc cutters stands out. The mounting plate was arranged with its rotational axis perpendicular to the tunnel centerline in the cutter head holder, which combined with the rotation of the outer cutting head to cut a hemispherical face. Wilson's machine underwent various tests in 1853. After advancing about 3 m in the Hoosac tunnel (Boston, USA), the machine proved, because of problems with the disc cutters, unable to compete with the established drill and blast method [2].

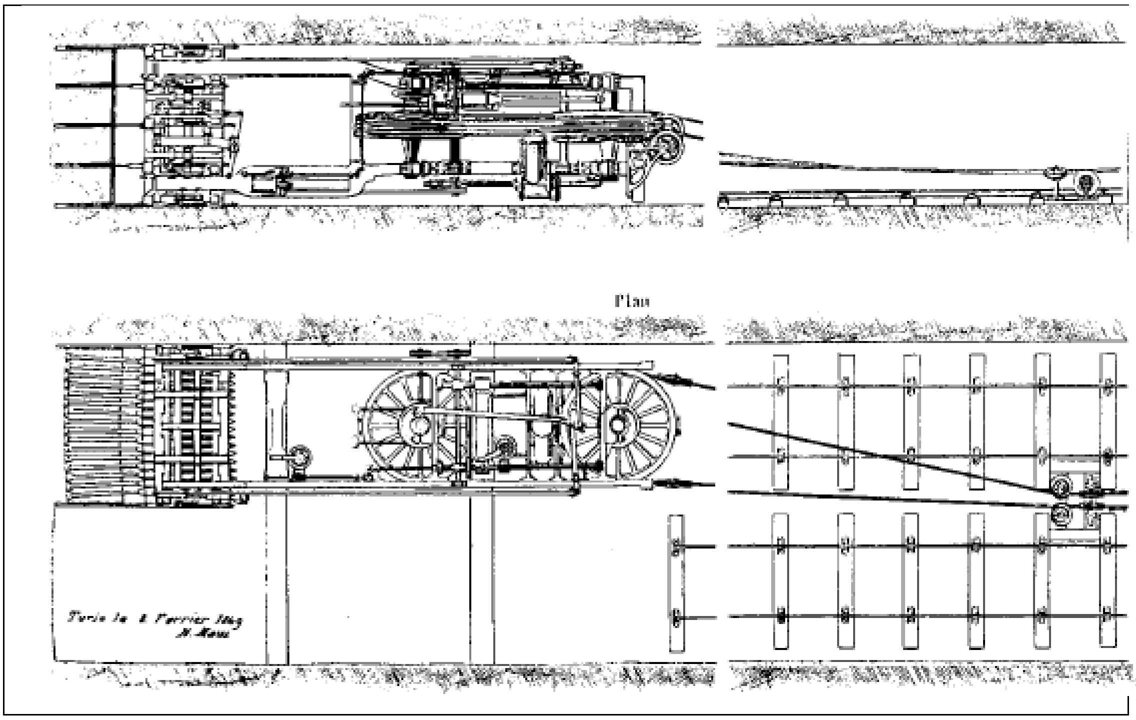


Figure3.1.1: Charles Wilson machine [2]

After his experiences with the TBM at the Hoosac tunnel, Wilson applied for a patent in1875 for an improved version of the machine (Fig 3.1.2). This was based on a completely new design of cutting head; no longer was the entire face to be excavated with cutting tools, but only an external ring and a central hole. This was to be achieved by mounting disc cutters at the outer rim and the rotational axis of the cutting wheel. After reaching the maximum cut depth, the machine had to be withdrawn to enable the remaining core to be loosened using explosives. The advantage was the precise profile of the excavation. This type of excavation with outer groove and central drilled hole proved to work well and was also used for other early tunnel driving machines like that of Maus, and this type of excavation has also been used from time to time since

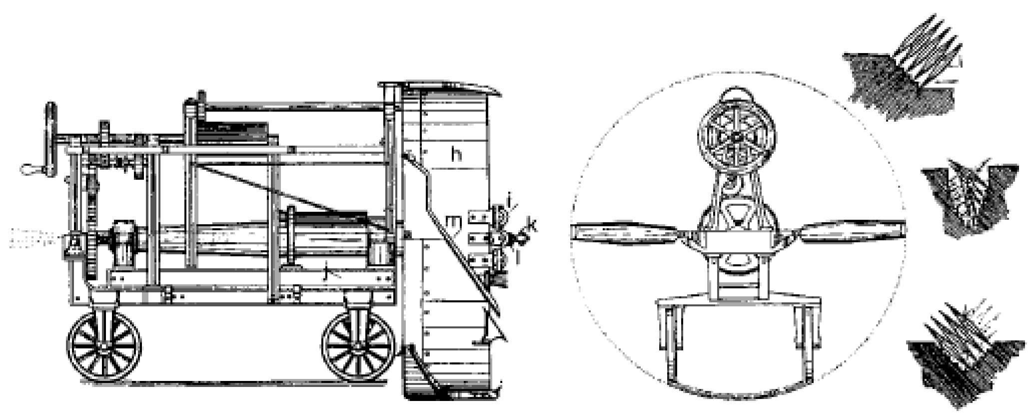


Figure3.1.2: Hoosac Machine [2]

Also in 1853, the same year as Wilson was testing his first machine in the Hoosac tunnel the American Ebenezer Talbot developed a tunnelling machine, which worked using disc cutters and a rotating cutting wheel. But this construction had the disc cutters arranged in pairs on swinging arms on the cutting wheel (Fig. 3.1.3.). The combination of the rotation of the cutting head and the movement of the cutting arms enabled the excavation of the entire face. Talbot's machine failed in the first tests boring a section of diameter 5.18 m. Looked at with modern eyes, it is possible to recognize in the arrangement of the disc cutters on cutting arms parallels to the System Bouygues tunneling machines used in the 1970s.

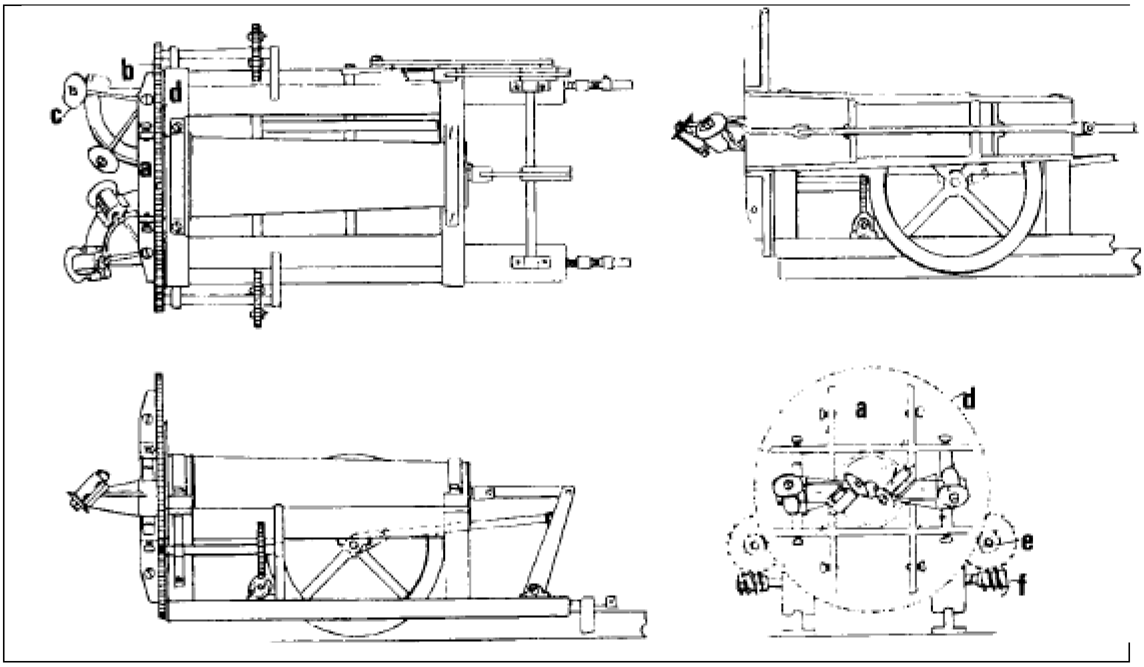


Figure3.1.3: Tunneling machine with drilling head 1853 [2]

Cooke and Hunter (Wales) proposed an entirely new system with their patent from 1866 Instead of a cutting wheel turning about the tunnel centerline, three drums rotated about a horizontal axis transverse to the tunnel. The central drum had the largest diameter and ran ahead of the others, while the outer drums extended the cross section. The excavated section had a box shape with right-angled extensions. The direction of rotation was meant to clear the muck from the face during boring. The machine was never built, but the idea of a rotating extraction drum was found again fifty years later in tunnelling machines like the Eiserner Bergmann (Iron Miner)

After Frederick E. B. Beaumont had already applied for a patent in 1863 for a tunnel ling machine equipped with chisels and used this unsuccessfully for the construction of a water tunnel, he applied in 1875 for a patent for a tunnel boring machine with a rotating cutting wheel.

The cutting wheel consisted a number of radial arms mounted on the end of a horizontal shaft. The tapered cutting arms were fitted with steel bits. The tip of the drilling bit formed a large conically ground chisel. The driving force was to be produced by a hydraulic pump driven by compressed air.

This patent was taken up by Colonel T. English and further developed for his own machine, for which he applied for a patent in 1880 .There were cylindrical holes in the cutting arms for the drilling tools, into which chisel bits were screwed. The new idea of this construction was that the bits could be exchanged without having to with draw the machine from the face. The arrangement of the bits on the two cutting arms was designed to cut concentric rings into the working face, so the remaining rock between the grooves would break off during cutting. A lower frame formed the base frame of the machine with equipment to carry away the muck and the drive for the drilling head. An upper frame held the actual drilling equipment, which was pushed forward by a hydraulic cylinder. So it was possible for the first time to push the cutter head forward without releasing the bracing of the machine to the tunnel walls. This system allowed high blade pressure and is still a principle of modern TBMs.

Beaumont built two machines to the patent of Colonel T. English in 1881 and used them to drive the Channel Tunnel (Fig.2.1.4.) The machines worked there very success fully from 1882 until 1883, when the work was stopped for political reasons. Altogether 1,840 m were driven on the French side and 1,850 m on the English side. The maximum daily advance rate was 25 m; a considerable achievement for that time There was no further application of tunnelling machines in the next decades. They were, however, successfully used in mining for cutting relatively soft rock. In the first half of the 20th century, tunnelling machines were used for driving galleries in potash mines. The first version from 1916/1917, called the Eiserner Bergmann, had a rotating roller fitted with steel cutters as a cutting wheel, which on account of its dimensions produced rectangular sections.[3]

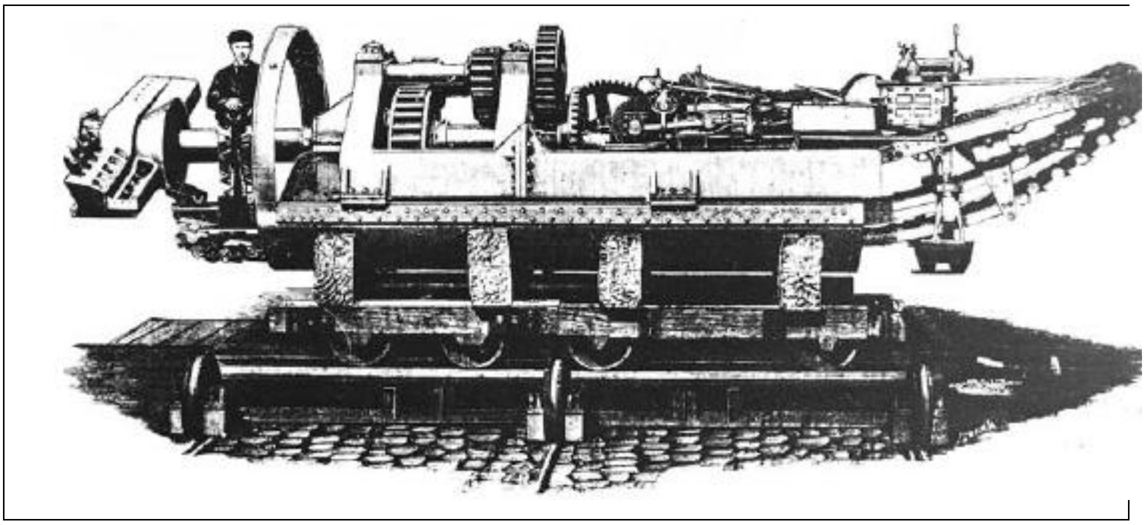


Figure 3.1.4: Tunnel boring machine by Beaumont, Channel Tunnel [3]

The next generation of gallery cutting machines built by Schmidt, Kranz and Co. from 1931 was more successful. The machine consisted of the main components drill carriage, bracing carriage, cable carriage and loading band The three-armed cutting wheel was fitted with needles and achieved on average advances of 5 m per shift. Five men were needed to operate the machine. The disadvantages of this machine which was also used in Hungarian brown coal mining were considered at the time to be the size, the weight, the poor mobility and the time wasted bringing the machine back. In practice, the machine was used for quickly driving investigation and ventilation headings. The similarity to the TBM built by Whittaker for the Channel Tunnel in the1920s is noticeable (Fig. 2.1.5). This achieved an average advance speed of 2.7 m/h in a test heading in the lower chalk near Folkestone.

The breakthrough to the development of today's TBMs did not occur until the 1950s, when the first open gripper TBM with disc cutters as its only tools was developed by the mining engineer James S. Robbins. Preliminary tests driving the Humber sewer tunnel in Toronto showed that, with only disc cutters and with considerably greater working life, the same advance performance could be achieved as with the intended combination of hard metal cutters and discs of the former TBM. Using this TBM in the Humber sewer tunnel, advances of up to 30 m/d were achieved in sandstone, limestone and clay Mechanical tunnelling at this time was primarily concentrated on stable and relatively soft rock. With the growing success of Robbins, further American manufacturers like Hughes, Alkirk Lawrence, Jarva and Williams began building tunnel boring machines. Machine types still current today like the main beam TBM or the kelly TBM had their origins at this time

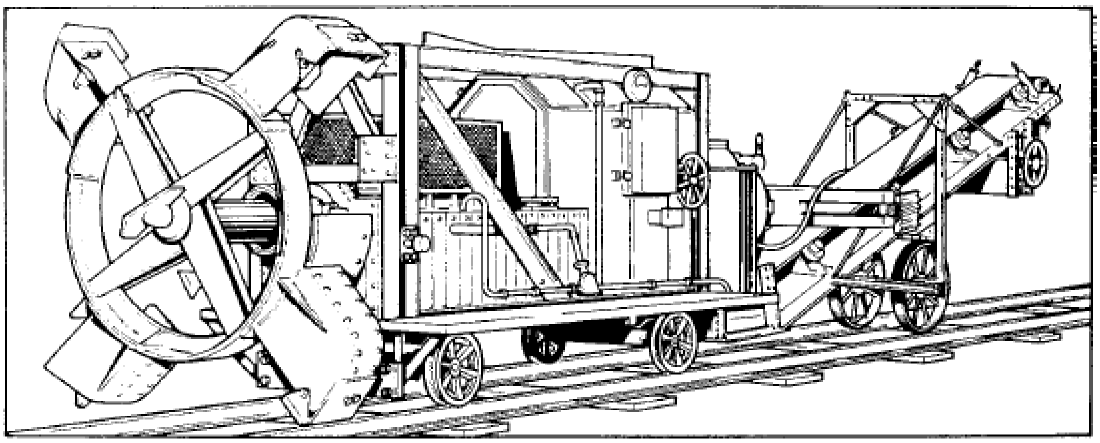


Figure 3.1.5: Tunnel boring machine by Whitaker [2]

After a slight delay, the development of tunnelling machines was also taken up in Europe. At first, however, different avenues of development were followed. Based on experience in Austrian brown coal mining with the Czech Bata machine, The Austrian engineer Wohlmeyer developed undercutting technology with rotating milling wheels (Fig. 2.1.6a). This technology did not catch on, and nor did that used by the Bade company with the cutting head divide into three contra-rotating rings fitted with toothed roller borers, which were already outdated at the time of the trial (Fig. 2.1.6b).Both types of machine were unsuccessful in tests in the hard rock of the Ruhr, although other Wohlmeyer machines were used successfully for the Albstollen heading and in the subsidiary headings of the Seikan tunnel Undercutting technology has been used and further developed over many decades by various manufacturers like Habegger, AtlasCopco, Krupp, IHI and Wirth because of the low thrust force required and the ability to drive non-circular cross-sections. The separation of the Bade TBM into a front section with cutting head and a rear section, which was hydraulically braced by four large pressure plates against the tunnel sides to provide reaction for the boring head carrier, and which is withdrawn after the completed travel of the advance cylinders however recognizable in modern double shield machines [2].

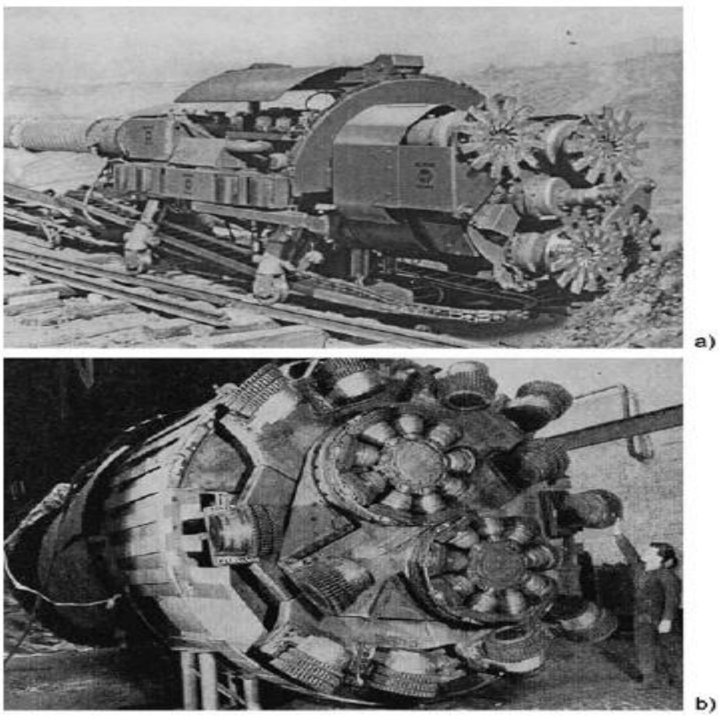


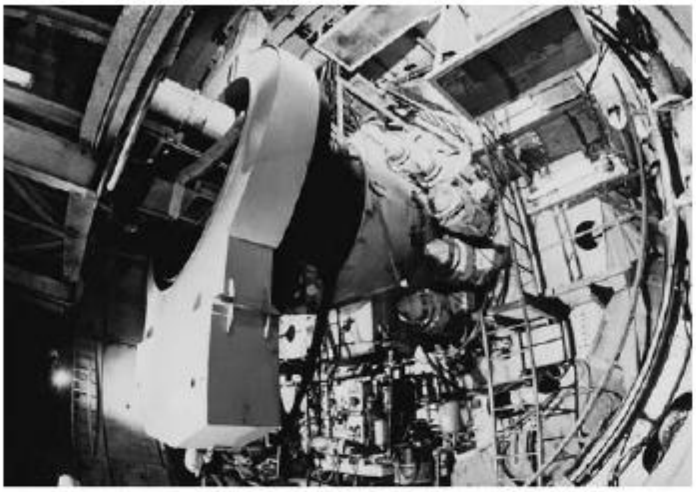
Figure 3.1.6: a) Wohlmeyer gallery cutting machine [2]

b) Tunnel boring machine operating in coal mine

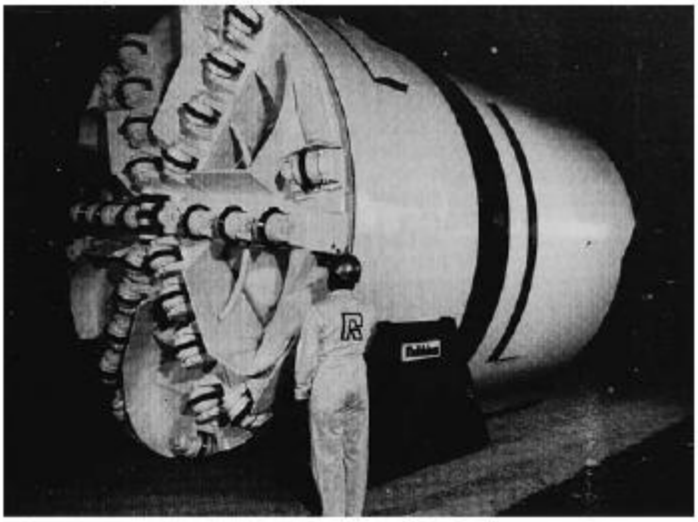
In the 60s, German manufacturers like Demag and Wirth began building tunnel boring machines of North American type. These machines were mainly intended to bore hard rock. Drilling tools from deep boring technology like TCI or toothed bits were mounted on the drilling heads. The developing technology for hardening the disc cutters enabled the use of this type of tool in really hard rock. At the end of the 60s, inclined headings and large tunnel sections were driven for the first time using the reaming method, the development of reamer boring being closely associated with the Murer Company

Progress in the 70s and 80s was directed towards driving in brittle rock and the enlargement of tunnel sections, with the consideration of the stand-up time of the soil/rock becoming particularly important. Encouraged by the successful implementation of a gripper TBM for the Mangla dam project in 1963 with a diameter of 11.17 m, a gripper TBM was also used for the construction of the Heitersberg tunnel (1 10,65 m) in Switzerland in 1971. The work necessary to secure the rock with steel installation anchors and mesh-reinforced shotcrete however made the hoped-for advance impossible. The required adaptation to the large cross-section was first achieved in 1980 by the modification of the Robbins gripper machine from the Heitersberg tunnel by the

Locher und Prader company to a shielded TBM with segmental lining for the advance of the Gubrist tunnel (1 11, 50 m) (Fig. 2.1.7a). Robbins and Herrenknecht have made shield machines of this type in diameters from 11±12.5 m. At the same time, Carlo Grandori developed the concept of the double shield TBM and in collaboration with Robbins, put it into practice for the building of the Sila pressure tunnel (1 4,32 m) in Italy (Fig. 2.1.7 b). The main intention of the development of this machine was to make the gripper TBM, which had then already proved very effective in geological conditions, more flexible for use in heterogeneous rock conditions. Since their first use in 1972 and the successful modification of this type of machine, double shield TBMs with customized segmental lining designs have achieved high advance rates under favorable rock conditions and have been made by all the well-known manufacturers, mainly in the medium diameter range. The capability of the double shield TBM design was demonstrated impressively at the end of the 80s in the chalk of the Channel Tunnel, which is favorable for tunnelling.



a)



b)

Figure 3.1.7: a) Single shield machine [2]

b) Double shield machine

Alongside the development of the TBM with shield, the manufacturers of open gripper TBMs began to investigate possibilities of improving their machines to enable any necessary lining to be installed earlier. Shotcrete around the machine was tested. The state of progress with large diameter TBMs today is the installation of lining elements immediately behind the boring shield or partial areas of the shield and the systematic installation of rock anchors. With smaller tunnel boring machines, the body of the machine obstructs the installation of lining around the machine using mechanical equipment with the result that where lining has to be installed quickly, this has to be done by hand with a corresponding reduction of the advance rate

The development of gripper TBMs at the moment is to enable the early mechanical installation of the lining around the machine in order to improve the boring performance by reducing the time taken to install the measures to secure the tunnel sides. Further reductions of the boring time would only lead to a marginal increase of advance rates, as today's TBMs already have availability rates of 80±90%.

For future development of tunnel driving with gripper TBMs, it is necessary to adapt the design of linings intended for conventional tunnelling to the special requirements of TBM tunnelling. The fear of a shield TBM jamming fast, which is repeatedly expressed, and the problem of rigid lining also demand innovative developments although no such case is known for relevant single-shield TBMs[3]

3.2. A Short History of Shield Tunneling

Sir Marc Ismambard Brunel, a British engineer and inventor was born in France. In 1825 Brunel began construction of the Thames Tunnel, the first tunnel in which a shield was used. Expected to take 3 years, it opened 18 years later in 1843 as a foot tunnel, but later became a part of London Underground as it known today.
Sir Marc's son, Isambard Kingdom Brunel an engineer who, in the UK at least, is still an inspiration for many young civil engineers became resident engineer for the Thames Tunnel construction works. The shield consisted of 12 3 – foot wide vertical sections with three levels in each, forming compartments in which a man could work (Figure 2.2.1).Small angled jacks at the front of the shield held horizontal timber boards in position. A man working in each compartment removed a board one at a time and excavated a few inches of earth, replacing the jacks against the board in forward position. When the full face had been excavated, and the whole cycle was repeated. During construction the tunnel flooded many times and the death toll was formidable. Brunel nearly lost his life pursuit of his work.

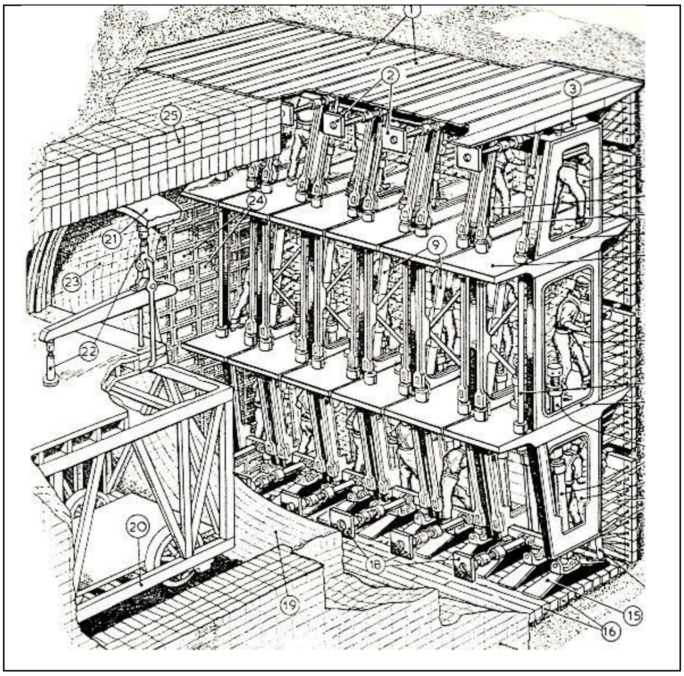


Figure 3.2.1: Shield used in construction of Themes tunnel (UK) 1825 [2]

In 1864, Peter Barlow patented a cylindrical tunneling shield and in 1869 used it to construct a pedestrian tunnel under Thames at Tower Hill. Compared with Brunel's 18 year marathon at Wapping 50 years earlier, the construction took just 6 months. The resident engineer for the scheme was JH Greathead who, over the next 15 years, designed various modifications into basic process so that the shield became as the Greathead Shield.

Figure 3.2.2: Greathead Shield (UK), 1879 [2]

The Greathead Shield consisted of a cylindrical shell with a sealable bulkhead. In clays it was used in open face mode with miners advancing through the bulk head to mine clay. In waterlogged sands the bulk head was closed and water was piped in and the excavated sands piped out similar in principle to a modern slurry tunneling machine. In mixed face conditions miners passed through an airlock built into the machine and worked under compressed air. It really was state of the art technology of today. Much of the London Underground railway tunnels were built using a Greathead shield.

The hydraulic shield (figure 3.2.3) as it was developed during construction of the Steinway Tunnels(now known as the Queensboro Tunnel), offered ground support and protection against flooding by the inclusion of a solid bulkhead that allowed the excavation of the tunnel to be carried out in compressed air. The incidence of flooding and face collapse was reduced considerably but, unfortunately the fatality rate did not decrease because workers spent long periods of time in compressed air without undergoing proper decompression procedures.

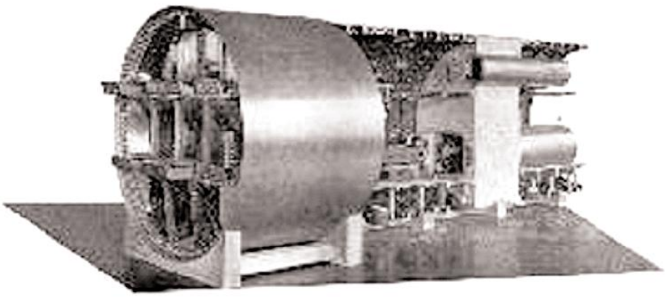


Figure 3.2.3: Hydraulic Shield Used in Construction of Steinway Tunnel, 1907 [2]

The tunneling shield developed very slowly over the following 60 years until John Bartlet patented the bentonite tunneling machine in England in 1964. The 1964 patent started a revolution in the tunneling industry. The Japanese began experimenting with slurry machines in the mid 1960s, the British in the early 1970s and the Germans with hydro-shield in the mid1970s. Difficulties in excavating ground containing boulders, and cleaning the slurry established limitations to the slurry method. In response the Japanese started developing the earth pressure balance (EPB) tunneling machine in the mid 1970s.The predominant ground conditions in the coastal areas of Japan suited the early development of EBP technology.

3.3. The Principle of Earth Pressure Balance TBM Tunneling

EPB tunneling is designed to reduced, as much as possible, the ground loss ahead, above and behind the TBM so that ground movements and surface settlements are maintained within acceptable limits. Together with controlling the volume of excavation, the management of tunnel face support pressure is of paramount importance for guaranteeing the successful operation of an EPB machine. The approach requires the application of a minimum effective pressure. In simple terms, it is required that the face support pressure be transferred from the bulk-head of the TBM to the face via solid particles (the excavated soil in the cutter head chamber) and not only through pore pressures or compressed air.

Ground movements likely to cause to property or structures above the tunnel are reduced while enabling the safe construction of tunnel linings within a completely sealed mechanized tunneling system. The sealed system balances the hydrostatic and active earth pressure in front with the forward advance and thrust of the TBM, and seals the tunnel against the inflow of ground and water during the excavation. This provides a safer working environment for the tunnel workers. With careful selection of soil conditioning agents, EPB TBM tunneling is an environmentally preferable method of excavation for soft ground tunneling where ground conditions are suitable.

The thrust force generated by the TBM is transferred from thrust rams located at the rear of the bulkhead onto the earth contained in the cutter head chamber, thus avoiding the uncontrolled intrusion of excavated ground into the chamber. The screw conveyor then extracts the excavated ground in the chamber. By balancing the thrust exerted by the machine and the rate of extraction of the excavated material from the cutter chamber to the screw conveyor, the pressure in the cutter chamber can be controlled. “Pressure loss” trough (along) also allows discharge to be at atmospheric pressure. This is referred to as excavating in pressurized mode, as opposite to unpressurized mode, where material can flow into the cutter head chamber in an uncontrolled manner.

The operator controls and adjusts the speed of the propulsion and the screw conveyor by directly monitoring and interpreting the readings of the earth pressures cells fitting within the cutter head chamber and the screw conveyor. Data related to rotation of the screw conveyor and propulsion system is transferred to a program logic control (PLC) system that is resident in the TBM. Continuous feedback from the earth pressure cells and forward pressures of the TBM thrust rams are provided to the PLC for interpretation, appropriate control commands or both.

For effective TBM operation, the cutter head chamber should remain full of a dense and incompressible material such that, in the case of faces instability (e.g., due to unexpected geological conditions), there is no void for the ground to collapse into the chamber. [3]

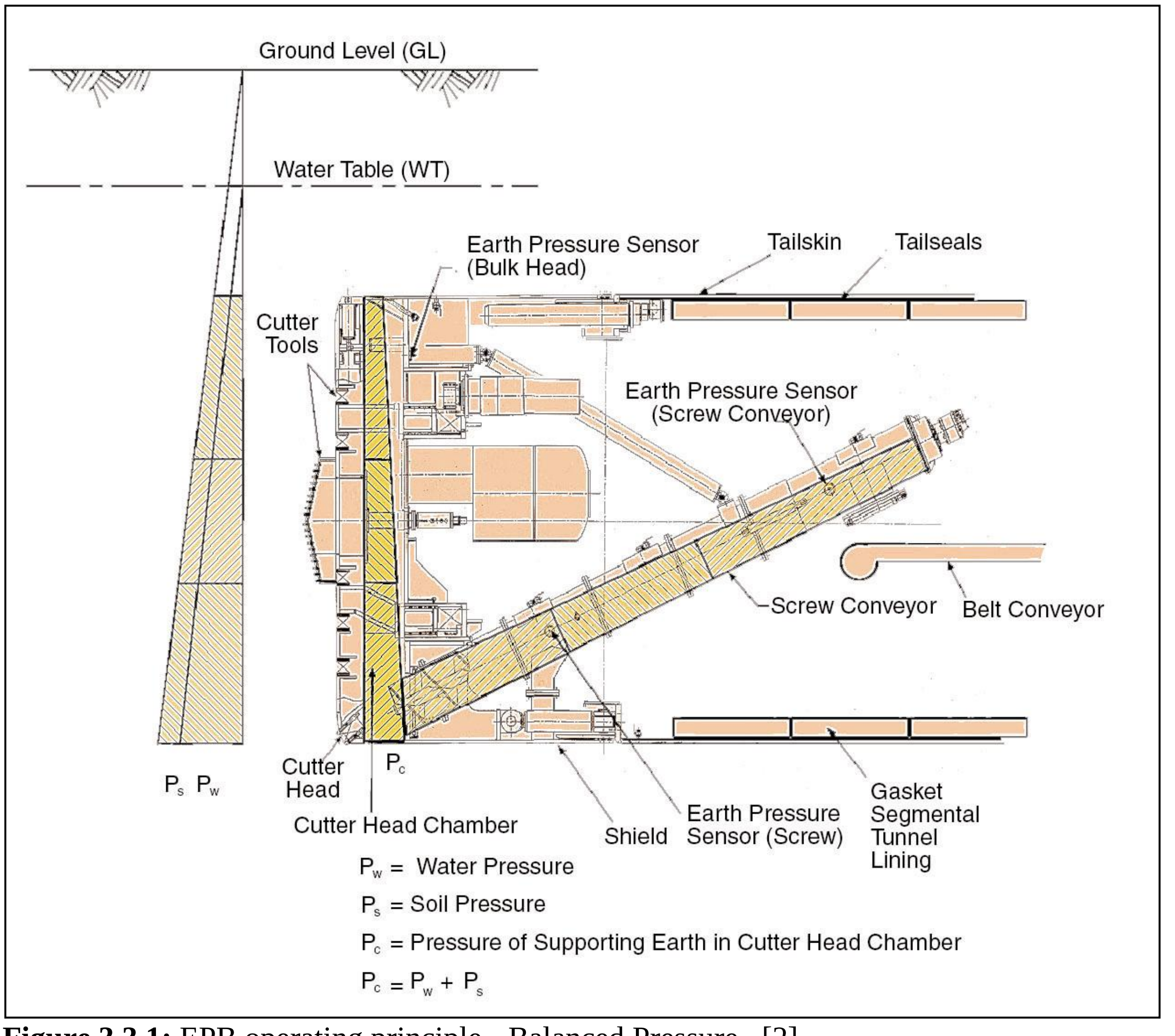


Figure 3.3.1: EPB operating principle - Balanced Pressure [2]

3.4. The Development of EPB TBM Technology

EPB TBM technology has advanced considerably over the last 20 years in Japan, Europe and Canada. The last 10 years have seen dramatic developments that enable EPB machines to cope successfully with difficult and potentially unstable ground conditions. This chapter looks at how the state-of-the-art in today´s soft ground tunneling technology developed over the last decade.

3.4.1. Canadian Development of EPB TBMS

In the early 1990s, Lovat EPB tunneling machines were being used on projects in Australia (Melbourne Sewer), Canada (St.Clair River), Germany (Mangfall Gallery) and Spain (Passilo Verde). Ground conditions on these projects varied between sands, shale's, soft clays, sandy silts and gravels. Means and methods of controlling water inflow and earth pressure on these early projects included the installation of flood doors (Figure 3.4.1.1) – now standard design on Lovat EPB TBMs) to provide protection against sudden inrush of water and control the inflow of material into the excavation chamber. The combination of a mucking ring and pressure relieving gates (Figure 3.4.1.2) provided the primary system of face control and spoil extraction.

The mucking ring was a 300-degree circumferential drum located within the cutter head chamber that surrounds the head of the primary conveyor (Figure 3.4.1.2). The primary conveyor, which was either a belt conveyor or screw conveyor, depending on the ground conditions and mode of operation, extended from the cutter head chamber through the bulkhead to the secondary conveyor, and transported the excavated material to the rear of the TBM. Loading plates mounted around the interior of the cutter head chamber help lift and load the excavated spoil onto the primary conveyor. The mucking ring directed the excavated material onto the primary conveyor.

The Lovat EPB TBM design incorporated hydraulically operated flood control at the face of the cutter head. The tunnel face remained isolated when the machine was not advancing by closure of the flood doors, which were also closed during replacement of the cutter head tools in order to maintain support to the tunnel face. As the flood doors were only used for isolation, they were either fully opened or fully closed. Leaving the flood doors fully opened when operating in earth pressure mode with screw conveyor permitted free flow of spoil into the cutter head chamber, therefore not creating a pressure drop across the cutter head.

As the TBM advanced forward, the screw conveyor auger was rotated and a guillotine door at the rear of the screw was opened to allow material from the cutting head chamber to discharge onto the secondary conveyor. The opening ratio of the flood doors and speed of the auger regulated the flow of material from the cutter head chamber and, therefore, the rate of advance of the machine. The flow of material from the screw conveyor could be regulated at any time by closing the guillotine door completely.

However, as was the case on the Melbourne Sewer Tunnel, replacement of the belt conveyor with a screw conveyor due to the large pieces of basalt and insufficient fine material to form a plug. As a consequence of the difficulty in controlling the inrush of sand and water into the TBM, problems occurred that resulted in damaged motors as well as difficulty in replacing cutter teeth.

This had a detrimental effect on penetration rates. The introduction of material locks and man locks (chambers in which men and materials could pass in and out of a compressed air chamber at the front of the machine) were one means of improving the management of these difficult mixed faces. Another improvement involved attaching skirts onto the primary and secondary conveyors so tunneling could continue under compressed air, if necessary. The seals on the pressure relieving gates were also improved.

Figure3.4.1.1: Flood doors in Closed and Open Positions [2]

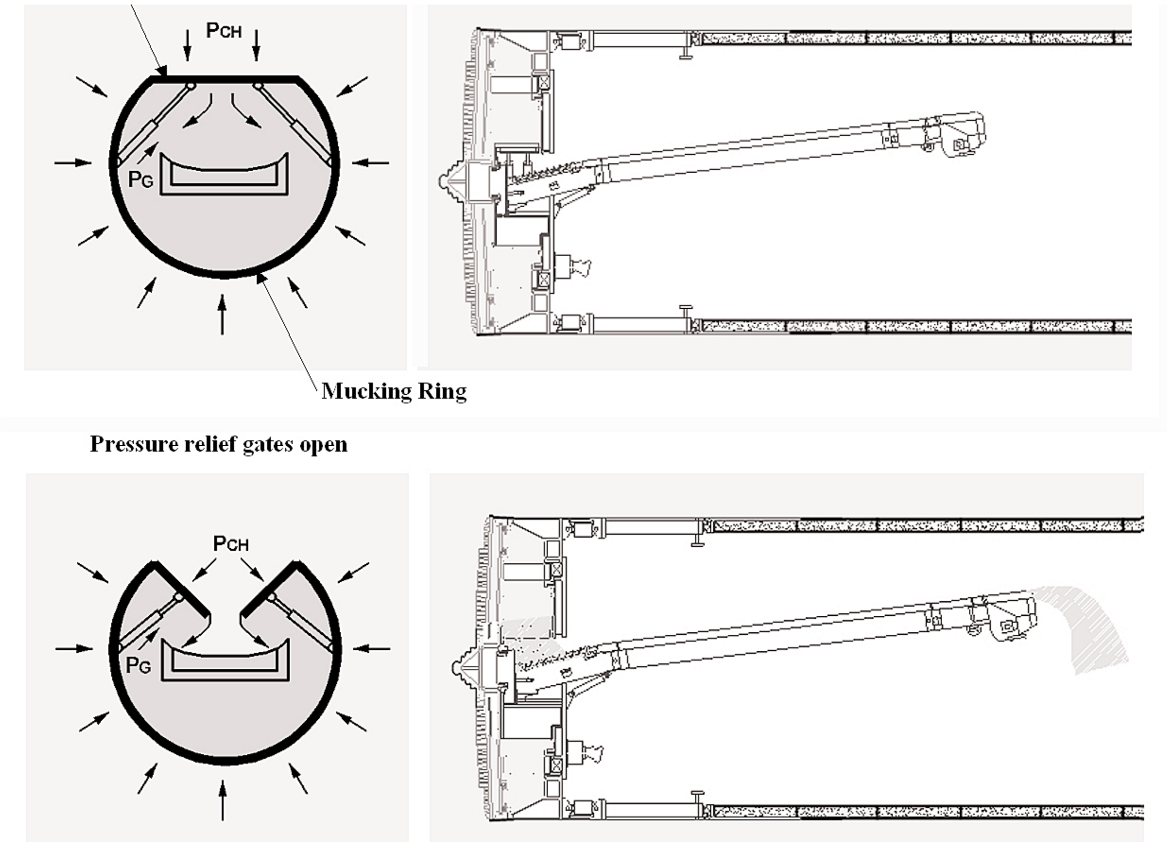


Figure 3.4.1.2: Pressure Relieving Gates and Mucking Ring in the Cutter head Chamber [2]

During construction of the Mangfall Gallery Tunnel in 1994, the contractor, in conjunction with Lovat, developed the early versions of the modern-day electronic data logging systems. The TBM´s mechanical and electrical performance was recorded continually during tunneling and transferred to an office-based computer for further processing. A graphical representation of the TBM´s operational characteristics could then be plotted. The information presented included the machine´s power consumption, cutting head torque, speed, shove ram forces and extensions, operational delays and earth pressures. This enabled the contractor on this project to optimize the performance of the machine, particularly when driving through difficult ground conditions, and provided Lovat with useful information for further refinement and development of component parts of the TBM.

Laser target guidance systems were also introduced for TBM steering control on the Mangfall Gallery project. The laser target guidance (ZED 26) monitored the vertical and horizontal deviations from the designed route during tunnel construction. The information from the unit was fed into a tunnel advance control system (TACS) computer program, which ran on a laptop computer located on the TBM. The TACS program aided the driver´s guidance of the TBM by graphically presenting the shield survey data to show present and previous positions, and by calculating projected tunnel alignment so the operator could align the TBM accordingly.

Development of cutter tools to deal with varying ground conditions was ongoing. Sticky clays balling at the tunnel face impeded progress of the TBM on the Pasillo Verde Tunnel (1994). Extra clay spades were fitted to the cutter head and water was added as a conditioner to improve the cut and transport of the material from the face to the conveyor.

In 1995, on the Jubilee line Extension Project (UK), Lovat, in conjunction with the contractor, started experimenting with foams as a conditioner to help lubricate the clay and reduce clogging in the screw conveyor. The London Clay generally allowed the TBM to be operated in unpressured mode. However, the tunnel drive encountered Thames Gravels and Thanet Sands, which required the machine to be operated in full earth pressure mode (approximately 200Kpa or 29 lb-square inch). It was on this project that Lovat used grouting through the tail-skin of the TBM simultaneously with cutting for the first time.

Another important development in the mid to late 1990s was the improvement in tailskin seals. Two rows of wire brush seals, know as tail seals, injected with grease, were becoming standard equipment to prevent the ingress of ground and grout back into the machine during tailskin grouting operations. Grease injection into the cutter head seals was introduced when the operating pressure had to be greater than 200 kPa. In addition, Lovat had now introduced and patented the Lovat segment erector, which was based on a rotational arm with a radial clamping device to facilitate the safe placement of tunnel segments firming the tunnel ring. The erector was capable of longitudinal, radial and rotational movement and was able to travel from within the tailskin in order to remove the tunnel ring covering the tail seals, if tail seals had to be replaced due to excessive wear. On the San Diego Sewer Tunnel (1997), Lovat introduced improvements to the bearing designs. A new profile for the lip seal was made of a denser and harder wearing material housed within a tighter more compact seal close to the bearing itself. The labyrinth of the main shut down would be triggered should the grease feed stop. On this project the use of foam conditioning was further developed as well. Foam was injected through nozzles on the cutter head as well as directly into the cutterhead chamber and into the casing of the screw conveyor to condition the material and facilitate optimum EPB operation.

By 1998, the Lovat EPB machine used in the construction of the Humber Waste Water Treatment Project (UK) was fitted with wight flood doors, ripper teeth, injection ports for soil conditioning, an air lock for compressed air working, advanced real-time data loggers and scraper and disc currents to deal with the variety of ground conditions encountered (laminated clays, glacial sands and gravels). In addition, Lovat had simplified the means of probing ahead of the tunnel face by introducing a probe drill mounted on the segment erector.

In 1998, on the Singapore North East Line Mass Rapid Transit (MRT) tunnels, Lovat gained extensive experience in further refining the RPB machine to deal with mixed faces of soft, mobile material and hard rock. Screw conveyors had replaced the mucking rings and were standard components for all EPB TBMs in Singapore. Lovat´s previous experience and developments culminated in a state-of-the-art EPB machine, specified and bought by the project owner, and used in the construction of the Changi Airport Line tunnel in Singapore in 1999.

All major components of the Changi Airport Line TBM, such as main bearing, main bearing seals, screw conveyor, gears, electric motors, gearboxes, hydraulic pumps and motors had a specified minimum operating life and all had some means of monitoring performance. The sealing systems of the machines were also rigorously specified, with monolithic multi-lip seals being the norm, along with a monolithic main bearing. A comprehensive data logging facility, including a TACS guidance system, recorded critical operational aspects of the TBM on a continuous basic and related the information to computer monitors in the surface offices of both the contractor and the resident engineer.

The TBM´s abrasion-resistant characteristics included a minimum thickness of 14 millimeters of chromium carbide plate on the cutterhead and cutters. Also included were wear detectors to detect the wear of the cutters during tunneling; wash-out prevention on the cutter tools (discs, scrapers and ripper teeth) and support housings; wear-resistant materials used for the screw conveyor auger and casing, and the ability to replace al tools from within the cutter-head.

Other principal features of the machine included positive articulation between the shield and the tailskin to improve steering control, continuous tailskin grouting, provision for ground treatment and probing, allowance within the tailskin for dismantling of a completed ring and comprehensive soil conditioning with foam polymer and water.

Unusual features introduced for this TBM were an on-board diesel-powered emergency generator fitted with catalytic converter, which would keep the lights, pumps and ventilation running in the event of a main power failure, and an extra gantry at the back of the TBM trailer housing an air conditioning system.

Four Lovat EPB TBMs were, at the time of this writing, constructing the East-Central Interceptor Sewer (ECIS) project in Los Angeles, California, USA, with more EPB TBMs planned for the North-East Interceptor Sewer (NEIS). The ECIS project involves more than 17 kilometers of 3.3- meter-diameter finished tunnel, constructed through unconsolidated alluvial clays, sands, and gravels, above and below the groundwater table. Challenges include construction through residential areas, limited areas for construction staging and a tight construction schedule; the tunnels are supported with a one-pass bolted and gasketed segmental lining, and then lined with corrosion-protected reinforced concrete pipes.

3.4.2. Japanese Developments over the Last Decade

Largely as a result of ideal ground conditions for EPB tunneling in the coastal regions of Japan, Japanese manufactures began developing EPB TBMs as early as the 1970s. From 1970 through to the early 1990s, Japanese TBM manufacturers were often setting the pace in technology advancements on EPB machines in which their European and Canadian counter parts would follow. There are now five major manufacturers of Japanese TBMs (more than in any other country), namely Hitachi-Zosen, Mitsubishi, Kawasaki, Ishkawajima-Harima Heavy Industries (IHI), and Nippon Kohan K.K (NKK). Since the early 1990s, Japanese TBMs have been prolific in the construction of bored tunnels throughout Asia.

From 1998 to 2000, each of the above manufacturers supplied RPB TBMs to various tunneling projects in Singapore. Hitachi-Zosen and IHI supplied TBMs to the North East Line MRT project, while Mitsubishi Kawasaki and NKK provided EPB TBMs to contractors excavated some of the longest tunnels on the Deep tunnel Sewer System project.

Over the last decade, Japanese manufacturers and contractors have focused technology development on automation. These new developments have been used mainly in Japan, probably as a result of high labor costs there. Robotic segment assembly eliminates much of the manual labor required in the ring build area.

Japanese manufacturers have been designing TBM system with the injection of high precision, high speed tunnel lining assembly with greater safety. Generally, the system would be washer supplier and a bolt-fastening machine. The system automates the entire work sequence, from segment supply to completed tunnel lining.

Hitachi Zosen has been manufacturing TBMs since the early 1980s and have to date built over 100 EPB TBMs ranging in diameter form 3 to 8 meters. On visits by the author to its manufacturing plant outside Tokyo, Hitachi-Zosen explained some of the features of Japanese technology recently introduced into its TBMs, including the automatic directional control system and the underground echo sounder. The directional control system is composed of an automated surveying and attitude control system. The system includes a function for evaluating the characteristics of the ground and the reaction of the TBM so direction can be controlled without being affected by changes in soil conditions.

The underground forward echo sounder works on a similar principle as the sonar system used in submarines. While the TBM s stationary, sound waves are emitted. Reflected sound waves identify obstacles ahead of the TBM and the distinct boundaries between varying geological strata. Multiple sensors measure the time taken for the sound to return, and this makes it possible to determine the distance to the obstacle or strata boundary.

The author has not personally experienced either of these two forms of technology in practice but would point out that neither should be considered as a substitute for a through site investigation and sound knowledge of the geological conditions through which the TBM is tunneling.

Traditionally, Japanese-manufactured TBMs favored two-speed electric drive motors, but some, for example NKK (Singapore DTSS), have been supplied with inverter (variable speed) motors. The advantage of inverter motors is that the standard eight-pole two speed electric motors. Thus the efficiency of available torque to cut the ground, particularly in variable ground conditions, is increased..

Japanese manufactures generally use two basic types of cutterhead configurations n their EPB-TBMs- the spoke type and the slit type. Hitachi-Zosen explained that the prevalent ground conditions generally dictate what type of cutterhead it will use. The spoke type is preferred fro clays, silts, sands and gravels where the maximum size of boulder or cobble expected to be encountered can be discharged through the screw conveyor. The spoke type cutterhead has a flat profile and is generally fitted with scrapers and ripper type cutting teeth. Occasionally, disc cutters may be incorporated into a spoke type cutterhead. For the spoke type cutterhead, the opening ratio generally varies between 40 and 70 percent, depending upon the ground type and the required operating pressures.

The silt type cutterhead design is also applicable for clays, sands, silts and gravels but can be further applied to gravels plus boulders and cobbles and rock. The boulders or cobbles encountered must be broken into small enough pieces at the cutter face and, therefore, in addition to the scraper teeth and rippers, the slit type cutterhead would generally fit with disc cutters.

For the slit type cutterhead, the opening ratio is less than the spoke type and generally varies between 20 and 50 percent. The slit type cutterhead profile varies from flat through semi domed to full dome depending upon the ground type, with the flatter profiles being used in ground with a silty sand content.

The two basic types of screw conveyor used are the shaft type and ribbon type. Hitachi-Zosen introduced the ribbon screw type conveyor, which has proven effective in ground condition coarse gravels and cobbles due to the increased pith (spacing between successive flights) available. The shaft type is generally used for smaller diameter screw conveyors than the ribbon screw, although the ribbon screw requires greater torque capacity.

Other Japanese manufactures, Mitsubishi and IHI, offer features such as simultaneous cutting and segment assembly. This can be achieved by means of a double cylinder shield whereby the forward thrust rams pushing the cutterhead are contained within the front shield, and the tailskin in long enough to enable the construction of two tunnel tings. The rear facing thrust cylinders, extending from the rear of the front shield, transfer the thrust onto the previously erected tunnel rings. Each of the thrust rams can be independently released to enable the positioning of a new segment within the ring being built.

Preventative maintenance technology is also widely used on Japanese TBMs, including cutter bit wear detection; cutter seal fault detection, cutter bearing lubrication control and cutter bearing fault detection.

3.4.3. European Development

Herrenknecht AG led the way in the Europe in the design of the EPB TBMs and has a proven record of a accomplishment worldwide. Most recently, Herrencknecht ventured into the market in China. Line 5 of the Beijing Underground was excavated using a Herreknecht 6.2- meter-diameter EPB.TBM, through clayey soil with sand, gravel and silt, 10 meters below the water table. Four 6.5- meter-diameter Herrenknecht EPB TBMs are also constructing the North-South Line 2 in Beijing. Two of these TBMs were previously used in the construction of the North-East line MRT in Singapore.

Each of the four machines will bore approximately 2 kilometers in weathered mudstone-clay stone with unconfined compressive strengths up to 60 MPa with some clay areas. The machines are EPB TBMs fitted with disk cutters.

In Buenos Aires, South America, two 4.3-meter-diameter Herrenknecht EPB TBMs excavated the Saavedra-Moron tunnel comprising four drives of 3.56 kilometers. 4.2 kilometers, 3.7 kilometers, and 3-8 kilometers, at depths of up to 35 meters. Ground conditions were variable, ranging from quicksand under 2.5 bar pressure, through sandy clays to hard consolidated silts with calcareous nodules. Soil conditioning using a combination of foams and bentonite was used on this project.

The 2.2 kilometer long Quattro Venti tunnel in Italy was excavated using a 7.9 meter diameter Herrenknecht EPB through tuff, silt clay alluvial, and gravel with only 2 to 6 meter cover in places, and up t 3.5 bar water pressure in others.

In 1985, Herrenknecht, in association with WyassandFrytagg, patented a combined shield, known as the mixshield. The philosophy of the mixshield was based on the principle of the hydroshield (patented by WyassandFrytagg). Mixshields are normally used in slurry mode; however when conditions merit it, conversion to EPB mode is possible.

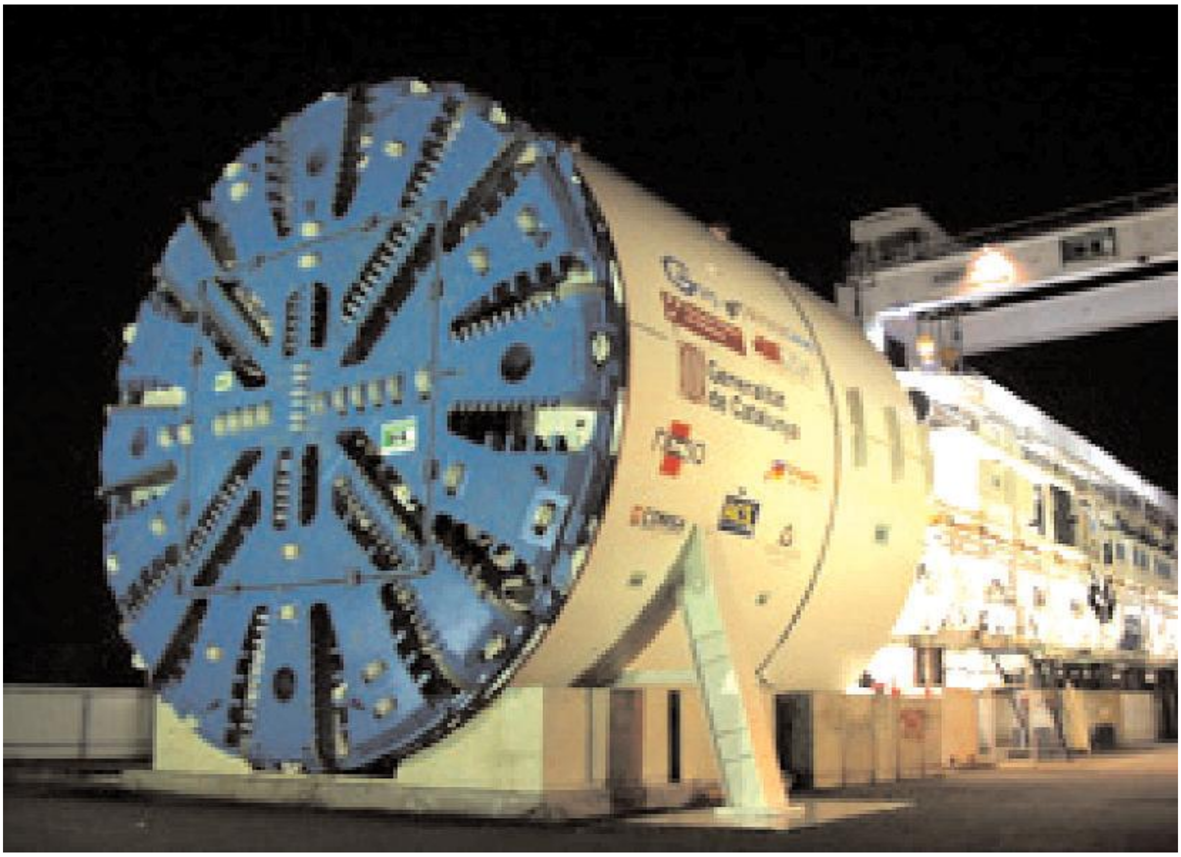


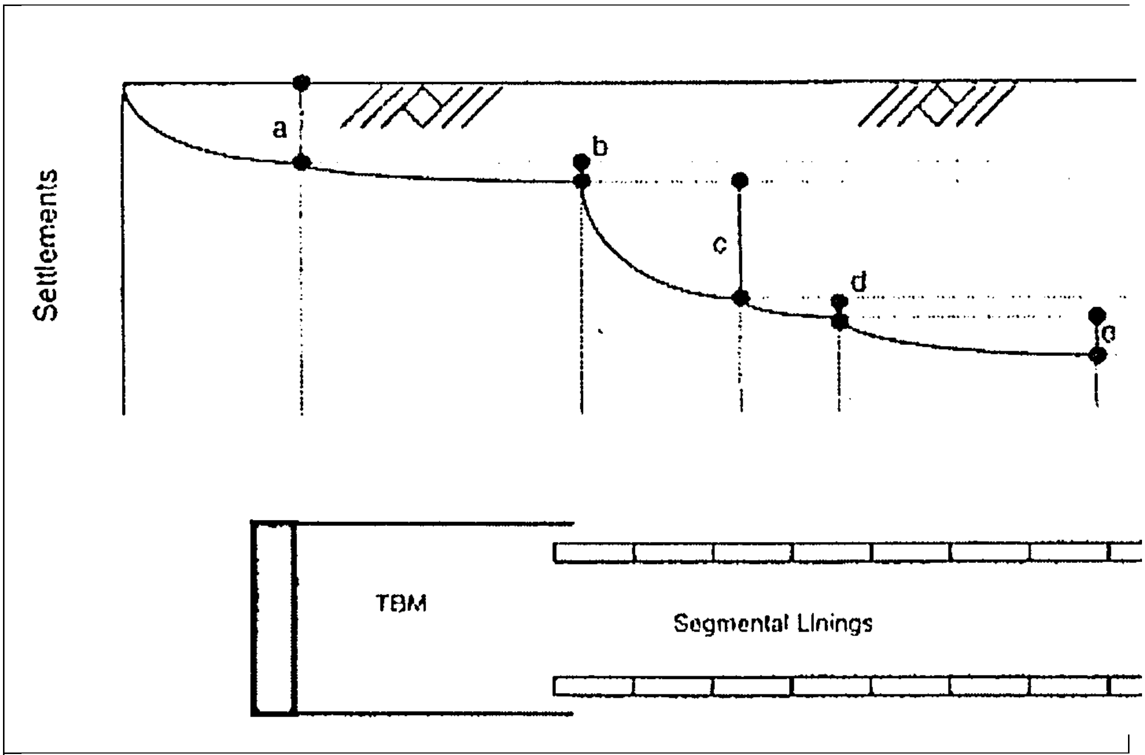
Figure 3.4.3.1: 12.06 Meter Diameter Herrenknechr EPB TBM, used for Barcelona Underground [3]

In 1994, the first conversion from slurry mode to EPB mode was carried out on the construction of the Duisberg Metro in Germany. In addition, this project was the first time in Germany that foam was used as a soil conditioner.

In 2002, Herrenknecht celebrated a quarter of a century of manufacturing tunneling equipment and TBMs. It was fitting that Herrenknecht celebrated this achievement with construction of the worlds largest EPB TBM (as of 2002), with a diameter of 12.06 meters, for the Metro No.9 Line in Barcelona, Spain (Figure 3.4.3.1). The TBM weighed 2,250 tons, had a maximum torque of 45,000 kilonewtonmeter (kNm), the highest torque that ever been installed in a Herrenknecht was also planning 14 meter diameter EPB TBMs for excavation of tunnels in Beijing, China.[3]

3.5. Control of Ground Movements Using EPB– TBM’s

Tunnel boring machines of any type will induce some degree of settlement as they advance through the ground. Figure 3.5.1 shows the general character of this settlement for an EBP – TBM. [5]



- Point a - Settlements ahead of the face of the TBM are the result of over-mi during the advance of the TBM.
- Point b - Settlements along the shield of the TBM are a function of the cut a face being a larger diameter than the tail shield.
- Point c - Settlements at the transition between the tail shield and the segme lining are the result of the variation in diameters between the two linings.
- Point d - Additional settlements after the passage of the tail shield of the T can be viewed as initial lining deflections.
- Point e - Long term settlement.

Figure 3.5.1: Typical Settlement Pattern EPB TBM [5]

3.5.1. Ground Movements Ahead of the Face

Ground movements ahead of the face are a function of the balance between the earth pressures in situ and the pressures maintained in the excavation chamber of the TBM by the conditioned soil. In order to achieve complete control over the system, the pressure maintained in the excavation chamber of the TBM should be as large as the combined earth and water pressure acting on the face. However in the operation of an EBP – TBM, the pressure maintained in the excavation chamber should be large enough to prevent the active failure of face, and large soil movements expected as the excavation chamber pressure nears the active failure criteria. Passive failure of the soil face will result in the heaving of the ground ahead of EBP-TBM. Face pressure distribution is given Figure 3.5.1.1

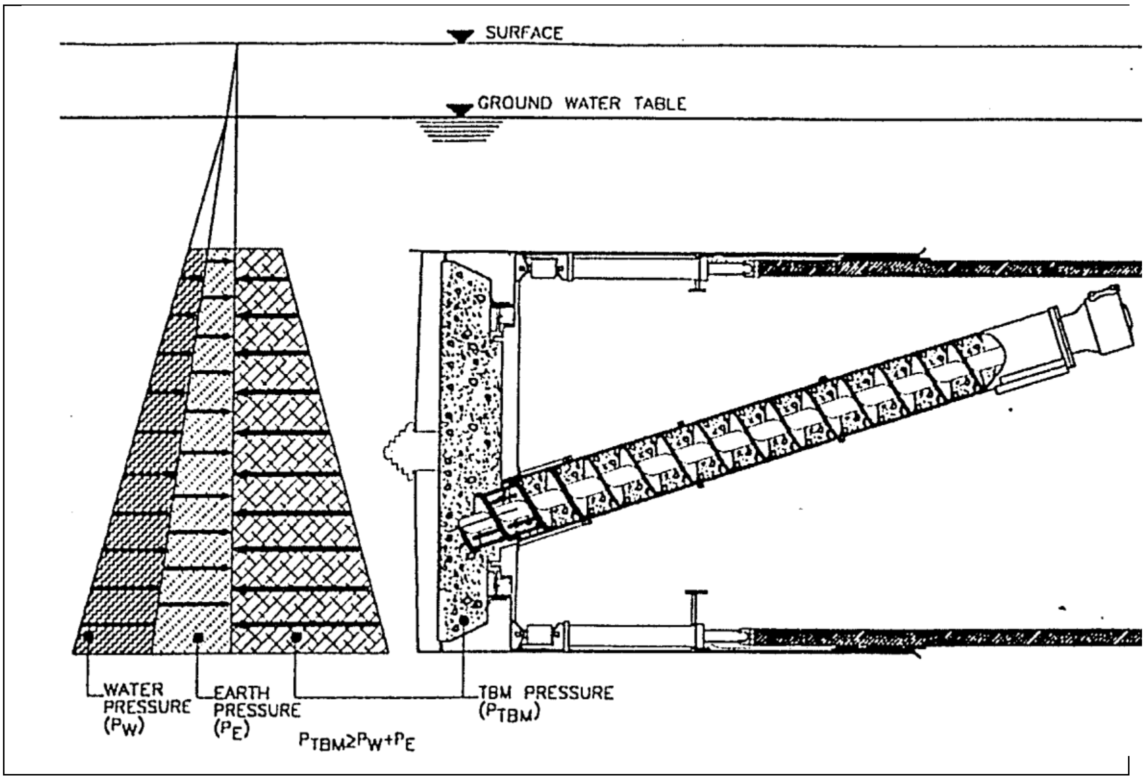


Figure3.5.1.1: Pressure distribution along TBM [5]

3.5.2. Ground Movements along the TBM shield

Ground movements that occur at points overlaying the shielded portion of the TBM are beyond the operator's control. When TBMs are manufactured, the diameter of cutting head is larger than the diameter of the shield. This overcut is necessary to reduce the drag on the TBM and allow the advance of the TBM.

3.5.3. Ground Movements when Transitioning between the Tail shield and Erected Segmental Lining

In Tunnel Boring Machines there is a transition between the shield of the TBM and the segmental lining. As the segmental lining is installed inside the shield it is impossible for it to be of the same diameter as the shield. As a result a gap always formed when transitioning from the shield to the installed segmental lining. To mitigate this source of ground movement the void created by the transition is injected with grout.

3.5.4. Ground Movements Due to Deflection of the Tunnel Lining

Segmental linings serve two important functions in tunnel construction. They provide the actual structure of the tunnel; of equal consideration, is that they provide a thrust base for the advance of tunnel boring machine. In addition the segmental lining is also subject to the loads imparted by the ground. Provided grout injection has been adequately performed the ground movements due to tunnel deflection will be very small [5].

3.5.5. Ground Movements Due to Long Term Settlements

Long – term settlements are typically associated with long term consolidation of the soil after construction of tunnel. This consolidation occurs when a tunnel constructed below the water table acts as a drain. This reduction in water table increases the effective stress in the soil causing the ground to consolidate and surface to settle.

4. FACE STABILITY

4.1. Basic Concepts Related to Tunnelling-Induced Settlements

Ground movements are an inevitable consequence of excavating and constructing a tunnel. Tunnel excavation causes relaxation of in-situ stress, which is only partially restricted by the insertion of the tunnel support. In fact, it is not possible to create a void instantaneously and provide an infinitely stiff lining to fill it exactly. Hence, a certain amount of the deformation of the ground will take place at the tunnel depth; this will trigger a chain of movements, resulting in settlements at the ground surface, which become more significant with the decrease in tunnel depth.

Settlements are mainly due to three components:

1. The short-term (or immediate) settlements caused by the tunnel excavation, which are a function of: the stability of the tunnel face, the rate of advance, the time necessary to install the tunnel lining and, in case of mechanized tunnelling, the time necessary to fill the tail-void. The immediate settlement along the tunnel axis starts at a certain distance ahead of the tunnel face and comes to a halt when the grout injection of the tail void has hardened enough to counteract any further radial displacement.

2. The settlements due to the deformation of the tunnel lining. This component can be relevant for large-diameter tunnels at shallow depth. However, it plays a negligible role in mechanized tunnelling in urban environment, where the loads are well- predicted and excessive deformations can be easily avoided by properly designing the segmental lining.

3. The long-term settlements, due to the primary consolidation (that normally occurs in cohesive, or compressible, soils during dissipation of excess pore pressure) and (2) secondary consolidation (a form of soil creep which is largely controlled by the rate at which the skeleton of compressible soils can yield and compress).

During the process of excavation, the unsupported or partially supported ground around the tunnel moves inwards as stress relief takes place. Thus, it will always be necessary to remove a larger volume of ground than the theoretical volume of the finished void. This extra volume excavated is known as “volume loss” (or “ground loss”) and it is expressed in terms of unit distance advance of the excavation that causes the relaxation (i.e. m3 per meter advance) The magnitude of the movements causing the volume loss is a function of the soil type, rate of tunnel advance, tunnel diameter, excavation method, and form and stiffness of temporary and primary support.

In the specific case of mechanized tunnelling, the individual factors contributing to volume loss are

1. The decompression at the tunnel face. The rotating cutters of the shield remove material from the tunnel face; during this continuous process, the ground protrudes out of the face from a zone of influence ahead and around the tunnel face. This gives rise to the “face loss” (Fig. 4.1.1a).

2. The excavation of a slightly oversized tunnel hole at the front of the shield in order to ease the advance of the shield. At least two factors result in a slight over excavation at the face of the shield. First, the cutterheads are made slightly larger in diameter to reduce the chance of the shield being stuck. This is often achieved by welding a steel strip or by simply welding “beads” on the outside of the shield cylinder. Second, the over-excavation at the face of the shield results from steering the shield to go around curves or just for steering in alignment. After the beads have passed, the ground has the opportunity to move inwards radically (Fig.4.1.1b)

Depending on the rate of deformation of the soil relative to the rate of advance, the excavated perimeter may close completely over the shield (Fig.4.1.1c).

3. The lining, which is of slightly smaller diameter than the shield, is erected inside the shield and the annular void between the lining and the ground is immediately filled, normally with injected grout. Thus, there is a further opportunity for the ground to converge radically onto the lining, until the grout has completely filled the void and has hardened sufficiently to resist the earth pressure, or if the void is not properly injected (Fig. 4.1.1d).

The sum of the two radial displacements (Fig. 4.1.1.1b, c) is termed “radial loss”. The sum of the “face loss” and the “radial loss” gives the overall volume loss, VL, resulting from the excavation of the tunnel.

Both the face loss and the radial loss can be properly controlled by adequate TBM driving procedures. In fact, in mechanized tunnelling the face loss is very limited if the tunnel face is properly pressurized and the radial loss is easily controlled by injection of an adequate volume of grout at the right pressure, with a proper grouting mix design, and through regularly maintained injection lines to avoid plugging.

However, properly pressurizing the tunnel face in order to prevent the face loss also requires a deep understanding of the potential failure mechanisms of the ground vs. TBM tunnelling, in order to define the most appropriate range of operational pressure distribution to be applied at the tunnel face according to the encountered geology, the groundwater height, and the depth of the tunnel. [1,6]

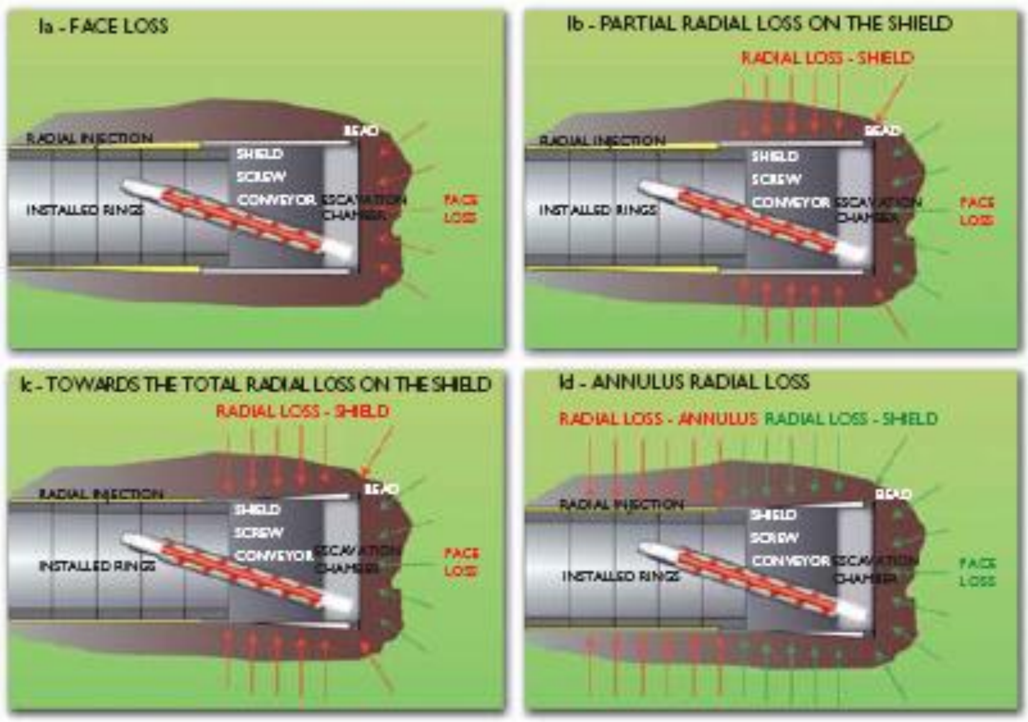


Figure 4.1.1: Factors contributing to the volume loss [1]

4.2. Stability Analysis of the Tunnel Face

One of the main objectives of the tunnel boring process is to adequately support the soil and to minimize deformations during and after construction. This is especially the case in urban environments, where the influence of a tunnel collapse or extensive deformations can be catastrophic, and even limited soil deformations may damage buildings To prevent this it is necessary to support soft and non-cohesive soils from the time they are excavated to the moment the final support is installed. Where groundwater is present it is also necessary to prevent a flow towards the tunnel face, as this flow may have an eroding effect on the tunnel face.

When a tunnel boring machine (TBM) is used to excavate the soil, the radial support and water tightness is first ensured by the shield and after that by the tunnel lining. At the face such mechanical support is impractical or impossible to combine with an efficient excavation process, and indirect ways of face support are used. Compressed air or a pressurized slurry may be used in case of a slurry TBM and a mixture of the excavated soil mass and varying additives is used in an earth pressure balance.

Whatever means of support is used, the pressure in the working chamber of the TBM should be kept at such a level that stable working conditions are ensured. It should not be as low as to allow uncontrolled collapse of the soil into the working chamber, nor as high as to lead to large deformations of the soil or to a blow-out and subsequent loss of the support medium. The actual range of allowable support pressure for a given tunnel project will depend among other things on the actual soil and groundwater conditions, the excavation method and the size and overburden of the tunnel. To find the minimal and maximal allowable support pressures, a number of models have been proposed in literature over the years to describe different possible failure mechanisms of the tunnel face and to calculate the properties of the support medium necessary to prevent collapse.

These models can be categorized into three main classes based on the type of failure mechanism they describe. Models which describe the behavior of a group of grains or a single grain at the tunnel face are called internal or micro-stability models. Models which describe a failure mode of a large part of the face or the entire face are called external or global stability models.

Some authors further subdivide the external models into local stability models, which do not directly influence the soil surface, and global stability models, which do. Other authors define local instabilities as those external instabilities which influence only part of the tunnel face, regardless of their influence on the soil surface, and that definition will be used in this thesis. Both micro- and local instabilities may well initiate progressive failure and lead to a global collapse of the tunnel face. A third class of models are those which in themselves do not describe the likelihood of occurrence of an instability of the soil around the TBM, but of the loss of the support medium, which in turn of course may lead to a reduction of the support pressure and subsequent collapse of the tunnel face.

A number of effects identified from field cases and laboratory tests is not included in current stability models. Especially the infiltration of the support medium into the soil in front of the TBM and the presence of excess pore pressures may have a profound effect on the stability of the tunnel face [7]

Micro-Stability:

Micro-stability, the stability of the single grain or a small group of grains at the tunnel face, is mainly a problem in soils with no or low cohesion and a slurry or air supported tunnel face. In such conditions grains may fall from the soil matrix under gravitational forces. When followed by subsequent grains the face may erode and this can introduce a local or global collapse. To prevent this type of collapse, a minimal pressure difference over the grains is necessary. This has no direct influence on the support pressure, but on the pressure gradient into the soil. As will be shown, this leads to a minimal shear strength requirement for the slurry.

According to Müller-Kirchenbauer a small cohesionless soil element at the tunnel face v is subjected to gravity and acted upon by forces from the support medium, $s_F = i_0\gamma_F$, and the surrounding soil (Figure 4.2.1.1). For a given slope α, failure occurs when $i_0\gamma_F/\gamma\,'\sin\phi = \sin(\alpha - \phi)$,

with i_0 the stagnation gradient, γ_F the unit weight of the suspension, $\gamma\,'$ the effective unit weight

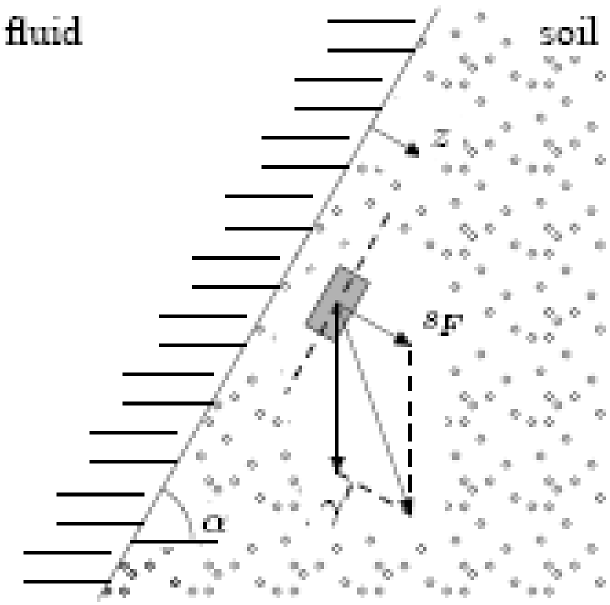


Figure 4.2.1.1: Forces included in the micro-stability analysis [7].

of the soil. The stagnation gradient is given by

$i_0 = 2\tau_F / rp\ \gamma_F$,

where τF the yield strength of the suspension and rp the equivalent pore radius of the soil capillaries. For the most common case $\alpha = 90^\circ$ this results in a minimum requirement for the yield strength of the support medium

$\tau_F \geq rp\ \gamma\ '/2 \tan \phi$

A number of researchers have made estimates of the equivalent pore radius rp, resulting inn slightly different estimates of τ_F . Kilchert for example uses

$rp = 2(1 - n)d10$ (2.4)

which relationship leads to

$\tau_F \geq d10\ (1 - n)\ \gamma\ '\tan \phi$

Except for its direct influence on the stability of a single grain, the yield strength has a major influence on the infiltration length of the support medium into the soil.

In cohesive soils the stability of individual particles will generally is secondary to the stability of the entire face. A problem could occur when a pressure gradient towards the working chamber is present. Such a situation could occur in an EPB shield with a support pressure below the hydrostatic pressure unfortunately, the stability analysis of an infinite slope, the approach followed by Müller-Kirchenbauer to obtain cannot be extended straightforward for a cohesive-frictional material.

Another approach would be to investigate the stability of an unsupported vertical cut ($\alpha = 90^\circ$) in a purely cohesive material subject to a drag force. A simple upper bound analysis assuming a straight failure plane and no drag force leads to a maximum height before failure

$h_c \leq 4 c_u \gamma$.

This is only slightly higher than the best known upper bound solution, which has a factor 3.83instead of 4. It is assumed that a seepage flow is present and results in a horizontal drag force γ_w i. In that case the maximum height of the cut can be estimated from

$h_c \leq 4 c_u \gamma\ 1\ f + p\ f\ 2 + 1$

with $f = i\gamma_w/\gamma$ or, inversely, the critical gradient can be determined.

In cases where the groundwater flow towards the face becomes a problem, a closer analysis is warranted. A possible approach would be to choose an arbitrary failure height for the slope, i.e. a-reference stress level, and then proceed it a slope stability analysis, e.g.Müller-Kirchenbauer's method, Bishop's method or a fully numerical (FE) solution method. Alternatively one could use a semi-empirical approach as used in the analysis of piping phenomena, but with less stringent safety factors, as the time span over which the slope has to remain stable is significantly shorter

4.3. Description of the Principal Elements of the 12 Analytical Methods for Defining Face-Support Pressure

4.3.1. Method of Horn (1961)

It provides the basis scheme of the three-dimensional failure model, composed of a wedge (with smooth surface), in the lower part, and a silo, in the upper part the model does not provide indications for practical applications. However, it has been used by several authors as a basis for further development [1]

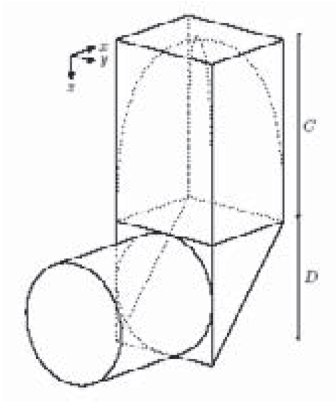


Figure 4.3.1.1.: The tunnel-free stability model of the method of Horn[1].

4.3.2. Method of Murayama (1966)

The soil weight (*qB*) acting on the pressure wedge is calculated in accordance with the Terzaghi (1943) theory, the failure surface is a logarithmic spiral (Fig. ap.4.3.2.1). The face stability requires the equilibrium between the moment of the acting weight forces (*qB*+*W*) and the resistant forces [force applied on the tunnel face (*P*) and shear strength along the failure surface].[1,8]

The method contemplates the iterative search for the solid-load width (*B*) that determines the more unfavorable loading condition and, therefore, the maximum stabilization pressure, *P*.

The basic equation is:

$$P = [W \times l_W + q_B \times B_1 \times (l_B + B_1/2) - c\,(r_{d2} - r_{a2}/(2\tan \Pi)]/(2R \times l_p)$$

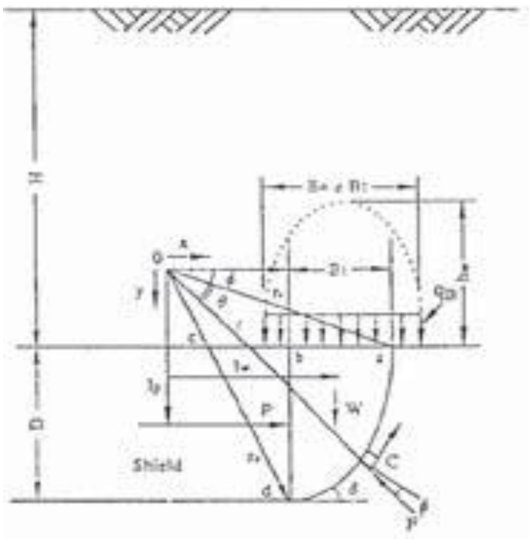


Figure4.3.2.1: The tunnel-face stability model of the method of Murayama [8].

4.3.3. Method of Broms and Bennemark (1967)

It provides a relation for the stability analysis of an unsupported opening in a cohesive un-drained material [9]. The stability ratio N is defined as:

$N = (qs - \sigma_T)/cu + (C + R) \cdot \gamma/cu$

where γ = soil density and; cu = un-drained cohesion.

Empirically, the instability conditions are associated with a value of $N \geq 6$. Therefore, the minimum stabilization pressure σ_T is:

$\sigma T = \gamma \cdot (C + R) + qs - N \cdot cu$ with $N \approx 6$

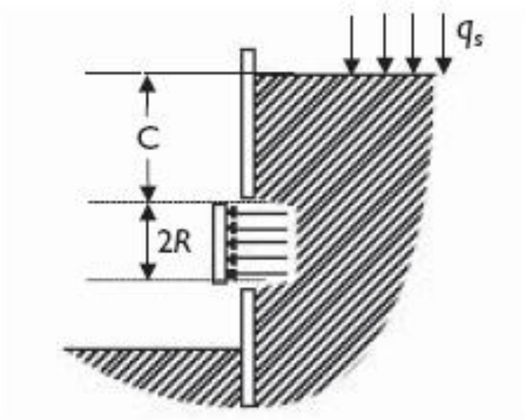


Figure4.3.3.1: The tunnel-face stability model of the method of Broms and Bennemark [9]

4.3.4. Method of Atkinson and Potts (1977)

The minimum support pressure for an excavation face in incoherent drained soil is determined considering two limit conditions: 1) $\gamma = 0$ *e* $qs > 0$; 2) $\gamma > 0$ and $qs = 0$, where γ = soil density and qs = surcharge. For the second case, two lower limit solutions are furnished. The solution, which is independent of the overburden, provides, in general, the result associated with the greater safety:

$S_{min} = [2kp/ (kp^2 - 1)] \times \gamma \times R$

where $kp = (1 + \sin \varphi)/(1 - \sin \varphi)$ and R = radius, and φ is the frictional angle of the soil.[10]

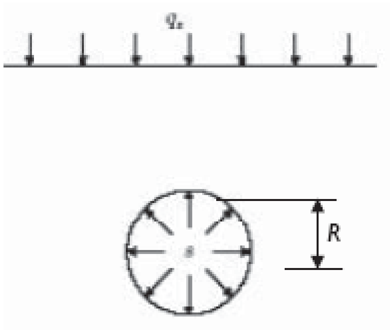


Figure 4.3.4.1: The tunnel-face stability model of the method of Atkinson and Potts [10].

4.3.5. Method of Davis et el. (1980)

This method allows the stability analysis of a tunnel with radius *R*, in a cohesive soil, where a rigid support is installed at a distance P from the face. Lower and upper limit solutions under general conditions are provided through diagrams, and two particulars cases are analyzed: $P = \infty$ (Fig. ap.4.5.1) and $P = 0$. In the last case, of particular interest to excavation using a shielded TBM, two lower limit solutions are provided as functions of the reference stress state model: cylindrical or spherical. The stability ratio, *N*, in the two cases is calculated using, respectively [11]:

$N = 2 + 2\ln(C/R + 1)$ [cylindrical]
$N = 4\ \ln(C/R + 1)$ [spherical]

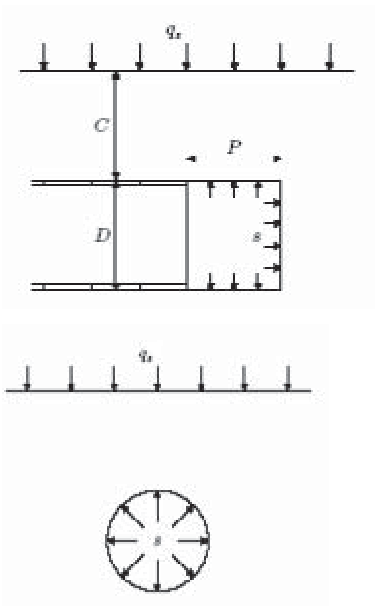


Figure 4.3.5.1.: The loading schemes of the method of Davis et al [11].

4.3.6. Method of Krause (1987)

It provides the minimum support pressure for the different failure mechanisms reported in the Figures ap.4.3.6 1. (a), (b), and (c).

The model with the failure surface consisting of a quarter of circle (Fig. 4.3.6.1 b) gives the maximum value of the stability pressure [1, 12].

$s_{min}[\max] = (1/\tan \varphi)\ (D\ \ \gamma'/3 - \pi\ c/2)$

In many cases, with the semi-spherical model (Fig.4.3.6.1c), the solution obtained is closer to the reality:

$s_{min} = (1/\tan \varphi)\ (D\ \ '/9 - \pi\ c/2)$

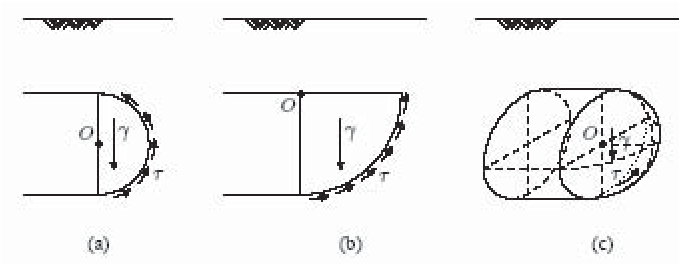


Figure 4.3.6.1: The various modes of instability as assumed for the tunnel face in the method of Krause [1].

4.3.7. Method of Mohkam (1984, 1985, 1989)

This method uses a 3D mathematical approach founded on the limit equilibrium theory, implementing a variational analysis to define the 3-D failure surface and the relative state of stress acting at every point of the model. Taking into account the support-free length before the installation of a stiff support, two failure mechanisms are assumed: one involves the face excavation (Fig. 4.3.7.1.a , b) and the other involves the tunnel wall (Fig. 4.37.1.), along the failure surface, respectively, logarithmic spiral and cylindrical.[13]

The load acting on the wedge is based on Terzaghi's arch effect.

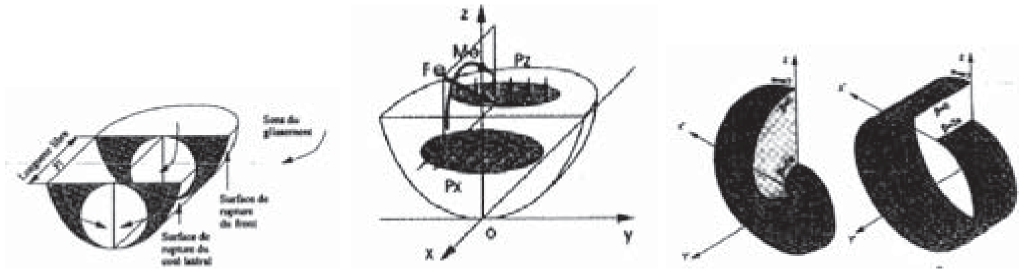


Figure 4.3.7.1: The failure mechanism assumed for the method of Mohkam et al [12].

4.3.8. Method of Leca and Dormıeux (1990)

This method is based on the upper and lower limit theorems with a 3D-modelling. The upper(+) and lower (–) limit solutions are obtained by means of a cinematic and a static method, respectively, giving thus an optimistic and a pessimistic estimation of the face-support pressure. In the case of dry condition, the face support pressure σT is (Ribacchi, 1994):

$$\sigma T = -c' \cdot ctg\varphi' + Q\gamma \cdot \gamma \cdot D/2 + Qs \cdot (\sigma s + c' \cdot ctg\varphi')$$

where Qγ, Qs = non dimensional factors (from nomograms), function of H/a and φ';a = radius of the tunnel; H = thickness of the ground above the tunnel axis.

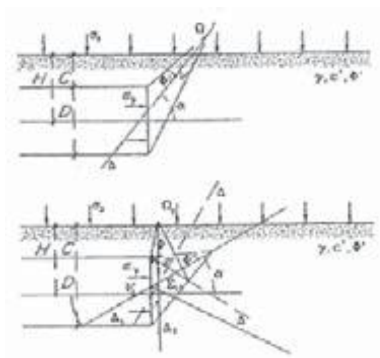
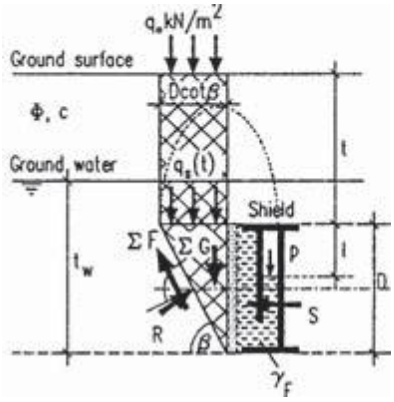


Figure4.3.8.1: The tunnel-face stability model of the method of Leca and Dormieux [14]

4.3.9. Method of Jancsecz and Steiner (1994)

According to the model of Horn (1961), Method 1, the three-dimensional failure scheme shown in Figure. 4.3.9.1. consists of a soil wedge (lower part) and a soil silo (upper part). The vertical pressure resulting from the silo and acting on the soil wedge is calculated according to Terzaghi's solution.



[15]

Figure 4.3.9.1: Method of Jancsecz and Steiner scheme.

A three-dimensional earth pressure coefficient ka3 is defined as:

$k_{a3} = (\sin\beta \cdot \cos \cdot - \cos^2\beta \cdot \tan\varphi - K \cdot \alpha \cdot \cos\beta \cdot \tan\varphi/1.5)/(\sin\beta \cdot \cos\beta + \sin^2\beta \cdot \tan\varphi)$

where: $K \approx [1 - \sin\varphi + \tan^2(45 + \varphi/2)]/2$; $\alpha = (1 + 3 \cdot t/D)/(1 + 2 \cdot t/D)$.

4.3.10. Method of Anagnostou and Kovari (1994 and 1996)

Solution for EPB shield

This method, later referred to as A-K method, is based on the silo theory (Janssen, 1895) and to the three-dimensional model of sliding mechanism proposed by Horn (1961). The analysis is performed in drained condition, and a difference between the stabilizing water pressure and effective pressure in the plenum of an EPB is presented. If there is a difference between the water pressure in the plenum and that in the ground, destabilizing seepage forces occur and a higher effective pressure is required at the face. However, accepting this flow, the total stabilizing pressure is lower than the pressure required in the case of an imposed hydrogeological balance. The effective stabilizing pressure (σ’) is:

$\sigma' = F0 \cdot \gamma' \cdot D - F1 \cdot c' + F2 \cdot \gamma' \cdot \Delta h - F3 \cdot c' \cdot \Delta h/D$

where $F0$,$F1$,$F2$,$F3$ are non-dimensional factors derived from nomograms, which are function of H/D and φ’.

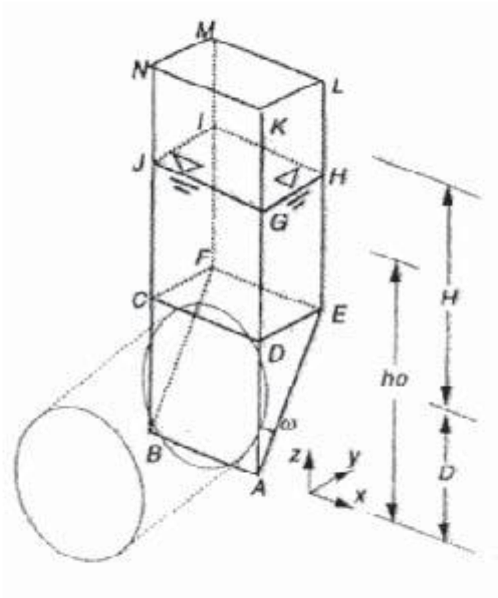


Figure 4.3.10.1.: The tunnel-face stability model of the method of Anognostou and Kovari[16]

The original analysis considers two values of k0, 0.8 and 0.4, for the prism and for the wedge (tunnel level), respectively.

If the material in the plenum is in a fluid state, i.e. σ' = 0, then solving the above equation for Δh, the necessary water pressure for equilibrium is obtained.

Solution for Slurry Shield

In case of a Slurry Shield, the work pressure (*pb*) must be greater than the external water pressure (*pW*) in order to avoid the water flow in the plenum. The stabilization pressure or, the delta pressure (Δ*p*), depends on the degree of penetration of the bentonitic slurry in the soil. The minimum value of Δ*p* is associated with the formation

at the face of an impermeable membrane, the cake, (case "a'' in Fig..4.3.10.2). According to such a hypothesis, some diagrams are supplied for the estimate of Δp as a function of the parameters of shear resistance, the water head, and the tunnel depth.

Instead, in the case of penetration (e) of the bentonitic slurry at the face, the stabilizing effect of the applied face-support pressure is lower than that in the "membrane" model by a factor equal to:

$[1 - e/(2D\tan\omega)]$ if $e < D\tan\omega$, or

$[D\tan\omega/2e]$ if $e > D\tan\omega$

For, respectively, the partial and the complete saturation of the wedge with the bentonite slurry.

Depending on the characteristics of the slurry and soil, it is possible to calculate the "stagnation gradient" $fso = \Delta p/e_{max}$ where e_{max} is the maximum penetration for the assigned ΔP.

The German norm DIN 4126 suggests, moreover, the following empirical formulation: $fso = 2\tau f / d10$ where τf is the shear strength of the bentonitic slurry and $d10$ is the characteristic size of the soil, determined from its particle size distribution.

The infiltration risk is small in soils with fine grain size, but is elevated in soil with coarse grain distribution. According to A-K, however, such risk is present essentially in the periods of shield stand-still, during which the stagnation gradient falls down and, with it, the safety factor F.

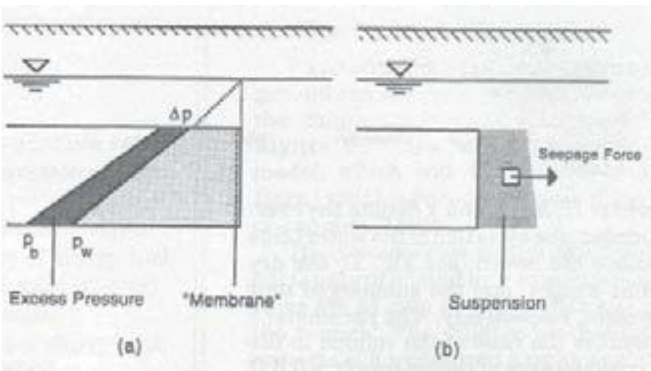


Figure 4.3.10.2: Membrane and filtration model (solution for slurry shield) [15].

Some relations for the calculation of the critical time of stability of the face, as a function of the characteristics of slurry and soil, are supplied, including the advancement rate of excavation (v). It is possible to calculate the critical speed ($v$cr) of advancement, under which the filtration determines the critical gradient of stagnation ($fs = fscr$ and, therefore, $F = 1$). In general, the much more elevated is the relationship v/k (where k is the permeability of porous medium) the lower is the depth of attainable filtration.[16,17,18]

4.3.1:. Broere Method (2001)

Broere pointed out some important limitations of the current analytical methods and, consequently, developed a solution which can take into account the following relevant features:

• The heterogeneity of the ground at the face.

• The soil arching effect in the evaluation of the vertical load.

• The effect of the penetration of the support medium into the tunnel face in terms of excess pore water pressure.

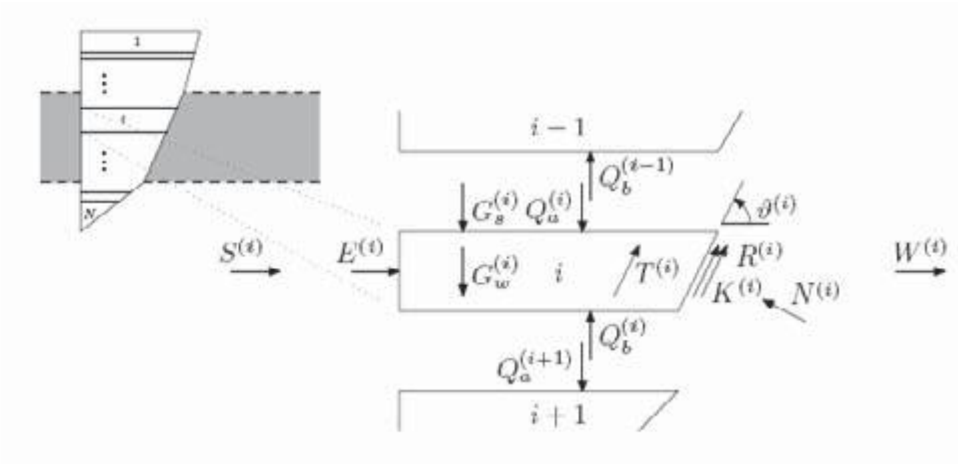


Figure 4.3.11.1: Definition of symbols for the multilayered wedge model of the Broere method [7].

The heterogeneity of the ground created, for example, by the presence of different stratified soils, is analyzed by assigning a set of geotechnical properties and calculating the relative weights and the forces, which are acting at each homogeneous layer, at each interface, and along the sliding surfaces.

Broere (2001) pointed out that for the simplified case of a single slide wedge in homogeneous soil, the resultant formulation corresponds to the result obtained by Waltz (1983) and Jancsecz (1994).

The Terzaghi theory, as well as the results of centrifugal test, suggests that part of the column above the wedge does not act as a load on the wedge.

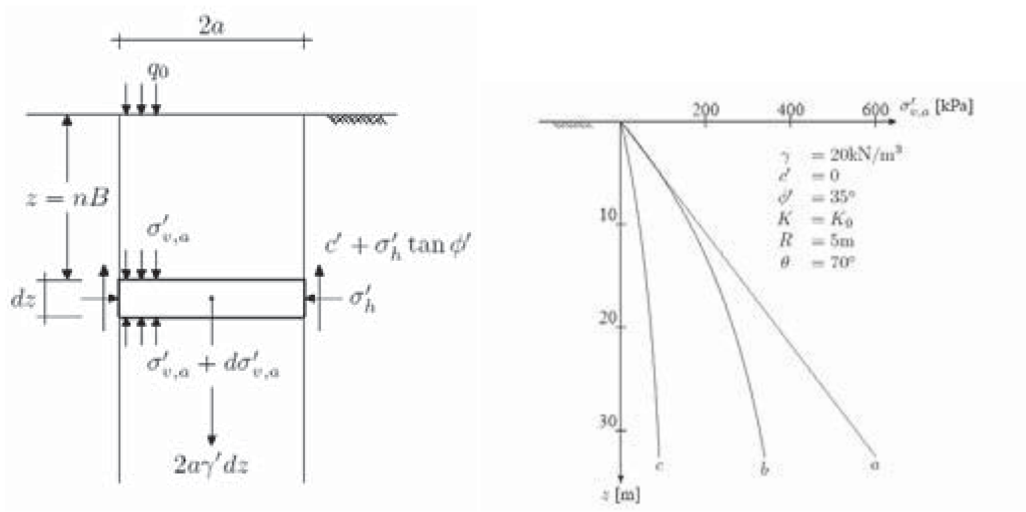


Figure 4.3.11.2 : Definition of forces acting on a strip of soil in an arching soil column according to the Terzaghi theory [7]

For the layer "*i*" with top $z = t(1)$the following formulation is proposed for a stratified soil, in the range $t(i) \leq z < t(i+1)$:

$$\sigma'^{(i)}_{v,a} = \frac{a\gamma'^{[i]} - c'^{[i]}}{K^{[i]} \tan\phi'^{[i]}}\left(1 - e^{-K^{[i]}\tan\phi'^{[i]}\frac{z}{a}}\right) + \sigma'^{[i-1]}_{v,a}(t_i) \cdot e^{-K^{[i]}\tan\phi'^{[i]}\frac{z}{a}}$$

The symbols used in the equation are explained in Figures 4.3.11 and .4.3.12.

Different hypotheses about the relaxation length "a" have been formulated and, finally, assuming a width of the wedge equal to the tunnel diameter, the following equations are proposed:

a. $a = \infty$ (no arching effect)

b. $a = R$(bi-dimensional arching effect, with R = radius of the tunnel)

c. $a = R/(1 + \tan\theta)$ (three-dimensional arching effect, where θ is the angle of the sliding surface).

An example of implementation of the three different approaches is reported in Fig.4.3.11.2. Furthermore, specific considerations have been formulated also for thee valuation of the horizontal stresses acting along the wedge sides.

However, the main issue of the model proposed by Broere involves the effect of the penetration of the support medium into the tunnel face in presence of permeable soil. As already described (method 4.3.10 by A-K), different mechanisms can occur depending on the permeability of the soil and the density of the support medium

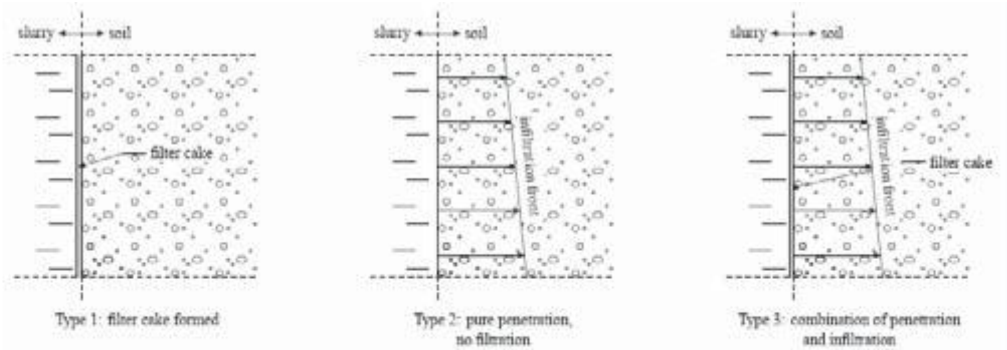


Figure 4.3.11.3: Typical cases of slurry infiltrations [7]

It should be noted that the model with penetration can refer to Slurry Shields, where bentonite slurry is injected, as well as to EPBS, where instead polymer foams are injected.

Particularly, the model of Broere differs from the A-K model in that the penetration of the medium during the excavation may produce an excess in the pore pressure in front of the TBM, as well as a reduction of the effective support force. This phenomenon can be considered significant when excavating soils with permeability in the range of 10–5–10–3m/s. As a consequence, the required support pressure could be significantly higher than that predicted by A-K method.

The effective support pressure (s') at the top of the tunnel face (zt) can be calculated using the equation below, by maximizing the value of $s'(zt)$ with respect to the wedge angle θ:

$$s'(zt) = [Gs - Ps + Gw + K' + 2T' - 2PT + S'\mathrm{dev}]/Z$$

where: Gs = overburden force on wedge; Ps = uplift force due to excess pore pressure; Gw = effective weight of the wedge; K' = effective cohesive force along sliding surface; T' = resultant friction force on the wedge side; PT = shear force reduction due to excess pore pressure; $S'dev$ = deviatoric support force; Z = a parameter which is a function of $(\varphi - \theta)$.

Given the slurry density the total support pressure can then be found by adding the total pore pressure at the far end of the wedge, i.e:

$$s(z)= s'(z) + p(w(z),z)$$

in which z is the considered depth and $w(z)$ is the corresponding width of the wedge.

Now the excess pore pressure Δs can be defined as the difference between the support pressure and the pore pressure at rest $p0$.

Broere developed specific equations to evaluate the distribution of the pore pressure in the penetrated ground, as a function of the support pressure, as well as of the pore pressure at the rest, time, property of the soil and of the muck.

An intense monitoring program from the surface supported by COB (the Dutch Centre Underground Bowen) during the construction of three tunnels in Netherlands (2 by Slurry Shield and 1 by EPB) gave the possibility to verify a good correspondence between the predicted and measured values of excess pore pressure, confirming this type of occurrence up to about 30 m in advance of the tunnel face.[7]

4.3.12: Caquot-Kerisel (1956) Method as Integrated by Carranza-Torres (2004).

Statistically admissible solutions – based on lower and upper bounds theorems of plasticity – are normally considered to be more rigorous than the limit equilibrium solutions. Among statically admissible solutions we can mention the solutions by Caquot (Caquot *et al.*, 1956); these solutions are derived for 2D circular tunnel sections but can be easily extended to consider a 3D spherical geometry.

Caquot's model considers the equilibrium condition for material undergoing failure above the crown of a shallow circular (cylindrical or spherical) cavity. The material has a unit weight γ and a shear strength defined by Mohr-Coulomb parameters c (cohesion) and φ (friction angle), while the distribution of vertical stresses before excavation is lithostatic and the ratio of horizontal to vertical stress is 1. A support pressure ps can be applied inside the tunnel, while a surcharge qs (from infrastructures or embankments) acts on the ground surface. For the situation presented in Figure 4.3.12 Caquot's solution defines the value of internal pressure (ps) as the minimum or critical pressure below that the tunnel will collapse. The Caquot generalized solution for dry conditions (which include the factor of safety, FS), can be represented by the following equation developed by Carranza and Torres (2004) [25].

$$\frac{p_s}{\gamma a} = \left(\frac{q_s}{\gamma a} + \frac{c}{\gamma a}\frac{1}{\tan\phi}\right)\left(\frac{h}{a}\right)^{-k\left(N_\phi^{FS}-1\right)}$$

$$-\frac{1}{k\left(N_\phi^{FS}-1\right)-1}\left[\left(\frac{h}{a}\right)^{-k\left(N_\phi^{FS}-1\right)}-1\right]-1\frac{c}{\gamma a}\frac{1}{\tan\phi}$$

where: a = the tunnel radius; h = axis depth below the surface; k = parameter that dictates the type of excavation [1 = cylindrical tunnel; 2 = spherical cavity]. It should be noted that Equation (1) is valid only when the given Mohr-Coulomb parameters lead to a state of limiting equilibrium – the situation in which the excavation is about to collapse. In general, the strength of the material will be larger than the strength associated with the critical equilibrium state of the cavity.

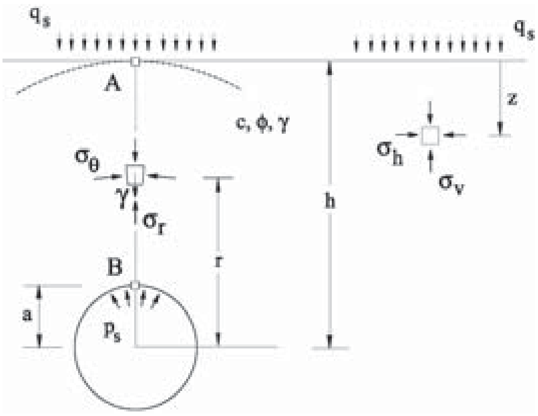


Figure 4.3.12.1: Basic scheme for the Caquot-Kerisel solution (Carranza-Torres, 2004).

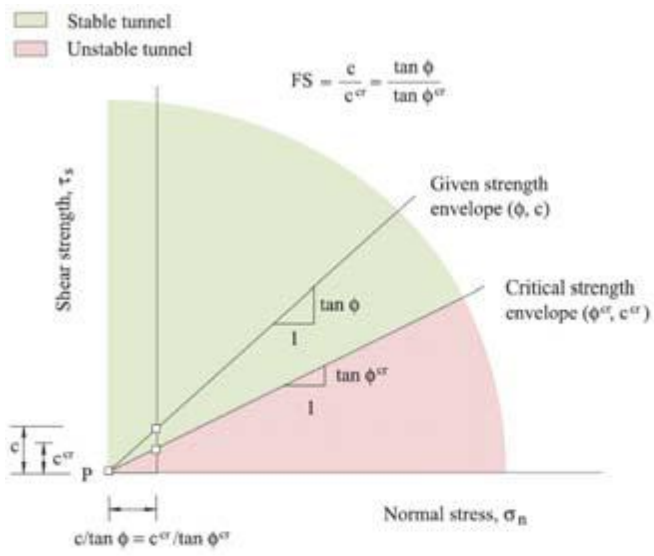


Figure 4.3.12.2: Strength reduction method (Carranza-Torres, 2004).

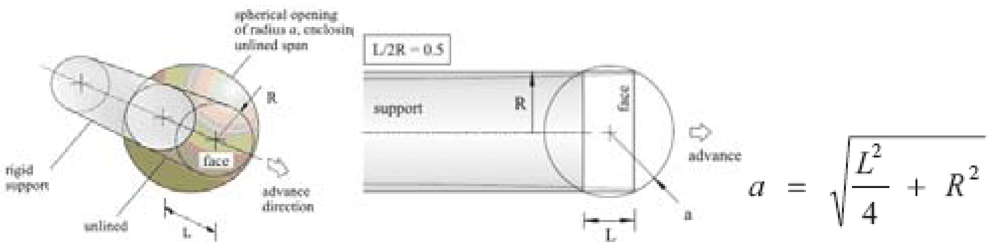


Figure 4.3.12.3: Calculation of the modified tunnel radius for face stability analysis (C. Carranza-Torres, 2004).

The factor of safety *FS* is defined as "the ratio of actual Mohr-Coulomb parameters to the critical Mohr-Coulomb parameters", as expressed in the following equations (Strength Reduction Method, Dawson *et al*., 1999); as indicated in Figure 5.15, this approach assumes a proportional reduction of the Mohr-Coulomb parameters [1].

$$N_{\phi}^{FS} = \frac{1+\sin\left(\tan^{-1}\frac{\tan\phi}{FS}\right)}{1-\sin\left(\tan^{-1}\frac{\tan\phi}{FS}\right)} \quad FS = \frac{c}{c^{cr}} = \frac{\tan\phi}{\tan\phi^{cr}}$$

5. KADIKOY - KARTAL METRO PROJECT

Metro line consisting of 21.6 km of two tubes starts from Kadıköy Rıhtım Caddesi, passing under İbrahimağa area and after Koşuyolu Bridge it follows the line under E-5. The project consists of 16 stations. A general view of Metro Line is seen in Figure5.1.

5.1. General geology

The Metro Line is planned to pass into Ordovician and Carboniferous aged sedimentary formations. Alluvial deposits are found in the river bed found in the same area.

The main rock formations are as follows:

Kurtköy Formation

29.7 % of the tunnel will pass into Kurtköy Formation which mainly consists of purple colored, sandstone, conglomerate and mudstone. Rocks consist of 70 % quartz grains, 20 %altered feldspars, 2-3 % micas and 1-2 % of opaque minerals.

Kartal Formation

29.9 % of the tunnel will pass into Kartal Formation. It mainly consists of limestone wit hinter bedded siltstone, sandstone and shale.

Dolayoba Formation

17.5 % of the tunnel will pass into Dolayoba Formation mainly consisting of limestone.

Trakya Formation

13.60 % the tunnel will pass into Trakya formation which is well known formation consisting of inter bedded siltstone and sandstone

. Alluvial deposits, Yelkentepe Formation, Denizli Formation, Gözdağ and Aydos Formations

9.3 % of the tunnel will pass into these formations. [20]

Figure 5.1: Route Of the project

5.2. Excavation in Mixed-Face Ground Conditions

During excavation of Kadıkoy-Kartal Metro Project collapses occurred around Kozyatagi Bridge due to fault zones in that area. After this situation contractor started a research in order to have a better understanding of risky areas along the tunnel route. After this research a risky area (Hasanpasa crossroad 3+700 – Otosan field 4+200) detected between Kadikoy – Kozyatagi stations [21].

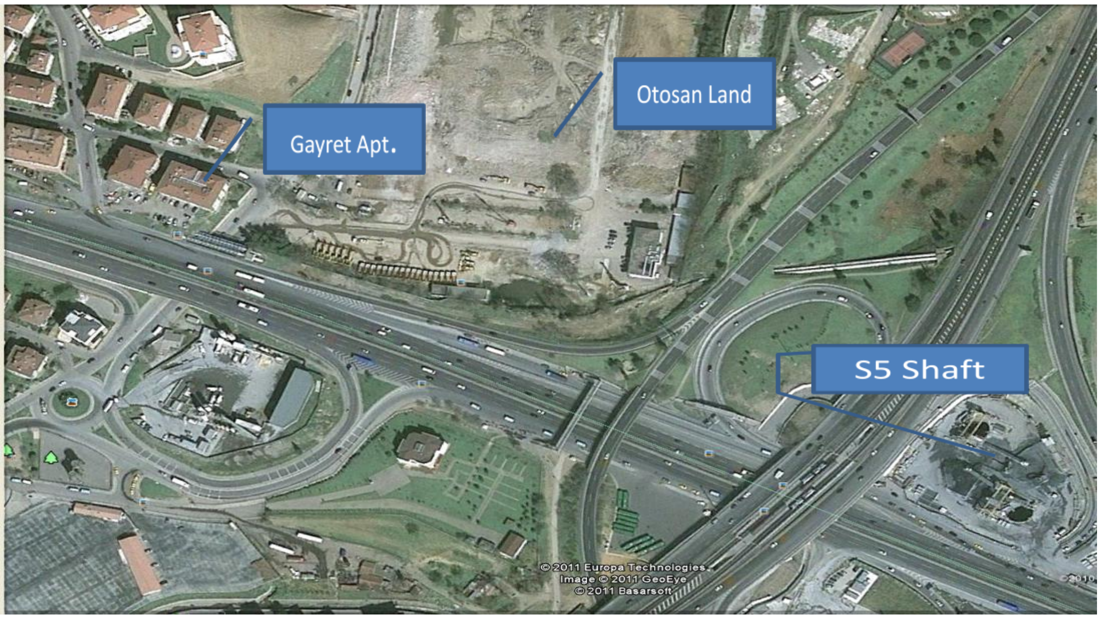


Figure 5.3.1: The Route of Hasanpasa-Otosan Land Tunnels (Google Maps)

5.4. The Geology of Between Hasanpasa Crossroad and Otosan Land Alignment

Along this alignment there are 4 main geological formations Kartal formation, Tuzla formation, volcanic units (diabase dykes) and alluvium. The geological cross-section of the alignment is given in figure 5.4.1.

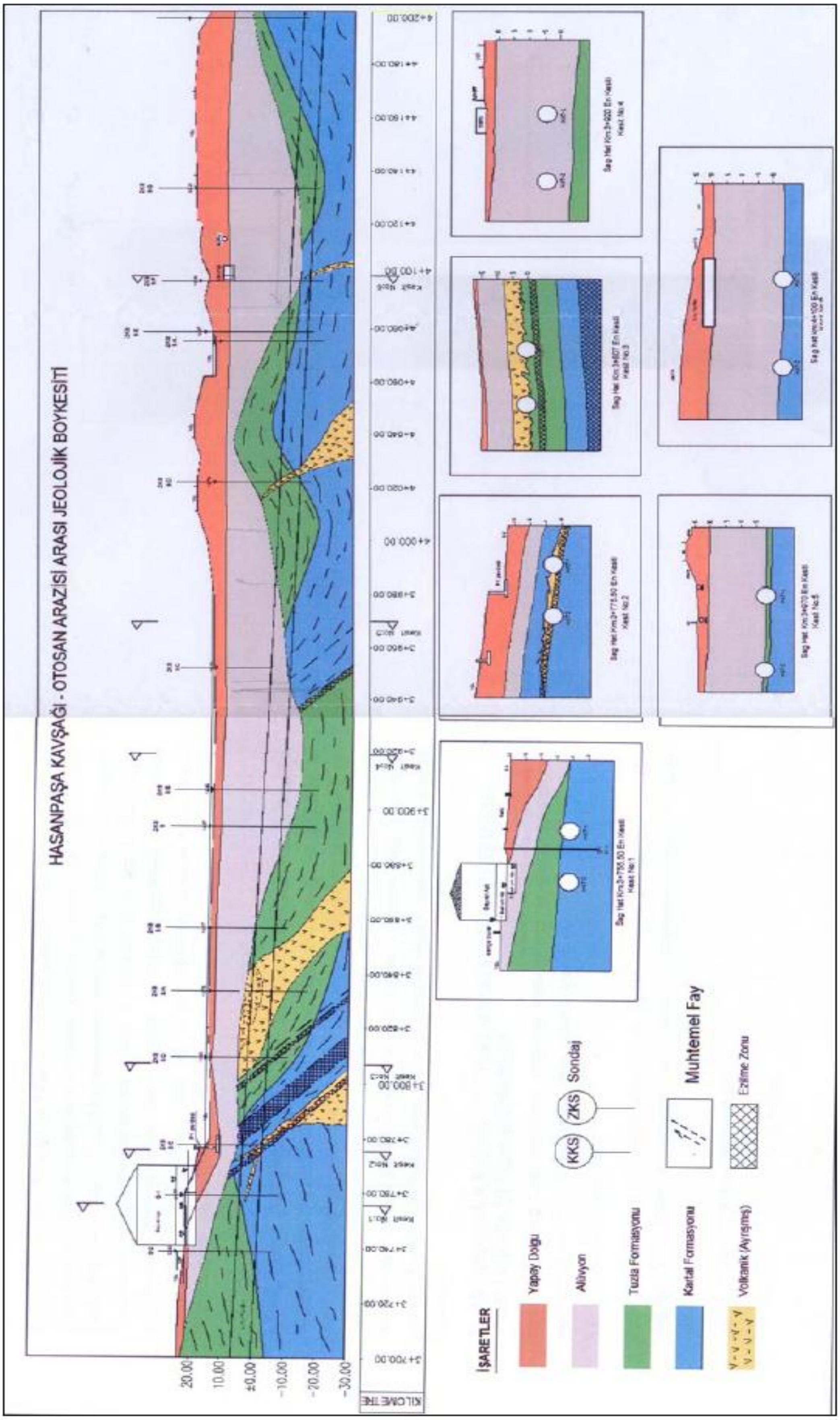


Figure 5.4.1: The Geololigical cross-section of Hasanpasa Crossroad- Otosand Land

5.5. The Geotechnical Features of Alignment

Rock Strata:

The RQD value of 55% rocks along route is under 25, which means the rock mass in that area is very weak. and it may cause instabilities.

Table 5.5.1.:RQD values of formation between Hasanpasa Crossroad – Otosan Land [21]

Formation	Kilometer	RQD %
Kartal For.	3+740 – 3+770 4+030 – 4+070	22,8
Tuzla For.	3+700 – 3+740 3+790 – 3+810 3+850 – 3+860 3+990 – 4+020	34
Volcanic unit	3+760 – 3+780 3+810 – 3+840 4+020 – 4+030	23,7

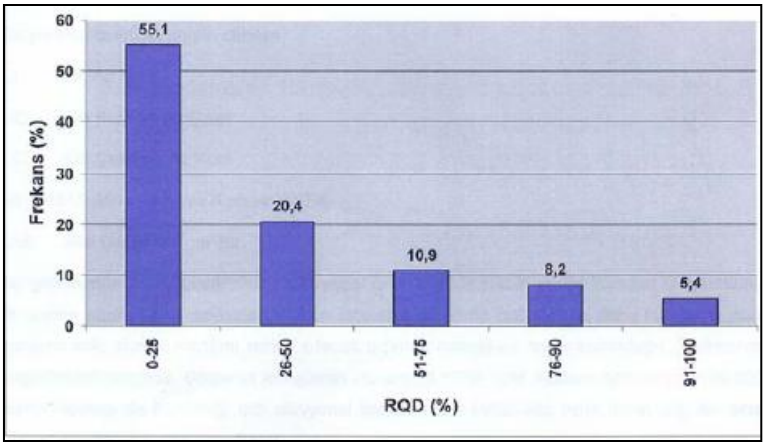


Figure5.5.1: RQD frequency of formation between Hasanpasa Crossroad and Otosan Land

Soil Strata

The 50 % of soil strata is clay and 50 % is sandy clay and gravely clay along the alignment which may cause tunnel stability problems.

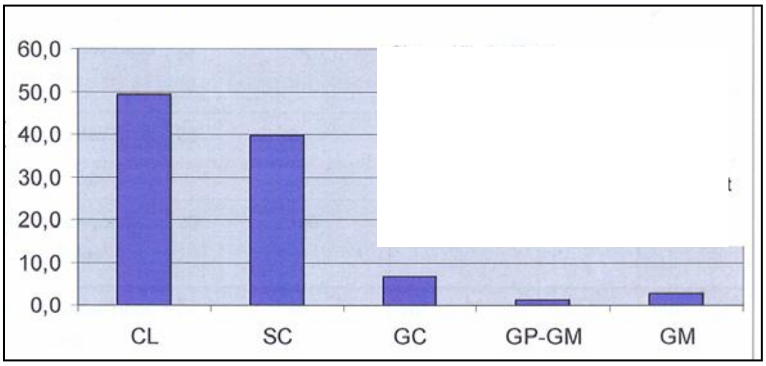


Figure 5.5.2.: The frequency of soil types

5.6. Design Parameters of Rock and Soil Strata

Some of design parameters of tunnels are given beloved.

Table 5.6.1: Some design parameters of tunnel [21]

KM	Formation	Unit weight (kg/cm³)	Cohesion (kPa)	Friction angle	Poisson ratio	Elasticity Modulus (Mpa)
3700-3775	Kartal /Tuzla	24	80	30	0,30	250
3775-3865	Kartal /Tuzla	23	0	30	0,30	100
3865-3980	Aluvion	20	100	0	0,35	30
3980-4095	Kartal /Tuzla	24	80	30	0,30	250
4095-4140	Kartal /Tuzla Alluvium	20	100	0	0,30	30
4140-4200	Kartal /Tuzla	24	80	30	0,30	250

5.7. Expected Problems during Excavation

Tool Wear

Normally in mixed face conditions, the normal methods used for the determination of the cutter tool wear, mainly based on the abrasivity of the ground and the length run by the disk cutters, are not adequate to achieve a realistic estimation of the cutter's life.). Experience in mixed face conditions has shown that the average cutter life can lie well under 100 m3 of excavated ground per disk cutter.

The main causes for the high tool wear are listed below.

_ Overload of the disk cutters.

_ Poor material inflow which results in secondary wear and clogging of the disks.

_ Transversal shocks of the cutters against the blocky ground when the cutter head turns, which can eventually result in blocking of the cutters.

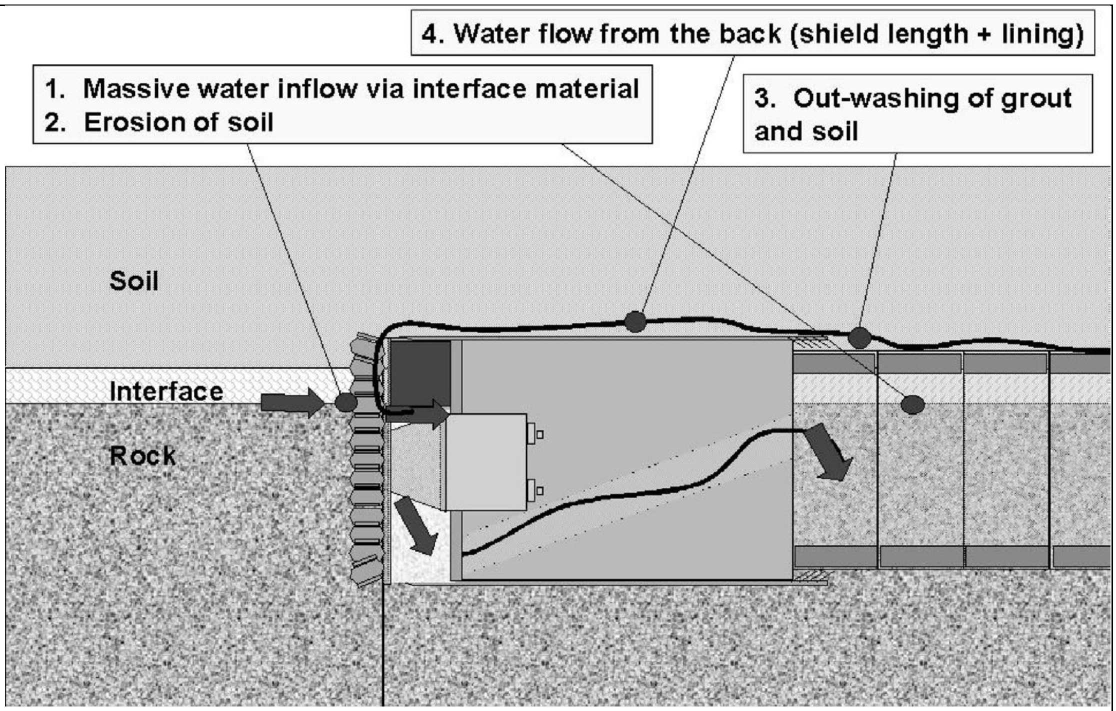
Furthermore, blocky face conditions do not allow the disks to work in the suitable conditions to attain their expected penetrations. Hence, advance rates are further reduced [23].

Tunnel Face Stability

It is clearly reported in (Steingrímsson, Grøv, and Nilsen 2002) that experience yields a decrease of the applied thrust in mixed face conditions, in order to reduce the massive vibrations caused by the cutters bouncing on and off strong and weak layers. Additionally, as mentioned above, the total thrust must also sometimes be reduced in order not to overload the disk cutters [24].

Therefore, due to the deficient deformability of the earth supporting medium in mixed face conditions, such a decrease of the total thrust results in an unwanted decrease of the confinement pressure.

Hence, not only the control on the load transmitted to the cutter tools is partially lost, but also the confinement pressure control is hindered, and thus the risk of ground failure increases.



1. Disturbance of the water pressure which creates a loss of equilibrium and a pressure gradient.
2. Water inflow into the mining chamber through the interface layers and into the annular gap.
3. Consequently, the erosion of the soft ground begins and its shear parameters become poorer. The area affected by the water flow and the erosion extends.
This results at the same time in a dramatic increase of the water inflow.
4. Erosion of the grout in the annular gap occurs.

Figure5.7.1. : Representation of the erosion and water inflow process in mixed face conditions [23]

Furthermore, the contact zones between soft soil and a hard rock are often related to an increase of the permeability. The higher permeability of the contact zone can yield water inflows into the mining chamber. A water pressure gradient is then created between the tunnel face and the annular gap at the tail shield. This process can eventually lead to erosion of the soft ground and of the grout injected in the annular gap in the way described next and illustrated in Figure 5.7.1.

As mentioned before, due to low overburden and difficult geological conditions settlements will occur. At this areas contractor run a successful risk management plan as seen Figure 7.2. Gayret Apartment was one of the risky areas during excavation.

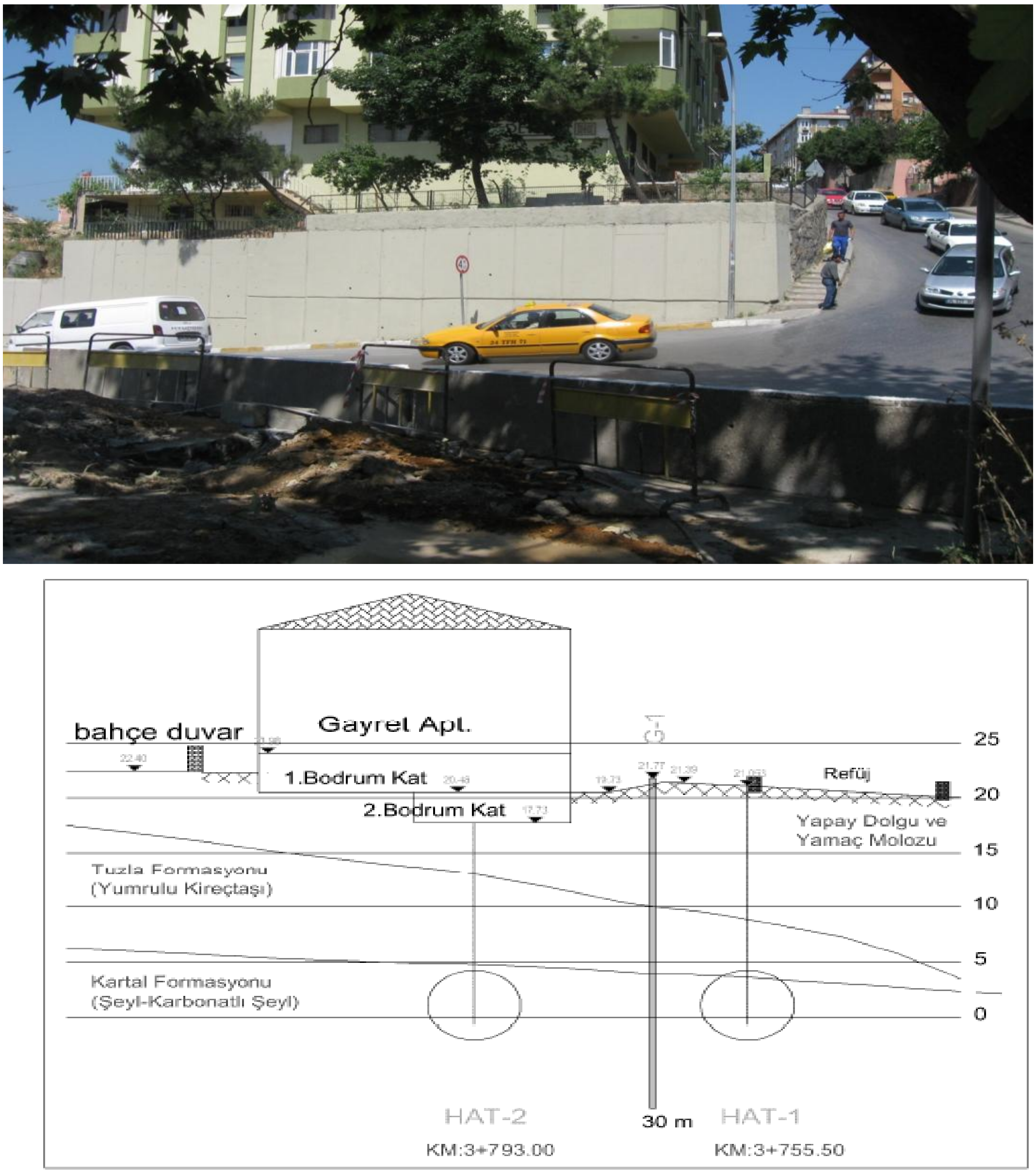


Figure 5.7.2.: Shallow overburden problem under passing Gayret Apt.

6. ESTIMATING AND CONTROLLING OF EARTH PRESSURES

There are many factors that affect the earth pressures mobilized during EPB tunneling. They may be divided into two major categories:

Static Factors: soil type, soil strength, ground water table

Dynamic Factors: rate of tunneling, mode of tunnelling (drained, undrained)

In practice only static factors are considered in theoretical earth pressure estimations.

There is a direct association between the earth pressure acting on shield or tunnel lining and the ground displacements along the tunnel at different stages of excavation, as shown in Figure 6.1. [4]

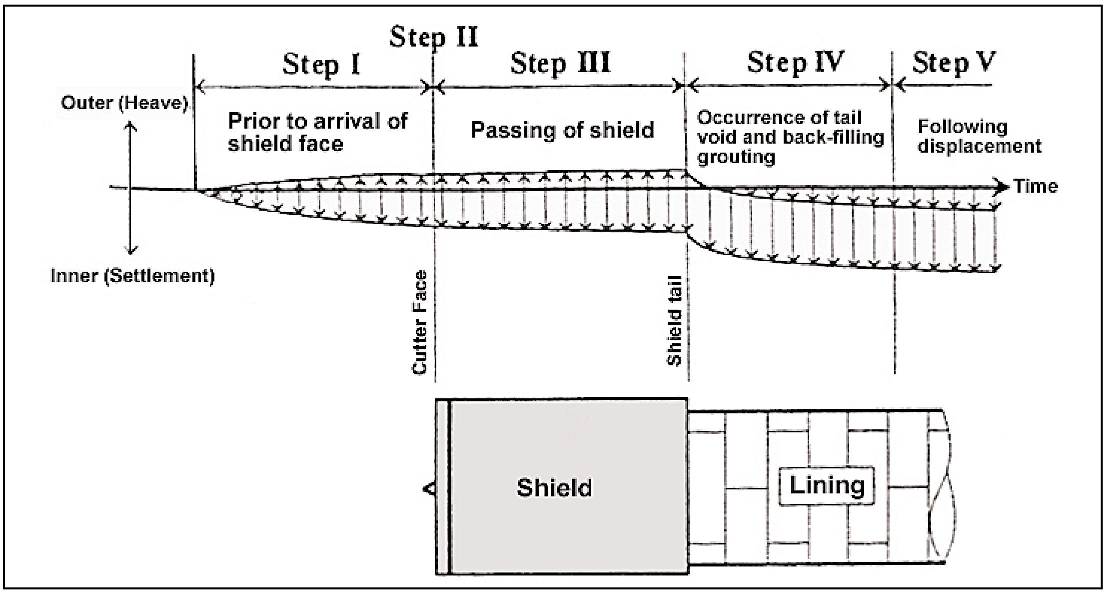


Figure 6.1.: Ground displacements along the tunnel at different stages of excavation.

In previous chapter face pressure calculation methods were presented. For Kadikoy Kartal Project three models are chosen taking into consideration of the geological conditions and input data. The geometrical methods are given below in figure 6.2., 6.3, 6.4

METHOD OF JANCSECZ & STEINER (1994)

According to the model of Horn (1961), the three-dimensional failure scheme consists of a soil wedge (lower part) and a soil silo (upper part). The vertical pressure resulting from the silo and acting on the soil wedge is calculated according to Terzaghi's solution.

A three-dimensional earth pressure coefficient ka3 is defined as:

ka3 = (sinβ · cos · – cos2β · tanφ – K · α · cosβ · tanφ/1.5)/(sinβ · cosβ + sin2β · tanφ)

where:

K ≈ [1 – sinφ + tan2(45 + φ/2)]/2;

α = (1 + 3 · t/D)/(1 + 2 · t/D).

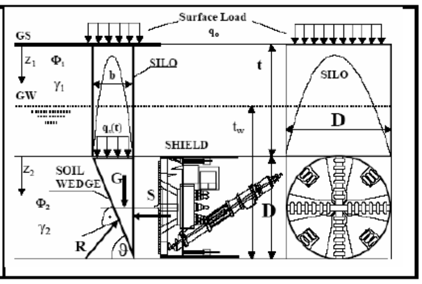


Figure 6.2.: Method of Jancsecz and Steiner

METHOD OF ANOGNOSTOU & KOVARI (1996)

This method, later referred to as A-K method, is based on the silo theory (Janssen 1895) and to the three-dimensional model of sliding mechanism proposed by Horn (1961). The analysis is performed in drained condition, and a difference between the stabilizing water pressure and effective pressure in the plenum of an EPBS is presented. If there is a difference between the water pressure in the plenum and that in the ground, destabilizing seepage forces occur and a higher effective pressure is required at the face. However, accepting this flow, the total stabilizing pressure is lower than the pressure required in the case of an imposed hydrogeological balance. The effective stabilizing pressure (σ') :

σ' = F0 · γ' · D – F1 · c' + F2 · γ' · Δh – F3 · c' · Δh/D

where F0,F1,F2,F3 are non-dimensional factors derived from normograms, which are function of H/D and ϕ'.

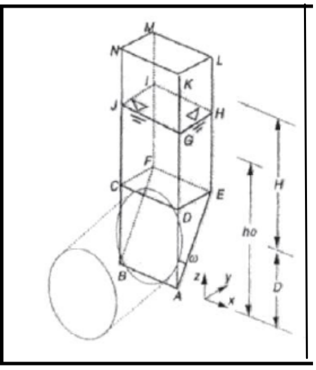
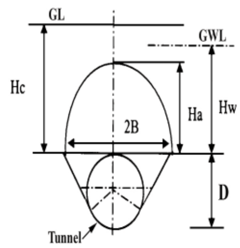


Figure 6.2.: Method of Anognostou and Kovari

TERZARGHI'S SILO THEORY

This method assumes limit equilibrum state of soil in front of the TBM



$$\sigma v=\frac{\gamma B-C}{k_0\tan\varphi}\left(1-e^{-k_0\tan\varphi Hc/B}\right)+P_0e^{-k_0\tan\varphi Hc/B}$$

Figure 6.3.: Method of Terzaghi

Considering calculation methods face pressures estimated using input data and results are given Table6.1.

Table 6.1.: Results of face pressure estimations

	Estimated Face Pressures (bar)			
Km	Anognostou and Kovari	Terzaghi	Jancsecz and Steiner	Observed
3775	2.30	2.75	1.72	2.02
3865	2.50	2.90	1.75	2.02
3960	2.80	3.00	1.75	1.86
4095	2.60	2.90	1.75	1.82
4140	2.20	2.45	1.63	1.65

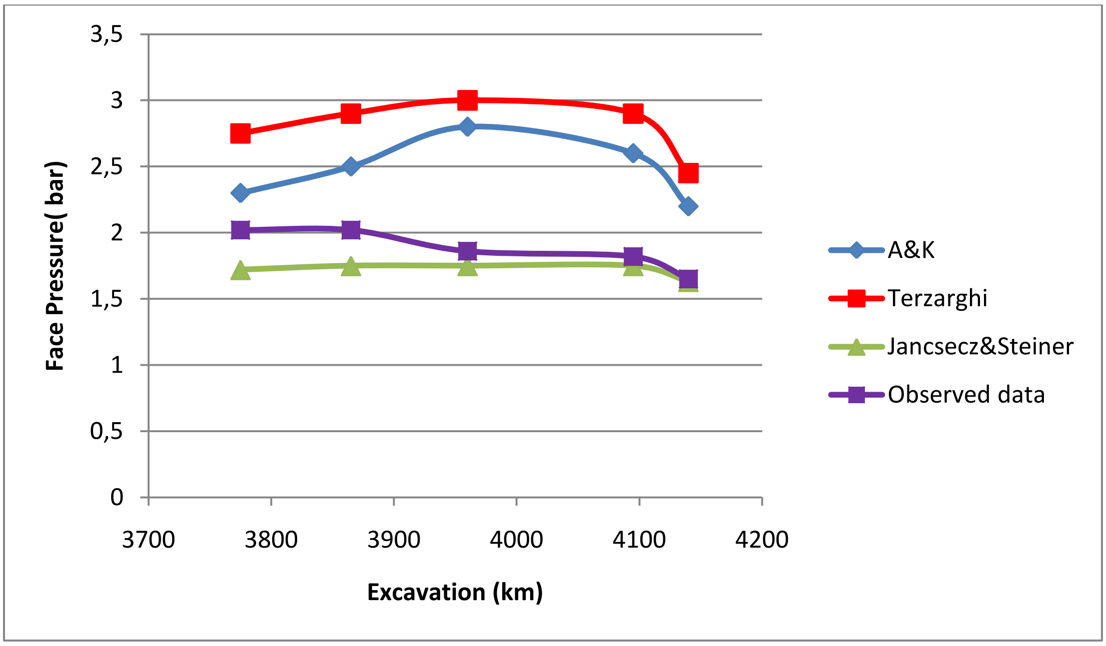


Figure 6.4. : Face pressure graph along section

Figure6.4. presents the estimated and observed face pressures for given chain observed values shows that face pressure is almost constant about 2 bars, and the best fitting calculation method is Jansecz and Steiners ‘ method. But considering the geological and physical conditions of the section applied face pressure should be increased during the passing alluvium (km 3900-4000) .This adversative opinion could be support by the operational difficulties (collapses low advance rate and difficulties to maintain face support) occurred during the excavation of this area. In the light of these information’s best fitting model for this section is Anognostou and Kovari’s method.

7. ANALYSIS OF PERFORMANCE DATA COLLECTED DURING CONSTRUCTION OF TUNNEL

This section presents the information obtained during the construction of tunnel. The section presents the characteristics of the excavated material, the information logged manually and by TBM data logger during tunneling.

7.1. Information Collected from TBM Data Logger

During construction of Kadikoy- Kartal metro tunnels data logging was undertaken by contractors in automatic form. After that for each ring torque, thrust, advance rate and face pressures were analyzed. A sample sheet of performance data taken from TBM data logger is given in Figure 7.1.1.

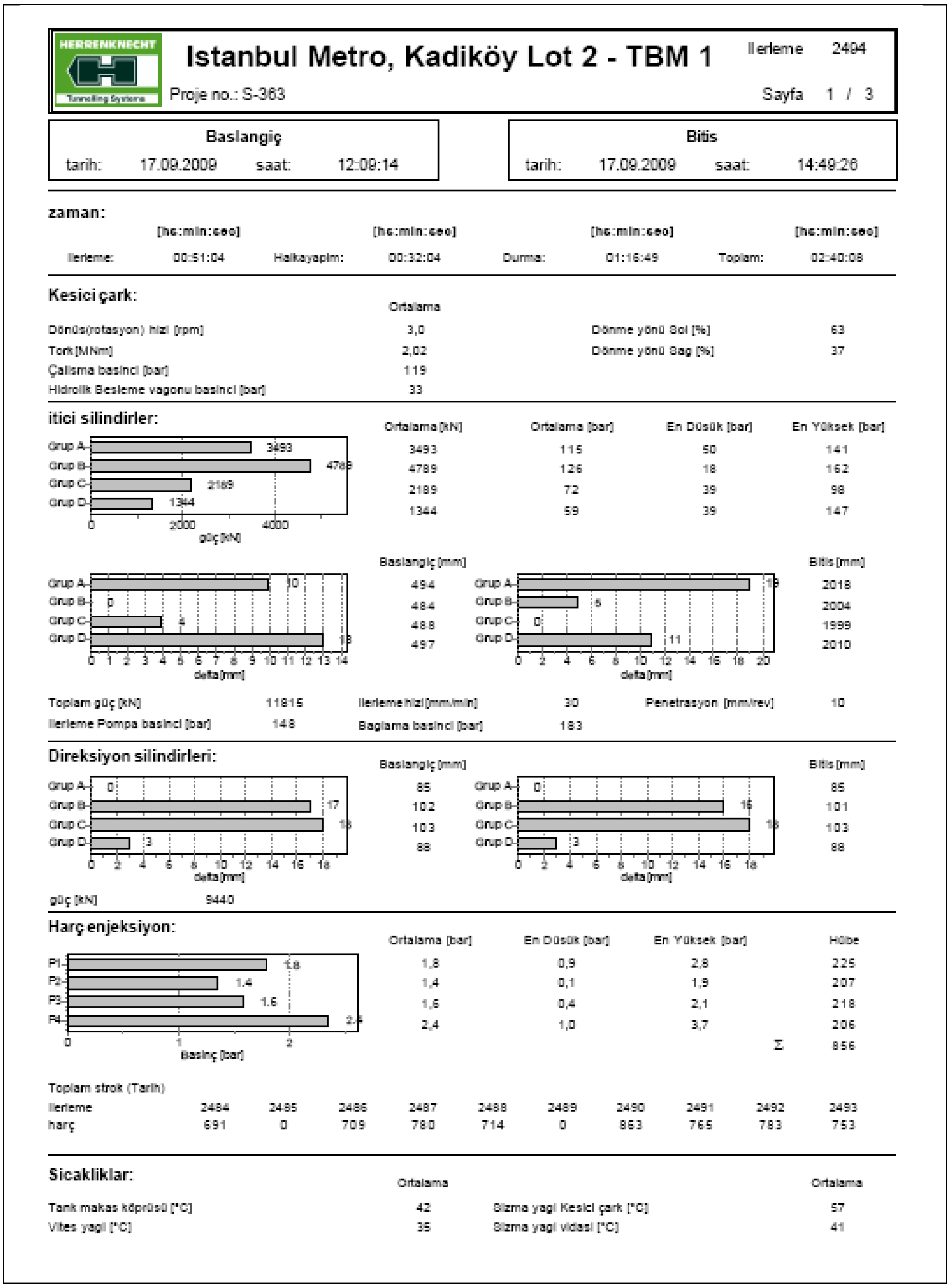


Figure7.1.1: A sample sheet of performance data from TBM data logger

7.1.1. Operational Torque

Figure 7.1.1.1.presents the operational torque of cutting head during the construction of tunnel. Shown on the plot are the mean of torque applied during excavation. The mean torque was determined by identifying all log entries made while TBM was advancing along the alignment. These values were transferred to a separate sheet where they were averaged for each tunnel ring number.

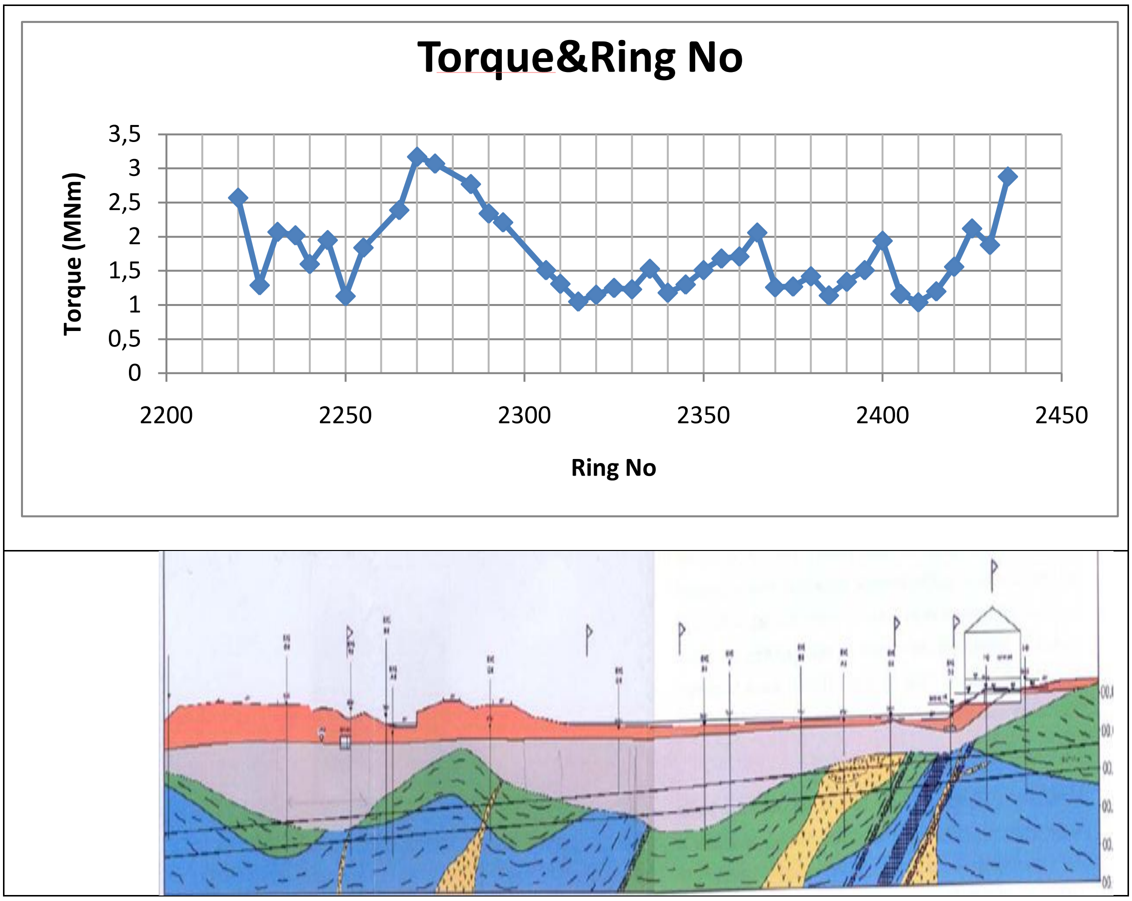


Figure 7.1.1.1.: The mean torque values along the alignment

It has been detected that during the excavation of Kartal formation (ring no 2225, 2260, and 2430) torque values increased up to 3 MNm and for alluvium (ring no 2340 – 2350) torque values decreased approximately 1 MNm. This could be explaining by the effect of geological features of ground on the TBM torque values.

7.1.2. Operational Thrust

Figure7.1.2.1. presents the operational thrust of TBM during the construction of tunnel. Shown on the plot are the mean of torque applied during excavation. The mean thrust was determined by identifying all log entries made while TBM was advancing along the alignment. These values were transferred to a separate sheet where they were averaged for each tunnel ring number

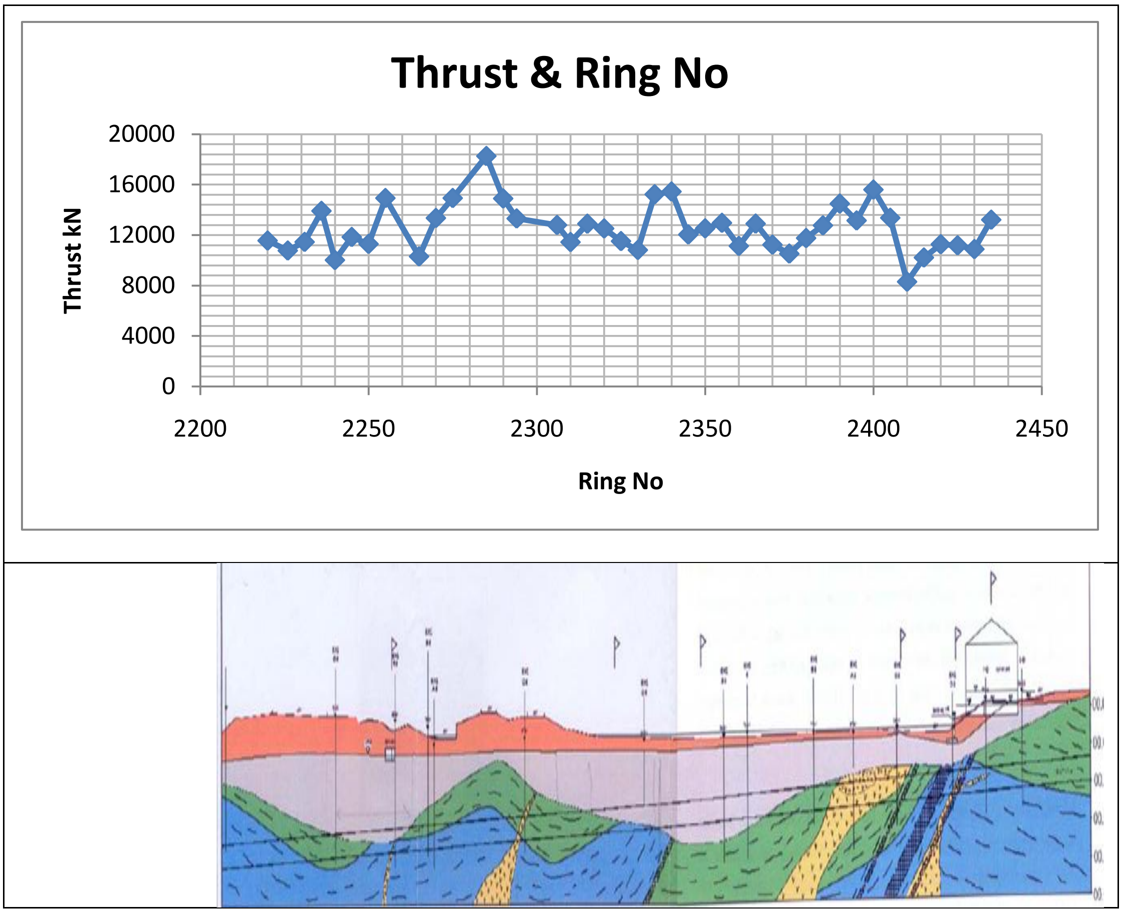


Figure 7.1.2.1.: The mean thrust values along the alignment

The mean thrust values shows that, during the excavation of contact zones and mixed face thrust values increased especially the passing of dykes thrust values increased up to 18000 kN.

7.1.3. Instantaneous Penetration Rate (IPR)

Figure 7.1.3.1. presents the instantaneous penetration rate of TBM during the construction of tunnel. Shown on the plot are the mean of instantaneous penetration rate during excavation. The mean instantaneous penetration rate was determined by identifying all log entries made while TBM was advancing along the alignment. These values were transferred to a separate sheet where they were averaged for each tunnel ring number

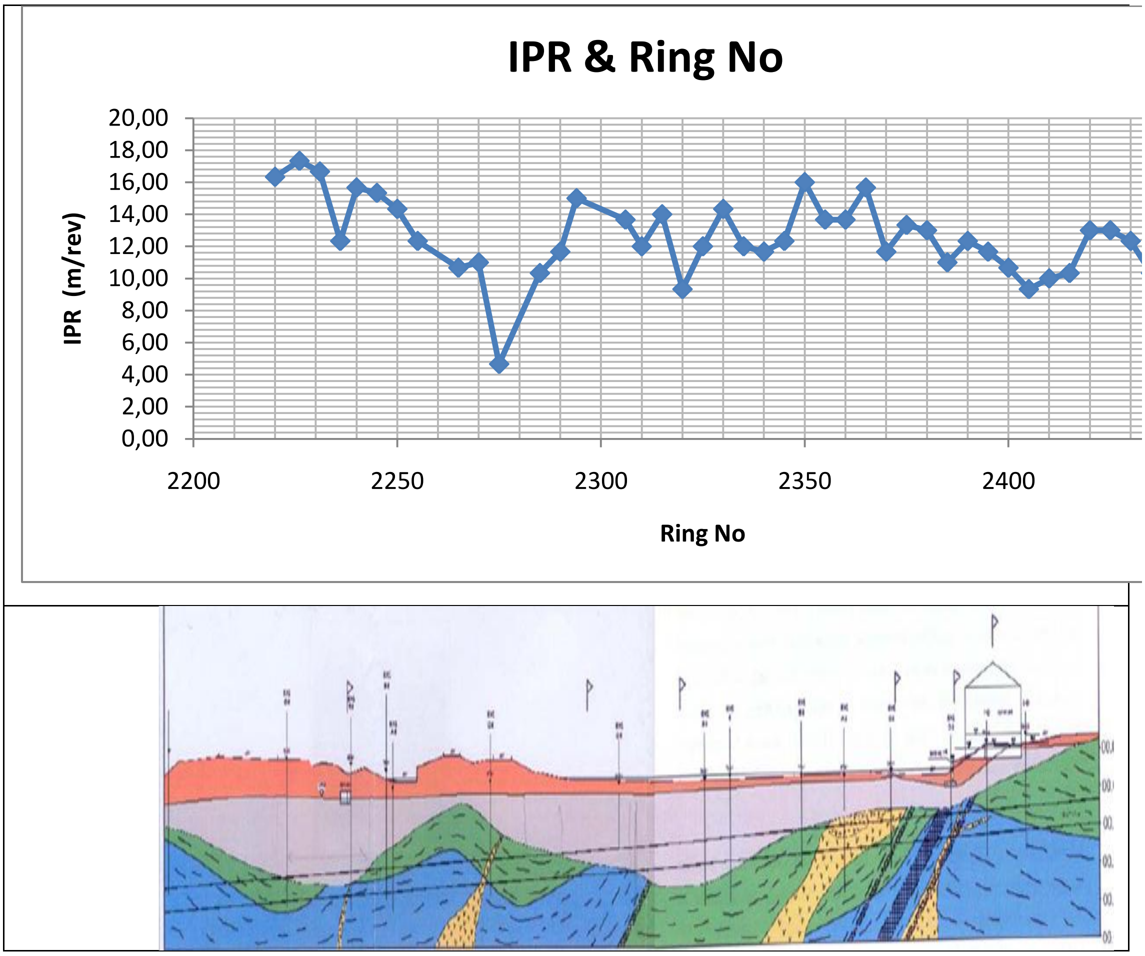


Figure 7.1.3.1.: The instantaneous penetration rate values along the aliment

Before the excavation of this section it was projected that the changing geological conditions would have a major affect on the instantaneous penetration rates. However looking closely the observed instantaneous penetration rates it has been seen that advance rates are similar. This situation could be explained by effect of operational conditions. When examining ring number 2275 it has been detected that advance decreased dramatically; during the excavation of this ring a collapse occurred so it affected the advance of TBM.

7.1.4. Observed Face Pressures

Figure 7.1.4.1.presents the face pressures logged during construction. The face pressure data was obtained from pressure sensors shown in Figure 7.1.4.2. .The value for the mean face pressure was determined by using the data entries for face pressure over the entire installation of any given ring.

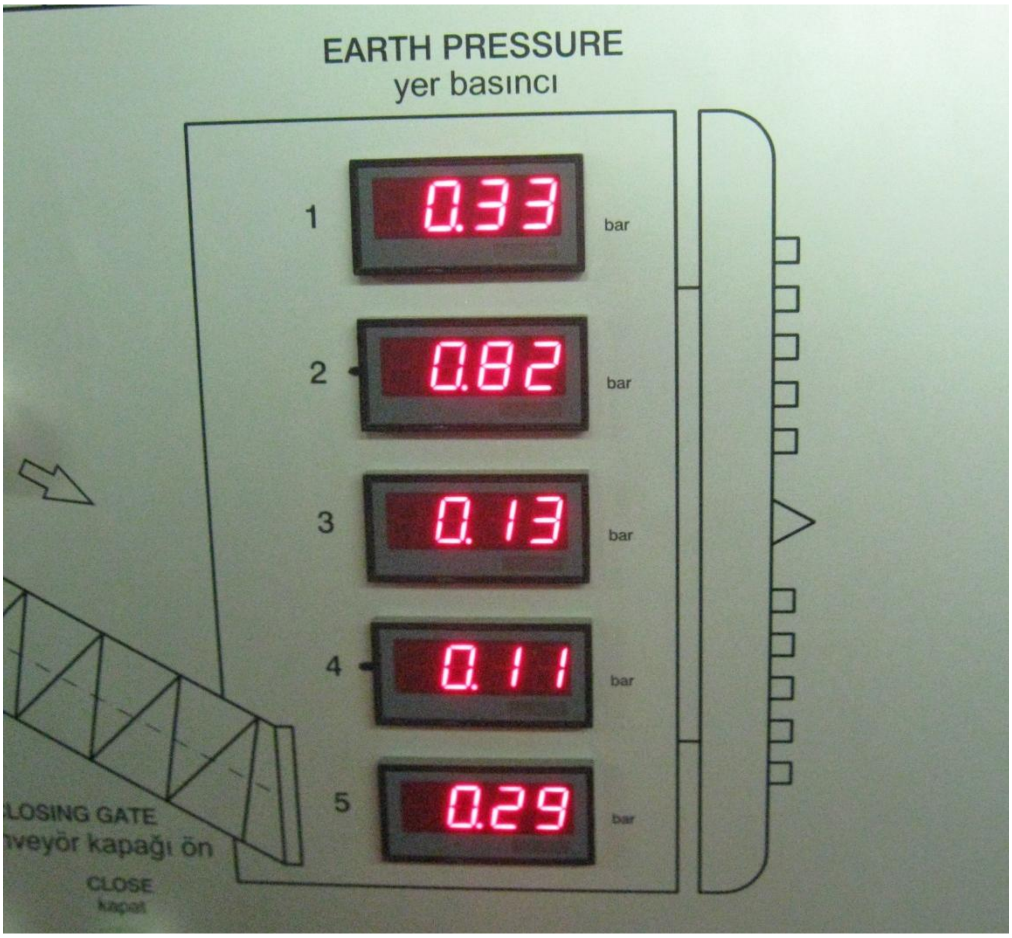


Figure 7.1.4.2: Face pressure sensors in operating cabin

During the analysis of face pressures for have better results only 3 sensors (1,2,5) are considered considering the clogging effect of sticky material around the screw conveyor.

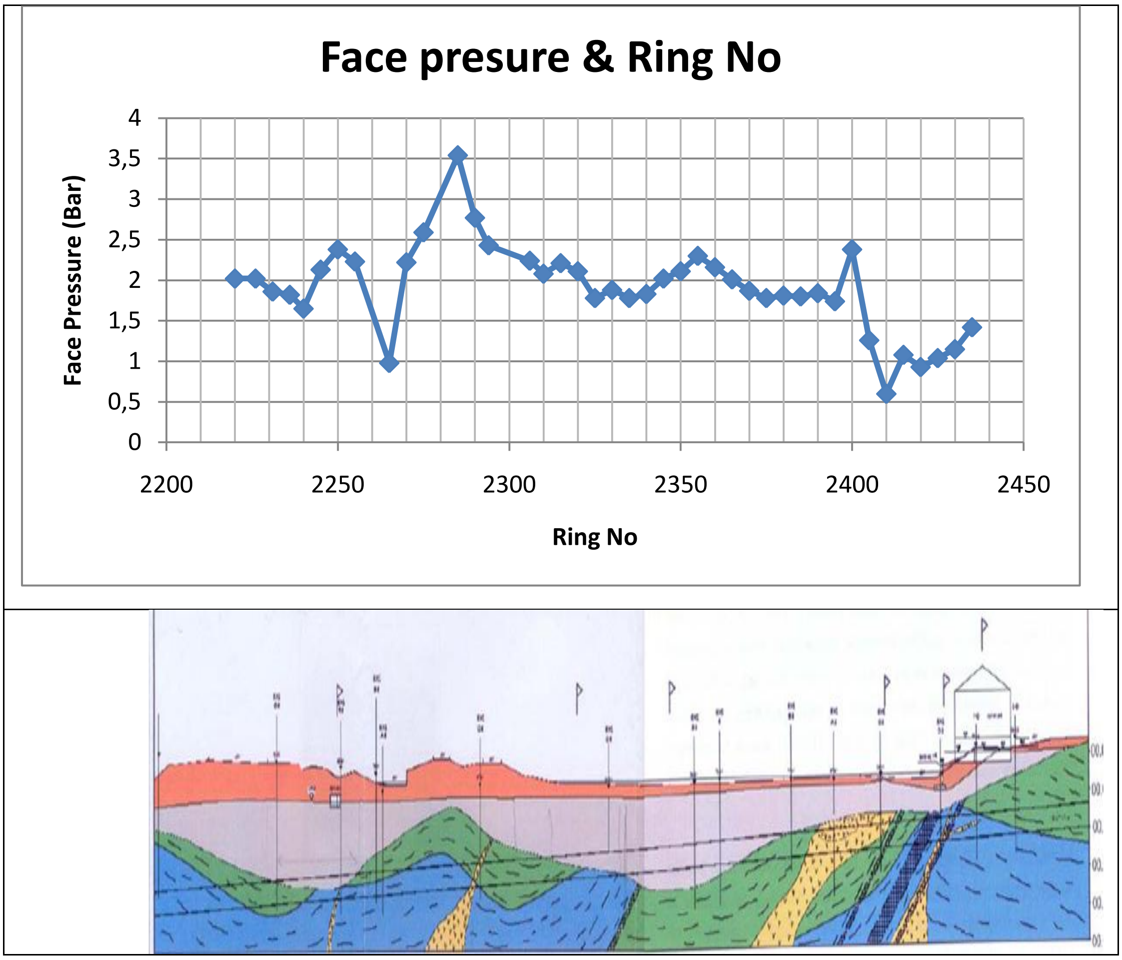


Figure 7.1.4.1.: The mean face pressure values along the alignment

Figure 7.1.4.1 shows that face pressures are changing between 1, 5 and 2 bars but when examining ring numbers 2265, 2270, 2275 and 2280 it is detected that face pressure increased dramatically. In the light of these values the shift reports checked and it had been seen that during the excavation of ring number 2265, required face pressure did not applied here at a collapse occurred and over filled the excavation chamber. During the excavation of following rings collapse enlarged and face pressures increased up to 3 bars.

7.2. Problems Faced During Excavation

In previous chapters mentioned that during excavations through mixed face conditions high tool wears expected shown in figure 7.2.1 .During the excavation of this section between rings 2275- 2290 laser cracks detected on disc cutters which could explain by overload of disc cutters and transversal loads shown in figure 7.2.2.

Figure7.2.1: Laser cracks on disc cutters

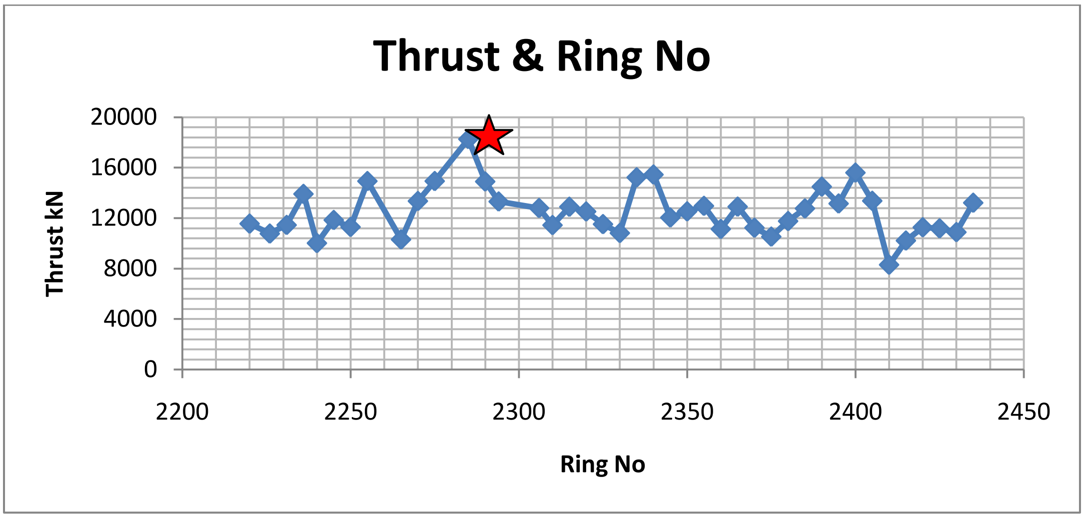


Figure 7.2.2: The mean thrust values along alignment

8. CONCLUSIONS

TBM performance in difficult geological conditions was analyzed in this study for Hasanpasa crossroad – Otosan Land alignment of Kadikoy–Kartal Metro, which is 500 meters. It has been detected that frequently changing geological conditions have a major effect on TBM performance, but not the only factor. Especially tunneling in urban areas, some other parameters such as shallow overburden, high safety risk and limited surveys also affect the performance.

Comparing with the previous daily productions it has been seen that daily advance rate decreased in this part of the alignment. This reduction on the performance was significant at especially dykes-main rocks contact zones. The selection of a mixshield tunnel boring machine can be considered as a correct choice for the construction of Kadikoy – Kartal Metro Project. Selection of closed shield tunnelling technology was based on the need of controlling the surface settlements and a better advance rate during tunneling in mixed face ground conditions. However, a collapse occurred during tunneling were not due to over mining which was well controlled but the complexity of the geology and low overburden.

Two buildings and a motorway were directly undermined by the construction of Hasanpasa Crossroad – Otosan Land alignment. For the first building Gayret Apartment, a successful risk management run in order to minimize the surface settlements. As mentioned before the foundation of building was strengthened enough to stand the tunnelling induced settlements. The settlements observed did not exceed the criteria established in the construction documents.

Face pressures for different chainages were estimated by using theoretical methods developed by Anagnostou and Kovari (1996) [16], Jancsecz and Steiner (1994) [14], and Terzaghi (given in O'Carroll, 2005) [4]. Observed face pressures indicated that the closest results were obtained by Jancsecz and Steiner's method. However, when the geological conditions of the alignment were analyzed, Jancsecz and Steiner's method could not indicate the variation of the geology. The method developed by

Anagnostou and Kovari was the most suitable method indicating geological changes, and the collapse experienced was the proof of this.

It has been detected that operational torque values increased up to a maximum of 3 MNm for Kartal (Limestone) Formation and decreased approximately to a minimum of 1 MNm for alluvium.

Operational thrust values increased up to a maximum of 18000 kN for Kartal (Limestone) Formation and decreased approximately to a minimum of 9000 kN for alluvium. Thrust values were also high in dykes but lower compared to Kartal Limestone. Several laser cracks were observed due to high thrust forces applied on the mixed face conditions of Kartal Limestone and alluvium on the disc cutters resulting in replacement stoppages. Also, at the same mixed face area, instantaneous penetration rates decreased to 5 mm/revolution. Instantaneous penetration rates were around 10-12 mm/revolution for alluvium.

REFERENCES

[1]**Guglielmetti V.,** 2007:Mechanized Tunnellingin Urban AreasDesign Methodology and Construction Control

[2] **Barla G. and Pelizza S.,** 2000 : Tbm Tunnelling In Difficult Ground Conditions . Proc. GeoEng2000, Melbourne, 2000

[3] **Maidl B., Schmid L., Ritz W**.,2008: Herrenknecht M., Hardrock Tunnel Boring Machines

[4] **O'Carroll J. B**., 2005: A Guide to Planning, Constructing and Supervising Earth Pressure Balance TBM Tunneling

[5] **Bosse M.,** 2005: Performance of EPB-TBM in Mixed face conditions Msc Thesis University of Alberta

[6] **Ezzeldine Y.O.,** 1995: Design of Tunnels constructed using pressurized shield methods Phd Thesis University of Alberta

[7] **Broere W.,** 2001:Tunnel Face StabilityandNew CPT Applications Phd Thesis Delft Technical University

[8] **Murayama, S., Endo, M., Hashiba, T., Yamamoto, K. and Sasaki, H.(eds**.) 1966: Geotechnical Aspects for the Excavating Performance of the Shield Machines. The 21st annual lecture in meeting of Japan Society of Civil Engineers

[9] **Broms, B.B. and Bennmark, H.,** 1967: Stability of Clay at Vertical Opening. ASCE Journal of the Soil Mechanics and Foundations Division. SM1 , pp.71–94.

[10] **Atkinson, J.H. and Potts, D.M**. 1977: Stability of a Shallow Circular Tunnel in Cohesionless Soil.Géotechnique 27(2) (1977), pp.203–215.

[11] **Davis, E.H., Gunn, M.J., Mair, R.J. and Seneviratne, H.N.** 1980: The Stability of Shallow Tunnels and Underground Openings in Cohesive Material. Géotechnique, 30(4), (1980), pp.397–416.

[12] **Mohkam, M. and Wong, Y.W.(eds)** 1989: Three Dimensional Stability Analysis of the Tunnel FaceUnder Fluid Pressure. Numerical Methods in Geomechanics. Rotterdam, Balkema, 1989,pp.2271–2278

[13] **Leca, E. and Dormieux, L** 1990.: Upper and Lower Bound Solutions for the Face Stability of ShallowCircular Tunnel in Frictional Material. Géotechnique 40(4) (1990), pp.581–606.

[14] **Jancsecz, S. and Steiner, W. ,** 1994: Face Support for a Large Mix-Shield in Heterogeneous Ground Conditions. Tunnelling 94, London, 1994.

[15] **Anagnostou, G. and Kovári, K.** 1994: The Face Stability of Slurry-Shield-Driven Tunnels. Tunn.Undergr. Sp. Tech. 9(2) (1994), pp.165–174.

[16] **Anagnostou, G. and Kovári, K.** 1996: Face Stability in Slurry and EPB Shield Tunnelling. In: R.J. Mairand R.N. Taylor (eds): Geotechnical aspects of underground construction in soft ground. Int.Symp. Balkema, London, 1996, pp.453–458.

[17] **Anagnostou, G. and Kovári, K.** 1996: Face Stability Conditions with Earth-Pressure-Balanced Shields.Tunn. Undergr. Sp. Tech. 11(2) (1996), pp.165–173.

[18] **Yuksel, A., Sozak, N.N., Gulle, G**., 2005: EngineeringGeology report prepered for Kadikoy-Kartal MetroProject: KK-GE-TR-GN-004, Anadoluray JointVenture, Istanbul.

[19] **Bilgin, N., Copur, H., Balci, C., Feridunoglu, C.,Tumac, D.,** 2006. Full scale rock cutting tests for TBM performanse prediction for Kadikoy-Kartal metro tunells. ITU Faculty of Mines, Technological Report Prepared for Anadoluray Joint Venture, Istanbul.

[20] **Sözak, N.,** 2008.: “Hasanpaşa-Kavşağı-Otosan Arazisi Arasındaki Güzergahta EPB-TBM tünelleri KazıOrtamları, Davranışları ve Önlemler.” Anadoluray,Teknik Doküman: KK-GE-TR-GN-014 (R00).

[21] **Bilgin, N., Copur, H.,** 2009:Kadıköy-Kozyatağı Metro Hattı Hasanpaşa Kavşağı–Otosan Arazisi Güzergahının (3+700–4+200 km)EPB-TBM’LER ile Kazılmasında Karşılaşılabilecek Sorunlar ve Çözüm Önerileri

[22] **Kim SH, Jeong GH, Park IJ** 2006: Evaluation of Shield Tunnel Face Stability in Soft Ground International Symposium on Underground Excavation and Tunnelling 2-4 February 2006, Bangkok, Thailand

[23] **Maidl, U.,** 2003. “Geotechnical and mechanical interactions using the earth-pressure balanced shield technology in difficult mixed face and hard rock conditions. Proceedings of Rapid Excavation and Tunnelling Conference, pp. 483-493.

[24] **Steingrímsson J.H., E. Grøv, B. Nilsen**. 2002. The significance of mixed-face conditions for TBM performance. World tunnelling, annual Technical Review. 435–441.

[25] **Carranza and Torres** 2004.Technical report, Geodata Spa. Italy.

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