MECHANIZED TUNNELLING IN URBAN AREAS

design methodology and construction control

Vittorio Guglielmetti, Piergiorgio Grasso, Ashraf Mahtab & Shulin Xu
EDITORS

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Edited by
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Breakthrough of the Herrenknecht TBM at 'Sao Bento Station', under construction. Porto Metro Line 'S' – Porto, Portugal.

Photograph by Piergiorgio Grasso, 2003.

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The entrance to the 'Sao Bento Station' of the Porto Metro, after completion. Porto Metro Line 'S' – Porto, Portugal.

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Preface

Fresh air to the conception of tunnels in urban areas

Sebastiano Pelizza

The world of underground engineering and construction has acquired a wide-ranging and high-level experience on tunnel construction with Tunnel Boring Machines (TBM), thanks to the remarkable increase in the number of tunnels that are becoming longer, going deeper, and growing larger in diameter; in other words, becoming more difficult to realize.

In urban areas, the acquired consciousness of preservation and care for the anthropogenic environment, accompanied by the improvement in the quality of life, has raised the level of difficulty and challenge in respecting the constraints deriving from human presence and, therefore, the necessity for a technological and intellectual approach to respond appropriately to these constraints.

This recent, invaluable experience gained from a series of accidents in urban tunnelling worldwide has made us aware that the TBM is simply not a fully mechanized tool integrating the various operations of the conventional excavation method for excavating more rapidly, and overcoming all (or almost all) the well-known problems and uncertainties. Instead, the TBM and the tunnel to be excavated, constitute a delicate and sensitive, unitary system, which should be managed with a new approach, rationally organized and scientifically sustained, in a unified context of research and design of the tunnel, the machine, and the environment.

In particular, all the principal risk factors are found to be associated with tunnels in densely populated urban areas, including the properties and services subject to risk, poor geotechnical conditions of the ground, presence of and consequent interference with water table, and the small overburden with respect to the excavation diameter.

The focus of this book is exactly on the problems of urban areas. Its authors want to analyze and propose not only the machines, but also, above all, the new special techniques for controlling the proper operation of machines and, consequently, the ground water drainage, the stability of the excavation face, and the resulting tunnel profile, for the purpose of minimizing the risks of subsidence. Therefore, a substantial portion of the book is dedicated to identify, evaluate, and manage such risks.

Framed in this particular manner, it seems to me that the book stands up above customary texts, in drawing attention to mechanized construction of tunnels in urban areas as a complex system that needs real or conceived certainties: adequate preliminary investigations for small depths must supply exhaustive information; scientific design that should not leave anything to be invented during construction; reliable and correctly equipped machines to face the foreseen potential emergencies; and planned construction managed by supervisors and technicians with demonstrable qualifications. In this sense we try to supply at Politecnico di Torino a serious contribution of training with the commitment of a Master's course of a year's duration on "Tunnelling and Tunnel Boring Machines".

To sum up, I like this book for many reasons, just a few of which I would like to highlight:

1. It brings fresh air to the conception of tunnels in urban areas, placing in the forefront the fight against the risks, thus supplying a reliable instrument for making rational and transparent choices to the decision-makers, who may be shocked by the many cases of damage and collapse manifested in urban tunnelling history.
2. It is useful for the TBM users and operators who, in facing their duty to make the machine run at its maximum capacity, must acquire the consciousness that the consequent risks should be very well evaluated, anticipated, and minimized. The book is also useful for the students to whom we must try to impart the sense of "scientific humility" (auto-criticism is never enough!) and who must, as quickly as possible, learn the lessons from the available, collective experience.
3. Another important reason for my appreciation of this book emerges from the above two reasons: it is written by experienced technicians who clearly intend to show, through specific examples in which they were directly involved, what the origin was for the manifested risks, how they were approached and overcome, and how these risks could be avoided in the future.

How much more useful is it in our profession to re-analyze the critical situations, rather than taking glory for a piece of work that was well developed without obstacles!

Sebastiano Pelizza
Professor of Tunnelling at the Politecnico di Torino, Italy, and
Past President (1995–1998) of International Tunnelling Association

Preface

A creative application of the principle of Risk Management

Harvey W. Parker

Cities are not sustainable without infrastructure and, in many cases, the best choice for much of this infrastructure will be a tunnel. Accordingly, there is already, and will be in the future, a great demand for tunnels to be constructed in difficult and crowded urban settings. Not only are the constraints that these urban settings pose to tunnel construction quite challenging, but there are also extremely demanding performance requirements for minimal disturbance to the public and to the surrounding utilities, structures and the environment.

Fortunately, the authors have written this book which does an excellent job in describing the special approaches and requirements currently required when designing and constructing tunnels in urban areas. Very little about this important subject is currently available in our literature, except for short articles and conference papers which do not have the space to develop the subject in sufficient detail. This extensive and comprehensive book allows the authors to share their great combined experience and creativity at a level of detail not available elsewhere.

The data and methodology presented by the authors range from guidelines and practical rules of thumb to sophisticated computerized analyses. The authors have unselfishly shared their vast experiences and impressions of future trends in the fields of design, analysis, construction, and management. Thus, this book conveys wisdom of experience while still offering the promise and creativity of a rapidly advancing state-of-the-art.

The points made by the authors are backed up by references and case histories giving the reader practical, common sense examples.

One of the principal themes of the book is that creative application of the principles of Risk Management can, and should be, systematically applied throughout the planning, design, and construction of every project. The authors develop the concept of continuous, intense, and detailed evaluation of risks, which richly interconnects the various phases and tasks of a project together, in not only a realistic but also a practical manner.

The geotechnical uncertainties and the constructability, management, and health and safety issues, as well as risk avoidance and acceptance of residual risk are presented in a practical way based on common sense. Examples and guidelines are given rather than abstract ideas.

Although this book primarily addresses larger-diameter tunnels, the principles and methodology, especially those associated with systematic risk management, can be applied to other tunnel and underground space construction that is required in the urban environment.

Again, the authors are commended for unselfishly sharing their experiences.

Harvey W. Parker
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Executive summary

The need for mechanized excavation of tunnels in urban environments has continuously increased in the last two decades, especially as a result of the global expansion in the number of tunnels being constructed for subways, railway underpasses, and urban highways.

The hazards associated with the tunnel construction in urban areas include poor ground conditions, presence of water table(s) above the tunnel, shallow overburden (often approaching feasibility limits), and ground settlements induced by tunnelling with potential damage to the existing structures and utilities above the tunnel.

Special technologies have been developed to control the stability of the face and roof of the tunnel and to minimize the surface settlements. The application of the technology for achieving the specified goals requires the use of the methodology presented in this book. The methodology is named PAT (Plan for Advance of Tunnel) and is based on the principles of Risk Management illustrated in Figure1.

A Risk Management Plan (RMP) is an essential component of PAT which involves the following sequential steps:

- Risk identification.
- Risk quantification.
- Primary response to the identified risks (mitigation measures, including correct design-construction choices).
- Evaluation of residual risk.
- Predefinition of countermeasures to the residual risks.

The topic of mechanized tunnelling has been addressed in numerous articles, but only in a few books. Furthermore, mechanized tunnelling in "urban areas" as a subtopic has not yet received the due attention that it deserves; This book is intended to fill this gap.

The book is structured starting from the fact that tunnel construction in urban areas is generally associated with high-level risks, which can cause potential damage to structures and/or people. Therefore, such risks must be identified, evaluated, and managed.

In other words, before starting with tunnel design and construction, the first step is to identify all potential hazards related to the excavation process (geology, design, construction) and to evaluate the likelihood of their occurrence and the potential consequences (impacts or damages). The second step is to decide if the level of an

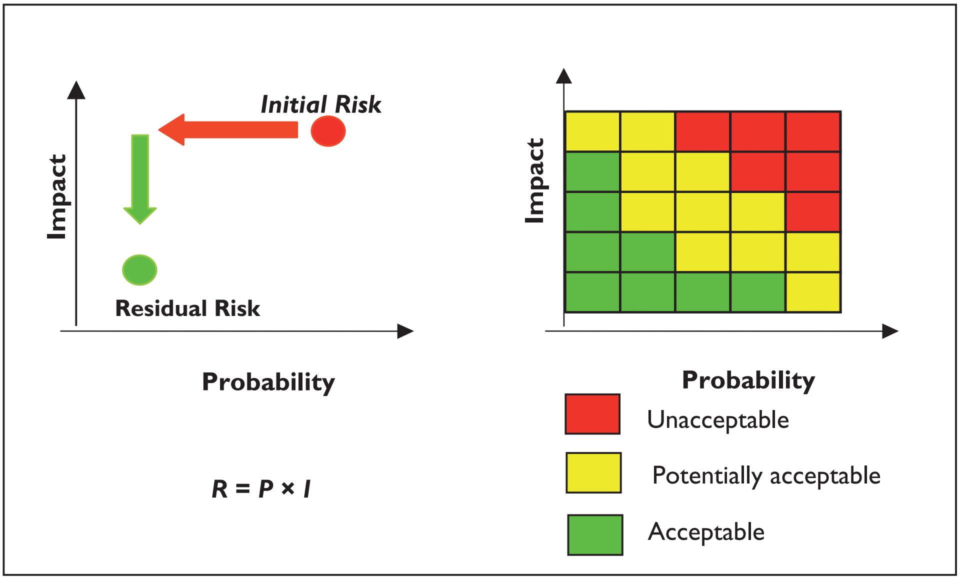

Figure 1 The principles of Risk Management.

identified risk needs application of mitigation measure(s). If mitigations were necessary, the last step would be to design them, for eventual activation during construction.

The application of an RMP demands that the design be developed using probabilistic methods and the resulting design should be checked and, if necessary, optimized during execution using the PAT methodology (Fig. 2). PAT is a "running" method that allows the update of the design and construction-control parameters for the stretches of the tunnel to be built, based on the results derived from the already-constructed stretches.

It is also illustrated in the book that the efficient application of PAT can be facilitated by a real-time monitoring system, implemented on a GIS (Geographic Information System) platform and accessible via the World Wide Web, for sharing information among all Parties involved. All monitoring data, including those from both the PLC (Programmable Logic Controller) of the TBM and from the various instruments installed on the structures, in the ground and on the buildings, are stored in a database according to their location and time of occurrence.

This database represents a kind of "flight recorder" which can help, not only to investigate the causes of any accident after the "plane" has crashed, but also to actively and continuously control and intervene to avoid the "plane" crash.

This book is the fruit of collaboration of about 20 engineers and geologists who have worked exclusively for GEODATA S.p.A., and it is a reflection of the essential activities of a company specialising in geo-engineering, which is without doubt multidisciplinary and requires effective integration of diverse competences and skills to be successful.

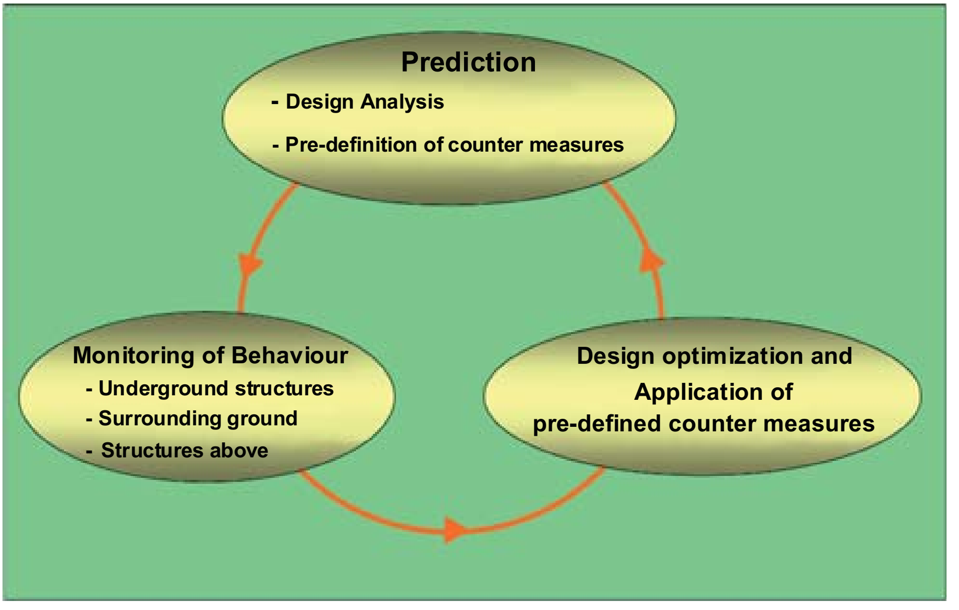

Figure 2 The principles of PAT.

Engineering of underground structures in congested urban environments has to be based on the assumption that practically nothing is certain about the major input parameters: the geotechnical and geomechanical interpretation of the ground behaviour, the assessment of the interaction between the tunnel structure and the surrounding environment, the construction variables and market factors, and the opinion and response of the final users of the infrastructure to be built.

We believe that 'engineering' a tunnel is an 'iterative' activity which, starting from probabilistic bases, must involve: (1) a comparison with the reality, gradually revealed by the construction, and (2) modification of the initial design and consequent adjustment of the design picture to the 'evolving' reality, through a dynamic and continuous design process (implementation, monitoring, checking, and optimization of the design) till the completion of the works, the moment by which the design shall be completed.

It follows that the construction and, especially, the control of the construction process should also be seen as an integral part of tunnel engineering.

Uncertainty and risks are real-world matters, which the modern man has learned to live with, and which the modern designers and contractors should face constantly by a logic of 'analysis' and 'management' of the potential events or hazards that are at the base of the risks.

The goal for the tunnel designer, suggested by this book, is to face the risk, understand it, quantify it, and mitigate it; or, in other words, to manage it through design, monitoring during construction, controlling the construction operations according to best practice, updating the design.

The book consists of 8 Sections, 7 Appendices, and one Annex.

In Section 1, the challenges related to the urban works are shown, highlighting the relevant links to the "design methodology", which is the scope of this book.

Section 2 introduces the philosophy of Risk Management, considered as a must for urban projects; this is followed by the description of the Plan for Advance of Tunnel (PAT).

Section 3 is focused on the project configuration and alignment design, particularly taking into account the environmental requirements and the already existing constraints along the tunnel route, including some particular problems about TBM logistics.

Section 4 deals with the correct choice of excavation methodology in response to the risks identified and evaluated following the RMP. It is stressed that the peculiar characteristics, which the machines should possess in order to work in an urban environment, are unique and many, so that it is convenient to consider them as a class which is associated with the so-called "City Tunnel Boring Machine", or City Machine for short. The correct choice of the right type of machine (complete with its proper equipment) is an essential element to ensure the success of a project. Therefore, in terms of risk analysis, the correct choice of the machine should be considered as one of the primary risk-mitigation measures for controlling the effects of the tunnelling-induced settlements (or a possible instability or collapse of the excavation face).

The subject of Section 5 is the engineering of the tunnel, which is subdivided into four subsections:

- study of the consequences that the tunnel excavation could induce to the buildings and other structures existing above and around the tunnel and the determination of the required counter measures, including the treatment of the ground and the reinforcement of the existing structures;
- design of the excavation face-support pressure, as an essential (but not unique) element to assure the maintaining of the required stability specifications;
- design of the final lining, made of rings of prefabricated concrete segments;
- design of the grouting of the tail void between the extrados of the lining and the excavated tunnel profile.

Section 6 is dedicated to the study of systems for controlling the excavation, using true and proper "secondary counter measures" aimed at further containing the residual risks. Subsection 6.1 is related to the process of implementing PAT. Subsections 6.2–6.3 concentrate on the actions to be taken to prevent dangerous events from happening in the use of Slurry and EPB machines, respectively. Subsection 6.4 is dedicated to the monitoring system, which data shall be integrated and collected in a GIS-WEB platform, giving all parties the access to all the information.

Section 7 is devoted to the subject of Health and Safety, an essential part of the design and construction control activities, which should not be neglected in urban tunnelling, even though in recent years very important advancements have been made in this direction. From the time of the grand borings in the Alps, when the fatal accidents were counted in terms of 10 per kilometre, the count was reduced to the magnitude of 1 per kilometre in the 1980s and nowadays it is minimized to the level of 0.1 per kilometre. Unfortunately, precise statistics on the accidents are not available, but the trend is surely what has just been stated above. However, we should not lower the alarm level because, today, we not only can save the lives of people and prevent the workers in the tunnel from being injured, but also we can try to improve the quality of life for the workers and the environment they have to work in by paying more attention to the safety measures.

The most significant experiences gained by the authors (in recent years) from mechanized tunnelling projects in city environments are presented in Section 8, Case Histories. The various problems of tunnel excavation and support in urban environment are solved by applying and gradually perfecting the most modern techniques as well as PAT, i.e. design and control of the construction operations.

Appendix 1 gives an overview of the machines for tunnel excavation based on the classification of ITA (International Tunnelling Association). Appendix 2 gives a short presentation of the TBM manufacturers in Europe, North America, and Japan. Appendix 3 provides relevant details on the methods for investigating the geological, hydrogeological, and geotechnical parameters, both in-situ and in the laboratory, which are applied in the various stages of development of a project. Appendix 4 gives a summary description of the methods for calculating face-support pressure, which were referenced in Section 5.2. Appendix 5 provides an example of Risk Management Plan, based on the experience gained in the EOLE project in Paris for the construction of the Line E of RER and in the St. Petersburg metro Line 1 project. Both of these projects are summarized in Section 8. Appendix 6 depicts an example of the typical procedure for tunnel excavation using an EPB machine, which is based on the experience from Porto, Torino and Bologna, discussed in Section 8.6. Appendix 7 gives a summary of the mechanized tunnelling projects realised in city environment in Italy.

Finally, we believe that the contractual aspects which tie the 'Constructor' or 'Contractor' with the 'Client' or 'Employer' should assume a great importance in the iterative process of design-construction-control for realizing a tunnel in urban environment. However, since this subject was not in our asset of knowledge, we invited an independent consultant, Dr. Ing. Gianni Alberto Arrigoni, to write a kind of 'Monograph' on this subject. Dr. Arrigoni has extensive experience in managing international tunnelling projects, both as an engineer and as a contract specialist and he graciously accepted our invitation to write this monograph on "Contract and Construction Aspects" of mechanized tunneling in urban areas, which forms the Annex to this book.

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1

Introduction: tunnels in urban areas and the related challenges

1.1 THE OPPORTUNITIES

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 re-distribution 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 (Parker, 2006a). 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 review 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 1.1 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 1.1 The biggest (in 2007) EPB Shield: the 15.2-m diameter Herrenknecht S-300 for Calle 30 project, Madrid.

- The rapidity and industrialisation 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 centres, 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 (Fig. 1.2), 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 (see project details in Section 8.6).

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.

Figure 1.2 Pictures of Kuala Lumpur SMART Project.

1.2 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.
- The accessibility for doing the necessary site investigations can be limited due to a lack of permission or to the occupation of the surface.
- 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, sub-surface, 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 behaviour 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.

1.3 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 (see Section 1.2) 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.

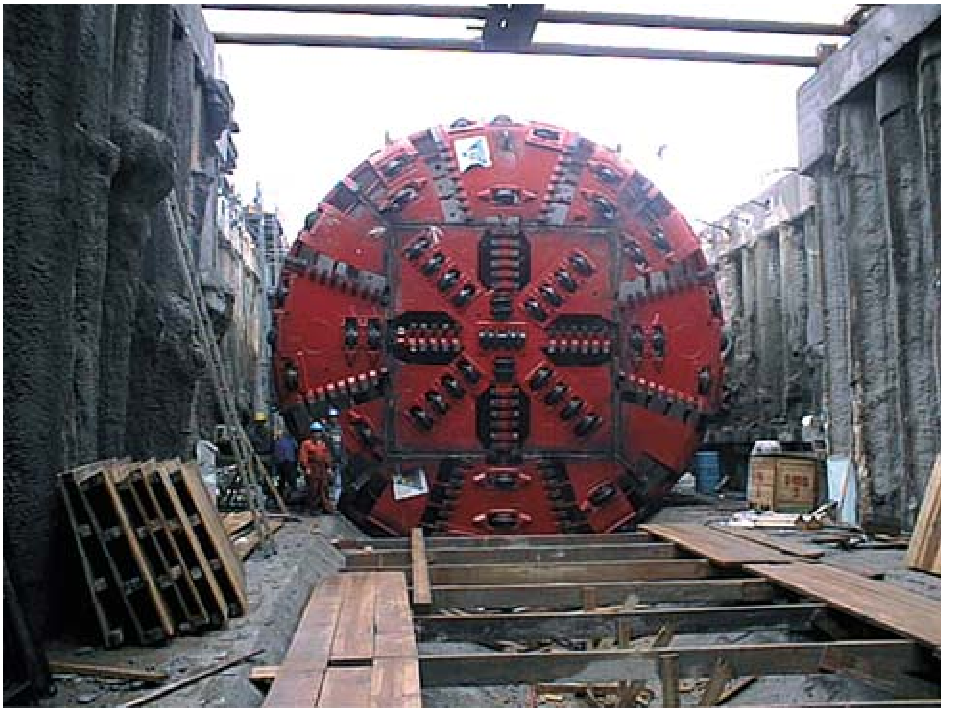


Figure 1.3 One of the Earth Pressure Balance Shields operating in the Porto Metro; the 8.7-m - diameter Herrenknecht TBM.

- 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.

For properly managing the risks in urban tunnelling projects the following key 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 (see Fig. 1.3 and Section 8.2). 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.

Nowadays, within the international tunnelling community there is an agreement that Risk Management is the key to success for all kinds of tunnelling work, especially for urban mechanized tunnelling (Parker, 2006a). In fact, "risk analysis and management" has been a constant theme on the agenda of the ITA's meetings, with guidelines, procedures and models being established for risk identification, analysis, and mitigation as well as for the best practice of risk management (ITA, 2006, ITA, 2004, Grasso *et al*., 2002, Reilly *et al*., 1999).

As shown in Section 8, in the last decade the authors developed and applied in various urban mechanized tunnelling projects the concept of Risk Management Plan (RMP). As already mentioned in the Executive Summary, the implementation of a RMP has been a demonstratively correct approach to success; the approach shall be explained in detail in some of the subsequent sections.

1.4 A BRIEF HISTORY OF MECHANIZED TUNNELLING

1.4.1 The first excavation machine

The tireless wormy shape mollusc, shown in Figure 1.4, with the scientific name of Teredo Navalis, will probably not be considered the precursor of mechanized excavation, but at least it provided the inspiration for this technology.

Until the early 1800s, tunnel building in urban areas was possible applying two different methods:

- Cut and Cover excavation.
- Tunnel excavation using timber frames inside the advancing cavity and immediately lining with masonry.

Those excavation methods were successfully applied in both cohesive and non cohesive ground, also in the presence of limited water seeping through porous ground or fissures, but never really under the phreatic surface.

By observing the feared Teredo Navalis excavating with its tough jaw and covering the hole with its excrement, Sir Marc Isambard Brunel found the inspiration for the technology that later on allowed, for the first time, to build a tunnel underneath the river Thames, in London (Fig. 1.5).

The first idea of tunnelling under a water table was in reality suggested in 1806 by the same Brunel, for the realization of a tunnel under the river Neva in St. Petersburg where in winter, because of the presence of ice blocks from Lake Lagoda, a bridge constructed with piers was seriously damaged year after year. Brunel finally submitted plans for a suspended bridge; only in 1818 did he patent for the first time his invention: the shielded excavating machine.

The opportunity to apply his technology arose in 1825, when the River Thames tunnel underpass started to be excavated. The first excavation attempt was done between 1825 and 1828 using a shield which was found unsuitable and so removed and substituted by a rectangular shield.

Stack (1982) described the rectangular shaped shield which was employed for the second attempt (1835–1843): it was made of cast-iron, composed by 12 compartments

Figure 1.4 The "Teredo Navalis", working on the excavation and the lining of "his tunnel".

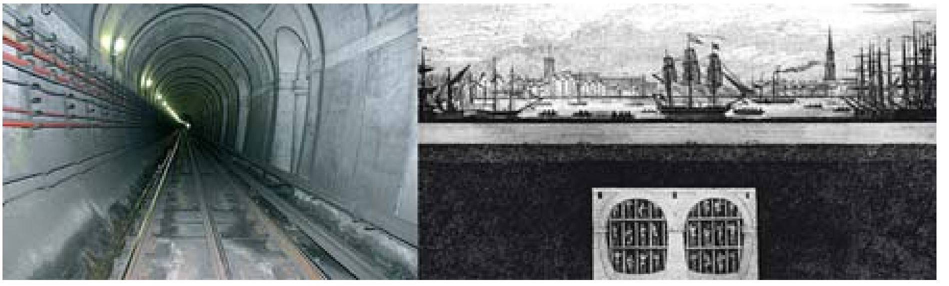


Figure 1.5 Brunel's shield in action on the left under the Thames (Mathewson *et al.*, 2006) which is today still part of the "East London Metro Line" (on the right).

each about 1 m wide, divided into three cells (upper, middle and lower cells). Each compartment was capable of independent movement (Fig. 1.6).

The entire shield was 11.43 m wide, 6.78 m high and 2.74 m long. Chisel shaped stoves or sliders attached to the top and bottom of each compartment slid forward to cut and support the ground immediately ahead of the shield.

Advancement was by means of screw jacks which thrust against the finished masonry of the tunnel. Each compartment supported its part of the tunnel face by means of 14 or 15 horizontal breasting boards, held in position of advance by means of a pair of screw jacks.

The 12 compartments were advanced alternately one at a time; each advancement step was no more than 15 cm. The best performance was 4.3 meters per week.

After the successful application of the Brunel machine, many inventors suggested evolutions and innovations to improve productivity, safety, and capability to face serious water inrush. Amongst them it is worth mentioning:

- S. Dunn (1849) for the first time a patent for a shield advancing as one piece was taken out;
- P.W. Barlow (1864) made the important suggestion that *[...] space as it is left between the earth and the extension of the tunnel may be filled by injecting or running in fluid cement [...].*

It was finally A.E. Beach (1826–1896) and J.H. Greathead (1844–1896) who actually built and used shields incorporating such proposals in the same year (1869), but Beach for the Brodway pneumatic railway tunnel in New York, while Greathead for the New Thames tunnel in England.

In particular J.H. Greathead excavated a new, 402 m long, tunnel underneath the Thames, this time using a circular shield with an external diameter of 2.18 m.

The construction of this tunnel was without particular difficulties, because of the low permeability of the clay involved which guaranteed advancement without the problem of water inflow.

To support the excavation, steel rings were used for the first time instead of timber frames. Greathead's shield became the model for the majority of shields built afterwards (Fig. 1.7).

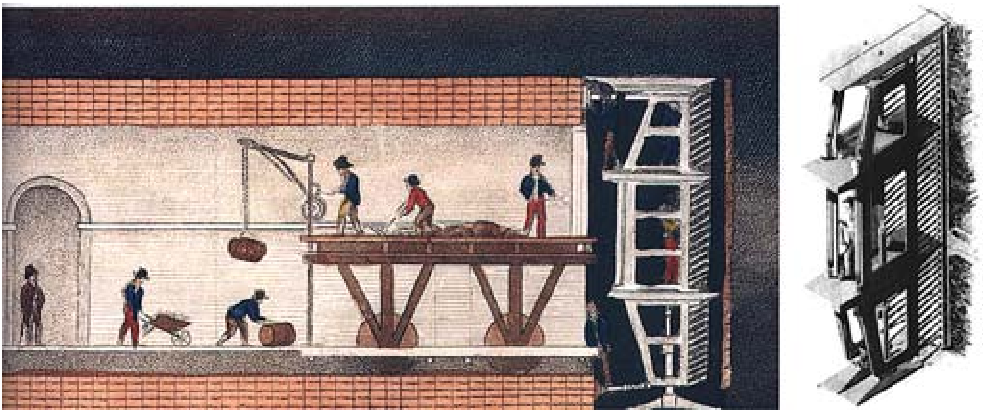


Figure 1.6 Brunel's shield representation system used for the first time for tunnel excavation under River Thames (London 1825–1843), where workers could safely excavate inside each cell (on the right).

Figure 1.7 A picture of the "Greathead's shield", taken in the 1900s, used for tunnel excavation since late 1800s, illustrating the circular shape and the metallic rings lining the excavation walls.

Subsequent improvements of shielded tunnelling machines were focused on two important aspects: the face support and a more industrialized process.

As a result, today's mechanized excavation methods can construct tunnels more rapidly, secure the safety of the workers and minimize the environment impact.

1.4.2 Introduction of compressed air

Despite the success of the Brunel shield application, the problem of controlling water inrush was not solved satisfactorily until the introduction of compressed air.

First successful applications of this face support technique were in Antwerp Dock tunnel (1879) and in the Hudson river tunnel, New York (1880).

In particular, the failed attempt to drive the Hudson river with caisson and compressed air in 1880 (Fig. 1.8) led Sir B. Baker and J.H. Greathead to suggest a combined use of compressed air with the shield technology to support both the face and the tunnel profile (Fig. 1.9). This important improvement made possible to drive successfully 1130 m of tunnel by mid 1891 (when works were interrupted for economic reasons) and many other tunnels in following years.

But improvements were still needed: several important problems were associated with working with compressed air, because the entire tunnel had to be maintained under pressure. Those problems were mainly related to:

- health problems for the workers, because they had to move frequently and rapidly back and forth between the front tunnel section under pressure and the rear tunnel section under atmospheric pressure;
- the non effective application of this method to large diameter tunnels, because the uniform compressed air pressure is not compatible with non uniform face-support pressure (which increases vertically downward).

Only in the late 1950s an innovative solution was found using a medium of high density to provide face support, which gave birth to the modern Slurry and EPB machines.

1.4.3 The first mechanized excavation examples

For many years excavation and muck removal were performed manually using picks and shovels, rendering the tunnel construction an unsafe and slow process. After many years of focused efforts by many engineers, in 1876 a mechanized solution was found enabling tunnel construction using a shield as an industrialized process.

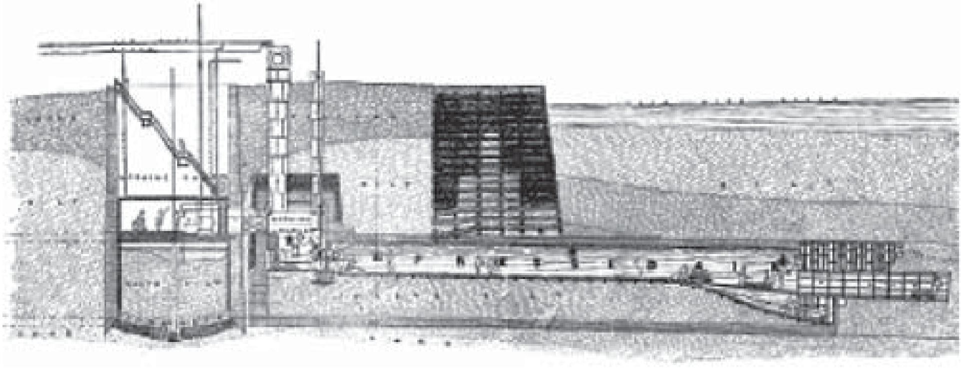


Figure 1.8 Scheme of the compressed air technology used in 1880 for tunnel construction under the Hudson River in New York (Burr, 1885).

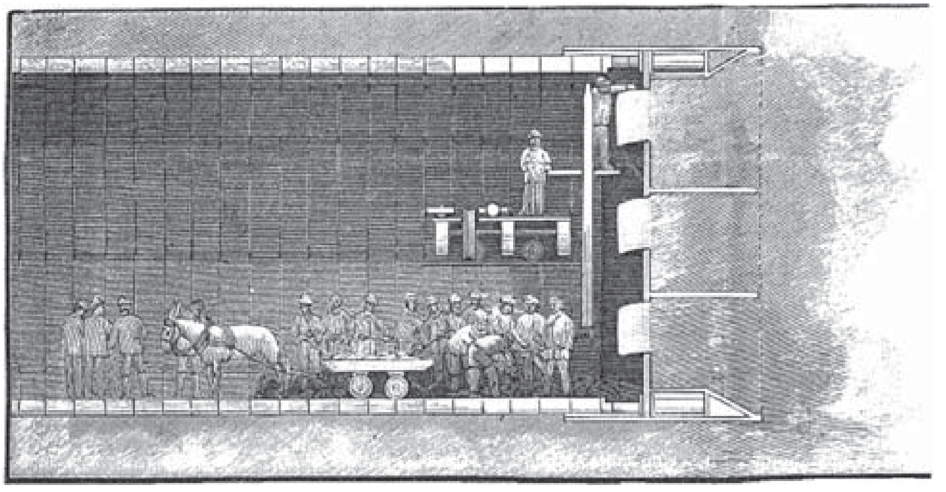


Figure 1.9 Greathead's compressed air shield in action in the silt deposit below the Hudson River in New York. The figure shows the implied circular shape, the steel lining, and the advancement for extrusion of the front (Scientific American, 1980).

The first patent in this regards was due to J. Dickinson and G. Brunton in 1876: a first true mechanized shield, which was later on further improved by F.O. Brown in 1886, J.H. Greathead in 1887–1889 and J.J. Robins in 1893.

Finally, the first mechanized machine was realized and used by J. Price in 1896–1897, for the excavation of the Central London Railway Line (Fig. 1.10).

According to Stack (1982), the Price machine had a cutterhead consisting of four radiating arms on which cutters or scrapers were mounted to dig and collect the loosened material. The scrapers served to lift up the muck allowing it to slide down by gravity along a chute to the waiting skips.

Successive improvement made the machine more reliable and more effective, reaching an advance rate of 55 m/week (obtained in the Charing Cross and Hampstead railway line).

1.4.4 The modern evolution

Starting from the prototype of Mechanized Shield by Greathead, the evolution of this kind of tunnelling machines went very quickly to the sophisticated current types of TBMs, following two principal roads: one for rock and the other for soft-ground.

Initially, machines for rock were only *open type*, i.e. without shield and with tunnel-temporary-support practically the same as used in the conventional tunnelling method. Then, to better face heterogeneous conditions, a shield was added to some rock machines, initially only a *single shield* (requiring the use of segmental lining to provide the thrust for advance) and, later on, an additional shield was added, i.e. the birth of the so-called *double shield* (allowing advance either with or without installing segmental lining, depending on the rock conditions to be excavated).

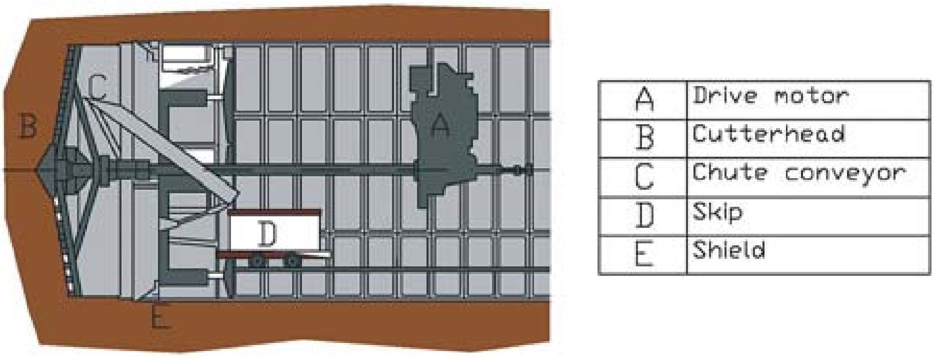


Figure 1.10 Price's shield developed from the prototype of Greathead's Shield, with the first cutterhead mechanically driven by an electric motor.

All three types of machines are currently in use and their choice naturally depends on the ground conditions along a given tunnel route.

The machines for soft-ground were historically *shielded machines*; they evolved rapidly to the type of machines capable of providing also an active support to the excavation face, in order to better control and reduce to a minimum level the risks of having both excessive surface settlements and face collapses.

A short description of all types of TBMs in current use is given in Appendix 1, following the classification scheme of the International Tunnelling Association (ITA).

For some special types of modern machines, it has become very difficult to classify them in a distinct category of machines: these special machines are often conceived and constructed for excavating in very sensitive environments and heterogeneous geological conditions. In fact, this category of TBMs are equipped both with the facilities of a soft-ground machine to control settlements and collapse risks, and also with the necessary tools to excavate in rock. A first example of this kind of special machine was the hydroshield used to excavate the EOLE tunnel in Paris (see Section 8.1) going through sands, marls, and limestones. Another important example was the EPB machines used in Porto Metro (see Section 8.2), which excavated ground ranging from residual soils to fractured fresh granite of Porto.

The machines described in this book are substantially those special TBMs which we call "city machines", because they are especially conceived for use in sensitive urban environment. The fundamental requirements for this category of machines are described in Section 4.

1.5 THE SCOPE OF THIS BOOK

The short history of mechanized excavation presented in Section 1.4 has demonstrated the human anxiety and need to search for excavation tools that are more and more suitable for difficult situations and provide, at the same time, increasingly higher levels of safety. In fact, after two centuries from the first idea of Lord Brunel to underpass the Neva river in St. Petersburg (1806) and 130 years from the first prototype of a slurry shield invented by Sir Greathead (1874), today we are still researching for a "perfect" machine, even though we know subconsciously that we will never find it.

Nevertheless, in this apparently hopeless search for perfection, some important achievements have constantly been made in terms of the safety, the speed, and the cost of the excavation. Furthermore, with each achievement accompanied by a new practical experience, the "feasibility limit" of the technology of yesterday is raised or moved a step forward by the technology of today, resulting in machines of larger diameters and enabling excavations in more and more difficult conditions.

It can be stated that today many projects exercising great vision can be constructed and operated because of recent significant advances in technology (Parker, 2006a). Accordingly, planning of tunnels and underground space can now be bold and visionary because new technology will develop during and after the planning stages that will positively affect the feasibility of the project. Technology is now developing at such a fast rate, that planners and decision makers have great opportunities and challenges to ensure that their planned tunnel or underground space will be at, or exceed, the state-of-the-art at the time their underground facility will be constructed.

The great benefits of the encouraging achievements have shown that not only we should not stop the search for the "perfect" machine, but also there is a constant need for both applied and basic research in the field of mechanized tunnelling, in general. Therefore, one of the purposes for compiling the available technological information and putting forward the concept of a "city machine" in this book is to stimulate more creative thinking regarding the current and future needs of research, in the light of sustainable development of urban environment while tapping the resource of underground space.

As explained in Sections 2 and 3, successful tunnelling in the cities requires a correct understanding of the urban environment which is made up of dense infrastructures on the surface, above-surface and subsurface utilities, and man-made ground as well as the natural geological medium. While the first three components may be, to a certain extent, documented and can be investigated with relative ease, the last element, i.e. the natural ground, represents the most difficult element to be understood. Due to the diverse origin and history of the ground found in different city environments, no two natural deposits are exactly alike with regard to their physical properties and behaviour. Indeed, as pointed out wisely by Peck (1969), the engineering properties cannot be specified, they can only be investigated, determined, and coped with under the physical conditions to which they may be subjected, whether in foundations, excavations (tunnels or trenches), or other engineering works. No two jobs are exactly alike. Yet the designer must design and oversee the construction of a project that serves its purpose safely and economically. This leads to other important objectives of compiling this book, i.e. to let the potential owners or clients of future urban tunnelling projects know what is the correct approach, methodology, scope, etc. for an urban tunnelling project, providing them (where appropriate) with checklists, so they can expect and ask for the best engineering services from the market. Consequently, another principal purpose of this book is to give a comprehensive review of the state-of-the-art of the mechanized excavation technology at the beginning of the third millennium and indicate particularly a methodology for the design and construction-control of mechanized tunnelling in city environments.

As previously emphasized, when analyzing the challenges faced by anybody wanting to construct a tunnel in a city, the only correct approach to deal with this type of problems is through a rigorous and complete analysis of the risks involved. It was also anticipated that the choice of the excavation method should necessarily be directed towards shielded TBMs with pressurized face support, such that the selected method should constitute the "primary counter measure" to the major risks identified in the project. It shall also be shown later that only through implementing a rigorous monitoring plan and control mechanism of the tunnel construction process, by means of a series of operative procedures ("secondary counter measures"), it will be possible to assure the success of the project.

All these themes will be treated in detail in the subsequent sections of this book, to arrive at the conclusion that the choice of the excavation methods, the monitoring, and the control of the execution, aimed at eventually modifying and/or optimizing the design, are all the elements of a wide process which should be managed in a unitary way. Figure 1.11 gives a flow chart of the key elements of a mandatory approach for tunnel construction in urban areas.

One may think that this approach is another version of the old "design as you go" method, which maintains (correctly) that the design has to be flexible enough for changes to be made during or subsequent to construction of remedial works. The concept is clearly valid for all kinds of large infrastructure projects, but it will especially find application in underground construction projects or geotechnical engineering works in general, where the uncertainty regarding the input data is generally high and is mostly related to the poor knowledge of the characteristics of the ground to be excavated or, in any case, to be "treated".

In the presence of uncertainties, and of possible unforeseen conditions, the Designer may be induced to be over cautious, taking into examination the worst cases or the poorest geotechnical parameter values (if not combining the worst of both!), producing thus a kind of "worst case design" generalized for the entire project, while the real necessity for such a worst-envelope case may be limited to just some parts of the project.

Similarly, it is possible to make a comparison between the advocated approach and the so-called "observational method", code-named by Ralph Peck in the distant 1969, the latter being opposite to the "predefined design method".

It is certainly useful to recall the definition of the observational method given by CIRIA (1997) as "...a continuous, managed and integrated process of design, construction control, monitoring and review which enables previously-designed modifications to be incorporated during or after construction as appropriate. All these aspects have to be demonstrably robust. The objective is to achieve greater overall economy without compromising safety" ("Meeting Report", Ground Engineering, May 1998).

It is also interesting to note the following disadvantages of the "observational method" pointed out by the members of the CIRIA study team (which include Sir Alan Muir Wood):

- Absolute planning is not possible.
- Restriction on method of construction.
- Need for good control system.

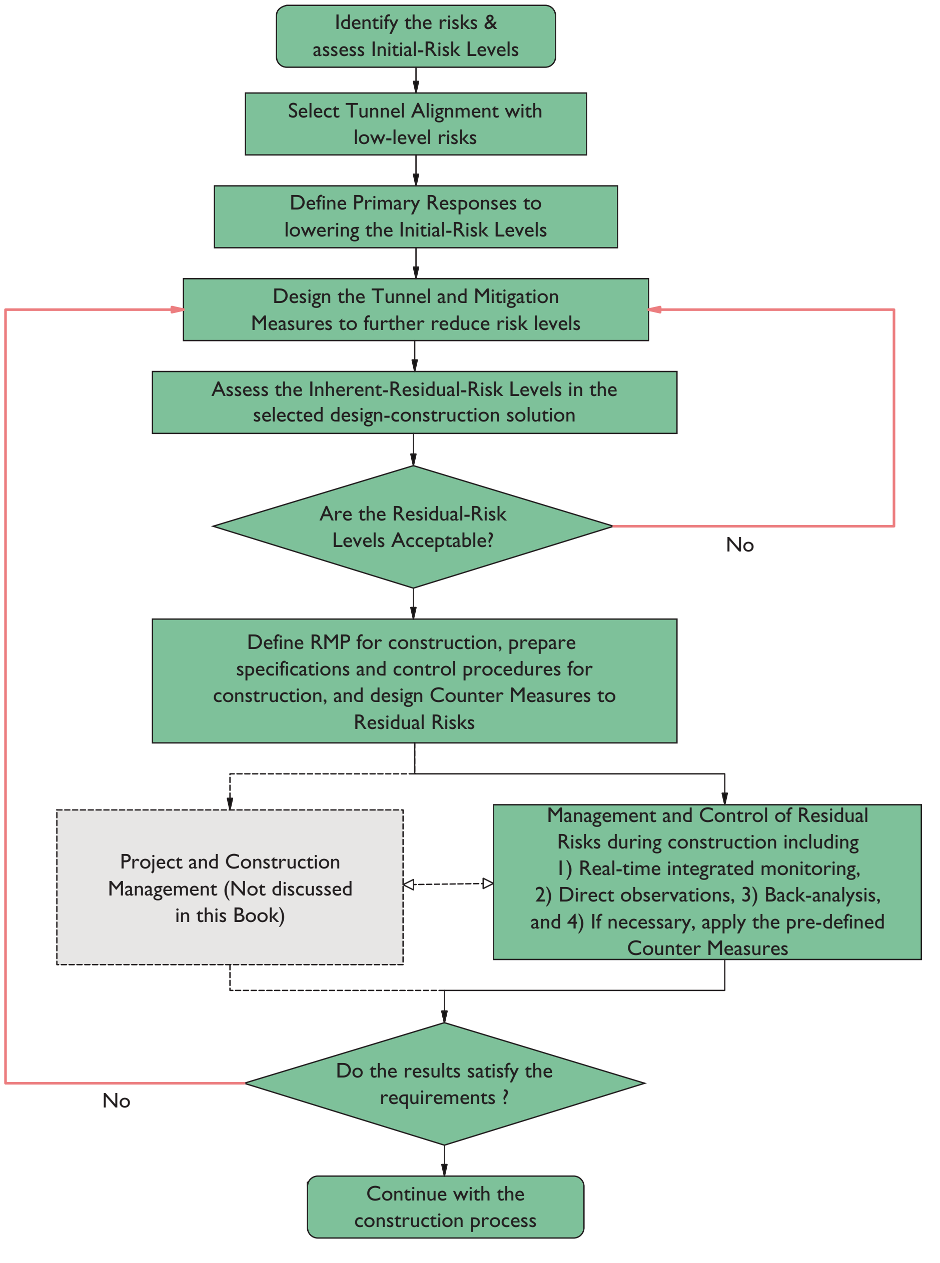


Figure 1.11 Key elements of a mandatory approach for tunnelling construction in urban areas.

- Sophisticated level of engineering and construction management.
- Ability to handle risk is mandatory.
- Ability to live with uncertainty.
- Only limited use on small nonlinear multidiscipline sites.

The principles enunciated in the earlier paragraphs are fully respected by the design method based on Plan for Advance of Tunnel, PAT, presented in Section 2.7, with a rational use of the probabilistic criteria and the elimination, at the same time, of most of the limitations of the observational method, thanks also to the modern techniques of probabilistic analysis, monitoring and control, and "Risk Management" available today. In fact, with the PAT methodology the above list can be reconsidered as follows:

- "Planning and Cost estimation" are not only possible, but also necessary for making informed/conscious choices, yielding more reliable results than those given by the conventional deterministic design, by applying tools like DAT (Decision Aids in Tunnelling, see Sections 3 and 5 for details).
- The "Restriction on method of construction" shall no longer exist, thanks to the modern techniques of mechanized excavation, taking into consideration that sometimes it is obligatory to use certain methods, for example, excavating in "closed mode", as a response to the results of the risk analysis (see Section 4).
- The subsequent "Need for good control System", "Sophisticated level of engineering and construction management", "Ability to handle risk mandatory", "Ability to live with uncertainty", have all become indispensable requirements for success of a project in urban environment and, after all, their presence is the only possibility for the success of the project in terms of respecting the construction time and cost (see Sections 5 and 6).
- Not "Only limited use", but "No limits to the use" of this methodology, indeed, the larger and the more complex a project is, the more it lends itself to be managed using the advocated methodology (see Section 8 "Case histories"), to assure the execution of the tunnelling works without causing any damages to the surroundings.

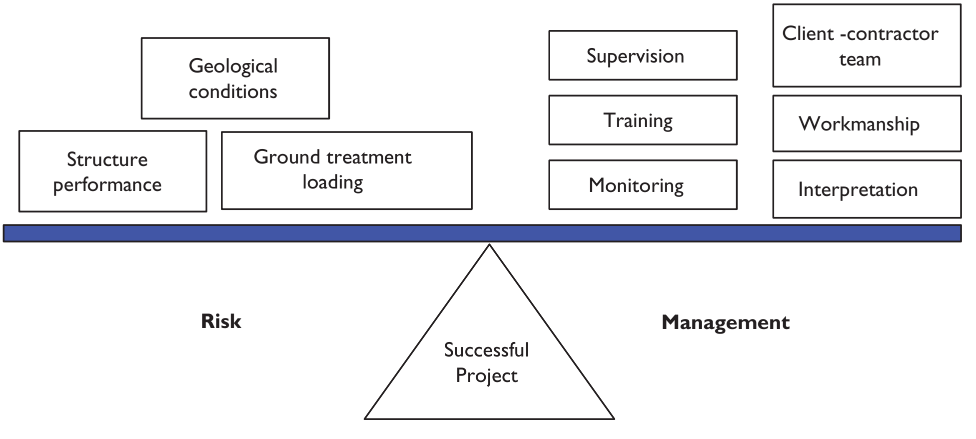


Figure 1.12 "Balance" of Risk Management, representing the precarious equilibrium of "Observational Method" (Ground Engineering, May 1998).

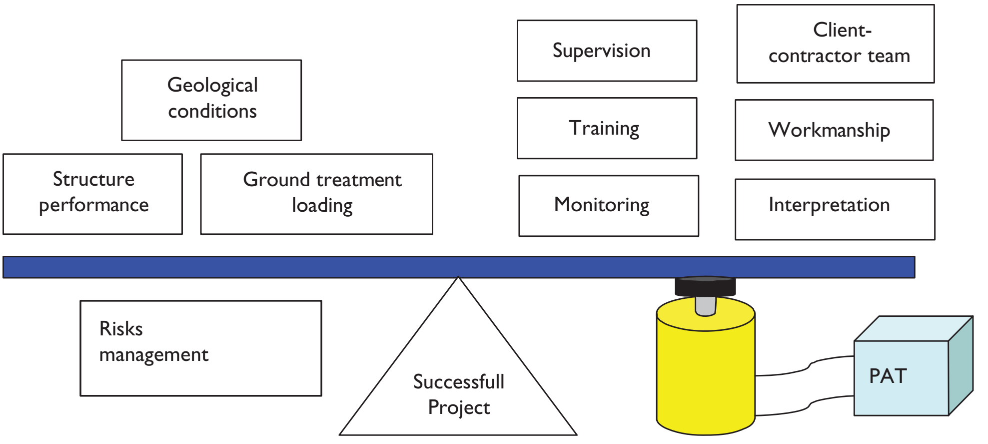


Figure 1.13 The "Active Stabilization" of the process, by means of the "Plan for Advance of Tunnel" (PAT).

This book intends to demonstrate what has been introduced in this section and is illustrated in Figures 1.12–1.13, showing how the use of the control techniques and the continuous updating of the design-construction protocol called PAT could help to "stabilize" actively the equilibrium, otherwise known as unstable by the observational method.

2

Initial risks: definition, analysis and management

Formal Risk Management has become an important tool in many technical fields and is being more widely accepted by the tunnel and underground industry. It is now (2007) becoming more common for underground projects to *systematically* and *continually* conduct formal risk management evaluations at all stages of planning, design, construction, and operation of underground projects.

Indeed, guidelines for systematic risk management published by the International Tunnelling Association (ITA, 2005) have become a standard for the industry. The insurance industry, especially the re-insurance industry, is very actively promoting risk management at all stages of a project in order to minimize insurance losses. An International Code of Practice, which follows the ITA guidelines closely, has been published by the International Tunnelling Insurance Group (ITIG, 2006). This international code was based on an earlier Code developed by the British Tunnelling Society (BTS, 2003). Accordingly, the practices of risk management described in this Section are being endorsed and enforced by a worldwide increasing number of projects.

In simple terms, the risk management approach consists in identifying and listing the potential hazards associated with the tunnelling activities, assigning a probability of occurrence to each hazard, and allocating an index of severity to the consequence. The next step involves a definition of the measures to reduce the probability of occurrence of an event and to reduce the severity of the consequence (the so called "mitigation measures"). An example of the use of the Risk Management Plan, RMP, is provided in Appendix 5.

In practice, the degree of risk associated with a probability-impact pair is rated, and the rate can be quantified as a product of the probability (percent) and the index of severity (or impact) as a percentage of the maximum conventional value, thus defining the "initial risk level".

In cases where the initial risk level is not acceptable, the relevant mitigating measures should be identified and designed. After application of the mitigation measures, an analysis should be performed to reassess the remaining risk level, obtaining an updated risk level, which is called the "residual risk level" and which should be examined for acceptance as the maximum risk level that is to be confronted with its "global cost", necessary for reducing or completely eliminating the risk itself. Figure 2.1, which a simplified version of Figure 2.2(b), illustrates the relationship between the initial and residual risks.

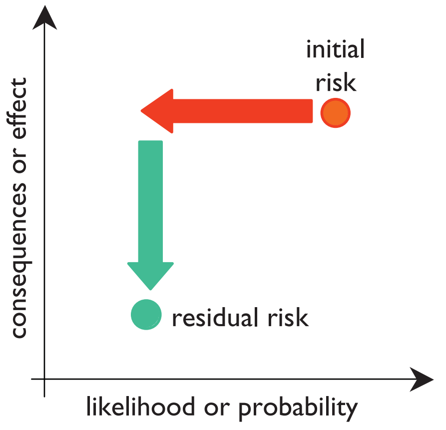


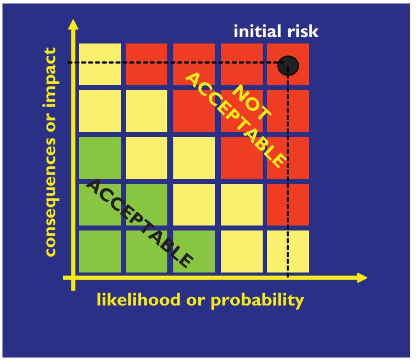
Figure 2.1 The risks level definition.

2.1 BASIC DEFINITIONS

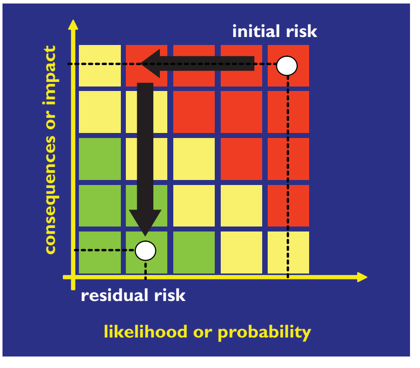
One of the principles of the Risk Management Plan, RMP, is a clear definition of the basic terms to avoid misunderstanding (Chiriotti *et al*., 2003). The various terms used in this section are defined below:

- "Hazard" is an event which may translate into a situation that has a potential to cause damage. To each hazard are associated a probability (or likelihood) of occurrence, P, and an impact (or consequence, or severity), I, in terms of safety, time, and cost.
- The risk, R, associated with an identified hazard is defined as the product R = PI, and is called "initial risk" (Fig. 2.2).
- The "project-based risk acceptability" is a set of criteria for defining if an initial risk in a certain context can be assumed or has to be reduced (through specific mitigation measures) at the design and/or construction phase.
- The "mitigation measures" consist of a set of predefined measures to be systematically implemented at various stages of a project in order to reduce each single, unacceptable initial risk, with respect to the acceptability criteria, by acting on its probability and/or its impact.
- The risk remaining after the implementation of the mitigation measures is called 'residual risk' (Figs. 2.1, 2.2b). Residual risks refer to the acceptable risk levels.
- The "key-parameters" are those elements on which the residual risks depend or by which the residual risks can be controlled.
- The "countermeasures" are those actions, defined at the design stage, which will be activated during construction according to predefined triggering criteria, should the key-parameters reach predefined thresholds.

Hazards are something we can do little about, except identifying them. The risks they pose can be (and should be!) reduced.



(a) Indication of acceptable and not acceptable ranges of risk



(b) Identification of residual risk

Figure 2.2 Initial and Residual risk values.

2.2 SOURCES OF INITIAL RISK IN MECHANIZED URBAN TUNNELLING

The main sources of risk addressed in this book, and associated with urban mechanized tunnelling projects, are related to geology and hydrogeology, design, and construction (Chiriotti *et al*., 2003).

- Geology and hydrogeology related risks could depend on:
  - limited investigations at design and/or construction phase (see Appendix 3 for details of geotechnical investigations);
  - lack of site accessibility;
  - inappropriate on site and/or laboratory tests;
  - insufficient understanding of rock mass/soil behaviour;
  - insufficient understanding of rock mass/soil response to tunnelling;
  - insufficient understanding of the peculiar mechanisms of ground failure;

  - lack of systematic face-mapping during construction,
  - lack of validation/update of the geological and hydrogeological model during construction, and
  - local conditions being different from those foreseen for the design.

- Design related risks could depend on:

  - insufficient experience of the Designer;
  - incomplete analysis of the potential risk scenarios;
  - incomplete evaluation of loading conditions acting on the lining;
  - low or difficult constructability of proposed solutions;
  - lack of design flexibility to adapt to the actual ground conditions;
  - inadequate method for predicting settlements and assessing potential damages to existing buildings/utilities;
  - inadequate definition of the operational ranges of the TBM's key parameters;
  - inadequate monitoring system and/or frequency of readings;
  - lack of predefined threshold limits for the monitored parameter;
  - poor or missing definition of counter measures, and;
  - poor or missing definition of triggering criteria to activate the countermeasures.

- Construction related risks could depend on:

  - inappropriate choice of the construction method;
  - poor management of the 'learning curve' period;
  - lack of Contractor experience;
  - lack of training of the personnel;
  - inadequate procedures;
  - incompatibility of the TBM with the ground;
  - major mechanical failures;
  - inadequate logistics;
  - inadequate face pressure;
  - inadequate injection of the tail void;
  - lack of TBM's parameter controls and/or review and interpretation;
  - insufficient probing ahead of the face;
  - occurrence of instabilities;
  - non-ideal performance or behaviour of the shield machine itself, and;
  - deviation of the actual ground-machine system behaviour from the theoretical one.

The above list is not exhaustive, but gives an idea of the complexity of the variables to be taken into account and draws attention to the need of working by comprehensive check-lists as the first step to address risk management consistently.

2.3 ANALYSIS AND MANAGEMENT OF RISKS: THE RISK MANAGEMENT PLAN

Urban mechanized tunnelling can be associated with a series of risks and the principal risks generally derive from the uncertainties and hazards associated with the geological, hydrogeological, and construction conditions or parameters, plus the political and/or public opinion constraints that require special attention. Materialisation of these risks can have negative impacts on the project performance with respect to time, cost, safety, and environmental aspects.

The Risk Management Plan (RMP) is a robust and transparent risk-management methodology composed of clearly identified steps and tools for managing such risks. The objective of implementing a RMP for a project is to ensure that all risks are reduced to acceptable levels and managed most efficiently. An RMP should be established based on four essential principles (Grasso *et al.*, 2002a; Chiriotti *et al.*, 2003):

- Risk Identification:
  - define project objectives and requirements;
  - establish the tolerance of the Owner to the risk, both for the degree of uncertainty and for the level of risk assumption;
  - characterisation of a Project Reference Scenario and identification of risks through the preparation of a Risk Register (i.e. a complete list of potential hazards and related initial risks) covering all the project disciplines and phases.
- Risk Quantification:
  - for each identified hazard, the potential causes are specified and the risk is evaluated through an assessment of its probability of occurrence and its impact on the project,
  - a preliminary estimate of the Project vulnerability to different types of risks is achieved if qualitative evaluation methods are used (e.g. engineering judgment), while a more reliable estimate can be provided if quantitative methods, such as probabilistic analyses, are used to define both P and I;
  - an order of priority is assigned to the identified risks and a selection is made of those risks which need to be considered later on and are not acceptable.
- Risk Response Development:
  - if a risk is unavoidable, it has to be mitigated by identifying a list of response actions: a design approach and/or a construction technique and/or an installation method to reduce the initial risk;
  - assuming that the mitigation measures have been implemented, the risk has to be re-evaluated in order to quantify the residual risk, taking into account the fact that, after the introduction of the mitigation measures, the responsibility for managing the residual risk may be changed;
  - systematically communicate and/or further reduce the residual risks.
- Risk Response Monitoring:
  - make sure that construction/installation procedures are in place for executing the works in accordance with the strategies identified, at the design stage, to reduce the initial risk;
  - design an efficient Plan of Controls to manage residual risks during construction, installation and testing; this implies that key parameters/indicators to control quality, safety, and progress of the works have to be

identified and relevant monitoring procedures (i.e. instrument type and location, frequency of the readings, alert and alarm thresholds, etc.) have to be put in place;
- design a robust Plan of Countermeasures to be implemented during construction if alarm thresholds are exceeded. For extremely critical situations, an Emergency Plan also needs to be prepared.

The Logical sequence of RMP components (or steps) is given in Figure 2.3.

The activation of the RMP assures the timely identification and resolution of potential problems. Hence, the RMP should be: implemented as soon as possible; integrated in all phases of the project, from conceptual design to exploitation; extended to every single investigation, design, and construction discipline involved in the Project. The objective is to reduce, to a level as low as reasonably possible, all the risks identified in each phase of the Project life and to implement the preventive measures for reducing risks during construction.

As a logical consequence, the RMP should be intended as a dynamic process that needs to be handled, updated, integrated, and communicated along the entire Project. Hence, ensuring that the Risk Register is periodically and systematically updated is one of the essential features for a successful application of the approach.

The implementation of the risk management should incorporate the different technical perspectives and involve the participation of all the concerned parties: Owner, Project Management, Supervisor, Contractor, Experts and Designer.

Project risks can also depend on how the management of Project interfaces is dealt with. In fact, urban mechanized tunnelling is generally related to huge infrastructure projects such as metros, urban railways, storm-water conveyance tunnels, and service tunnels for utilities, sewage or roads. Consequently, the Project consists of different disciplines that have to converge to deliver the final result in quality, on time, and within budget (i.e. reducing all the risks that are potentially against the achievement of these objectives). A method to successfully ensure this convergence and to facilitate the implementation of the RMP is the use of a '*Group of Permanent Coordination*', GPC, in a Centralised Design Management structure (Grasso *et al.*, 2007) aiming to identify, manage, and coordinate the interfaces between disciplines/activities during both the design and construction, to prevent blockages and problems before they arise, and to find solutions when there are conflicts. The Group of Permanent Coordination, GPC, can be set by the Owner or can be promoted by the Designer. The GPC is in charge of: guaranteeing the coherent and integrated development of the project, leading the onset of the interfaces among the disciplines, and addressing and managing the resolution of risks.

The establishment of an 'Exchange Table' within the GPC is not only a physical place of interface coordination where specialists meet each other, but it is also the expression of a shared working methodology that permeates into the domains of all the Project Actors. The Exchange Table is then the right place to activate and coordinate the RMP among the Actors, for example, by initially compiling the Project Risk Register.

Last but not least, when applying an RMP, the following factors should be kept in mind:

- "No construction project is risk free. Risk can be managed, minimized, shared, transferred, or simply accepted, but it cannot be ignored" (Sir Michael Letham, 1994 also reported in Clayton, 2001).

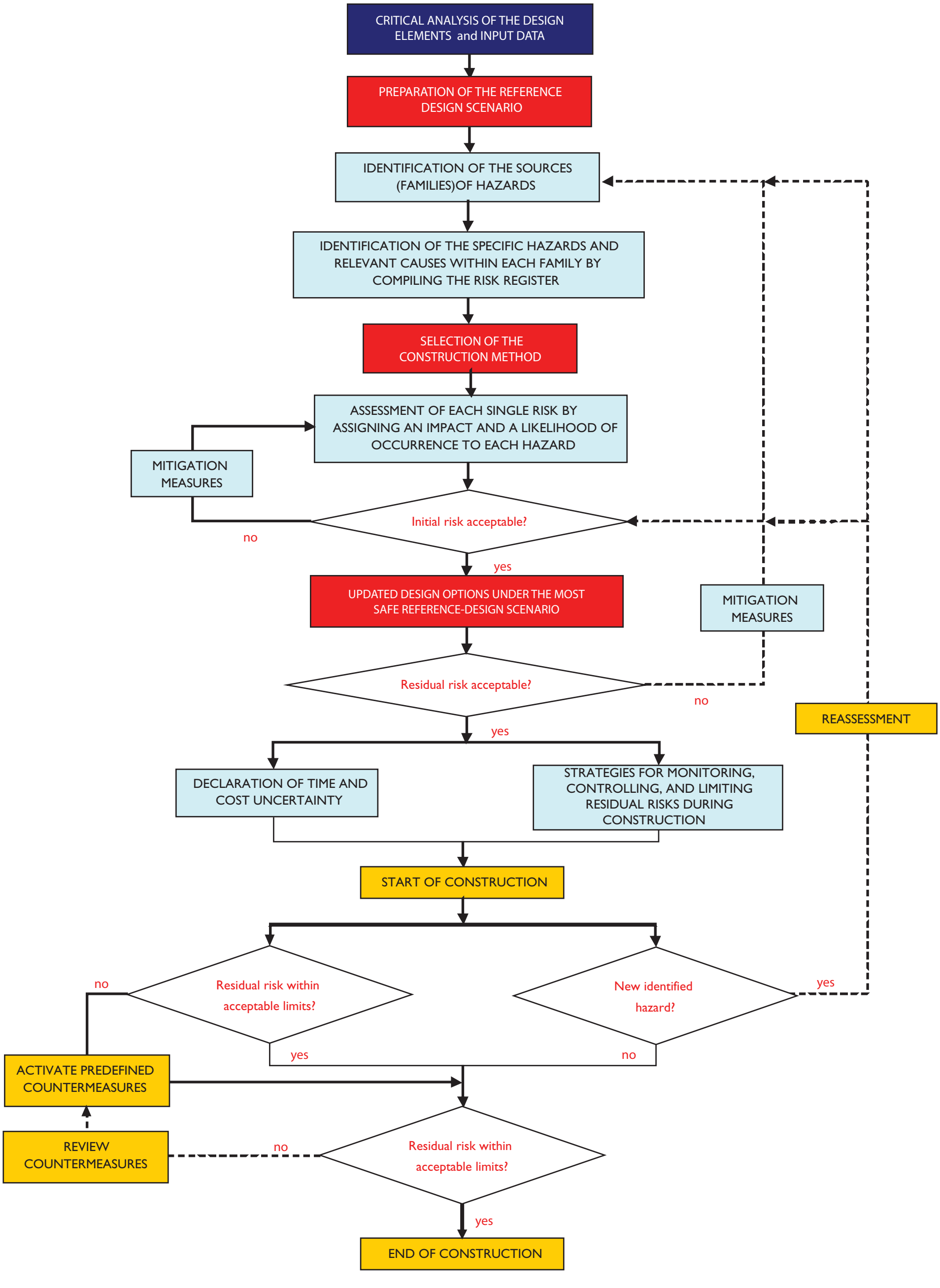


Figure 2.3 Logical sequence of the RMP steps.

- Realistically, not all risks associated with underground construction can be entirely avoided or mitigated.
- In fact, risk management will not remove all risks from the projects.
- For each risk, it is necessary to determine the level of acceptance.
- The RMP should be integrated in all phases of development of a project.
- The RMP is a dynamic process: initial risks will have to be constantly updated during the project life, while specific strategies for controlling the known residual risks will have to be put in place, and residual risks will have to be systematically re-assessed.
- The Client and the project manager must recognize that a certain risk will remain to be borne by the Client. This 'residual risk' must be accounted for in the Client's estimate of time and cost (Thompson *et al.*, 1992).

2.3.1 Identification of initial risks: the use of the Risk Register

Once the designer has identified the basic requirement of the Client and his tolerance to risk, the process of risk identification along the tunnel alignment and in the tunnelling process should be preceded by the definition of the Reference Design Scenario that involves the following actions (Chiriotti *et al.*, 2003):

- formation of a group of specialists according to the size and complexity of the project;
- a desktop study to gather all relevant information which could influence the choice of the construction methods: published data on the regional and local geological and hydrogeological conditions, utilities network, and sensitive structure along the identified project corridor; this information should be complemented later on by adequate surveys and investigations;
- collection and critical review of the experience gained from similar conditions, especially those where risks did actually become manifest, consulting also the Contractors and equipment Suppliers involved;
- collection of site investigation data and use of the field work done by experienced geologists and hydrogeologists to produce the best estimate of the geological model, the likely ground conditions, and their variability;
- identification of the possible construction techniques, and
- characterisation of the typical sections of the tunnel.

Once the Reference Design Scenario is defined, workshops and engineering judgment based on previous experience are used for identifying the associated risks through a check list (also integrating a check list from a similar project), i.e. start compiling the first part of the so-called Risk Register.

The Risk Register should be structured to include all of the following parts:

- families of hazards and, within each family, the list of hazards and their causes;
- quantification of the likelihood and impact of the hazards and, hence, of the risks;
- indication of unacceptable initial risks;
- identification of a specific strategy to reduce each initial risk (mitigation measures), and

- quantification of residual risks through a re-assessment assuming that the mitigation measures have been implemented.

The ultimate results of the analysis of initial risks shall provide a guide for making the necessary adjustments of the Reference Design Scenarios, selecting the best construction method for the tunnel, designing a plan of additional site investigations to reduce uncertainties, and identifying the optimum project alternative and all the relevant and necessary design and construction actions to be carried out.

If properly used, the Risk Register becomes an effective guide for development of the project, since it allows to support the strategic project decisions and to track, in a 'tabloid', the proposed action plan in terms of organisation, investigations, and the design and/or construction needs to reduce the identified risks.

In order to start listing the families of hazards, one should consider that designing the civil works of a new underground infrastructure in an urban area means starting from factual data to give birth to an ambitious idea that has to become a reality through the construction. Hence, the families of hazards have to be looked at in two parallel contexts: design input and construction methods. An example of Risk Register structure, which is not meant to be exhaustive, with the identification of hazards within different families of factual data and construction hazards, is given in Figures 2.4 and 2.5.

Among factual data and input information, families of hazards can be related to geology, hydrogeology, geotechnics, hydraulics, utilities, buildings, environment, road conditions, etc. For each family the list of peculiar hazards can be produced by interviews with key project participants and/or brain-storming within the project team. Figure 2.4 gives an example of a list of identified hazards related to geology, hydrogeology and geotechnics.

Regarding the selected construction method (e.g. EPB-TBM), families of hazards can be related to technology, start-up, drive, construction procedures, lining, tail void injection, dismounting of the TBM at the end of the drive, transport of the TBM through town, human factors, etc. Figure 2.5 describes an example of a list of identified hazards related to the drive mode and the segmental lining.

Causes can be intrinsic in the hazard or can be related to aspects that are not properly dealt with during investigation, design, and/or construction. An example of identification of hazard causes is given in Table 2.1.

The result of the brainstorming session will be a comprehensive list of potentially critical situations that will have to be quantified (i.e. quantification of the associated risk) and to be given a response at the design stage, through proper design choices or through design prescriptions for the subsequent construction phase.

A rule derived from experience is that a Risk Register should be updated constantly during the project life and used to communicate and share risk policy and residual risk acceptance.

2.3.2 The initial risk: qualitative risk analysis

Risk analysis can be both qualitative and quantitative, but in the relatively early step of the project qualitative risk analysis is often used. The qualitative risk analysis also becomes necessary whenever the nature and extent of the data are not sufficient for developing meaningful statistics and when the statistical analysis of the

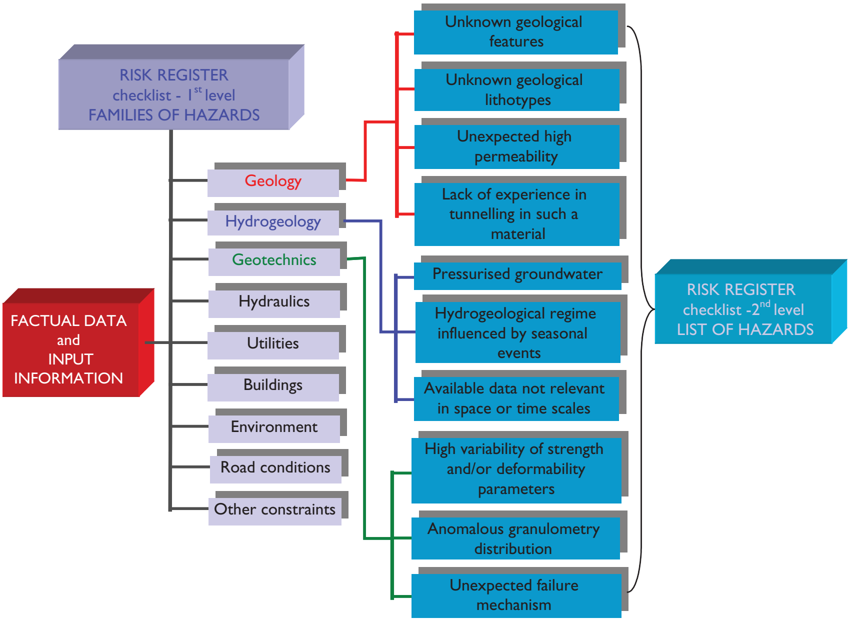


Figure 2.4 Example of the Risk Record for urban mechanized tunnelling: the 1st level (families of hazards) regards factual data and input information that leads to a first set of identified hazards (2nd level of the checklist).

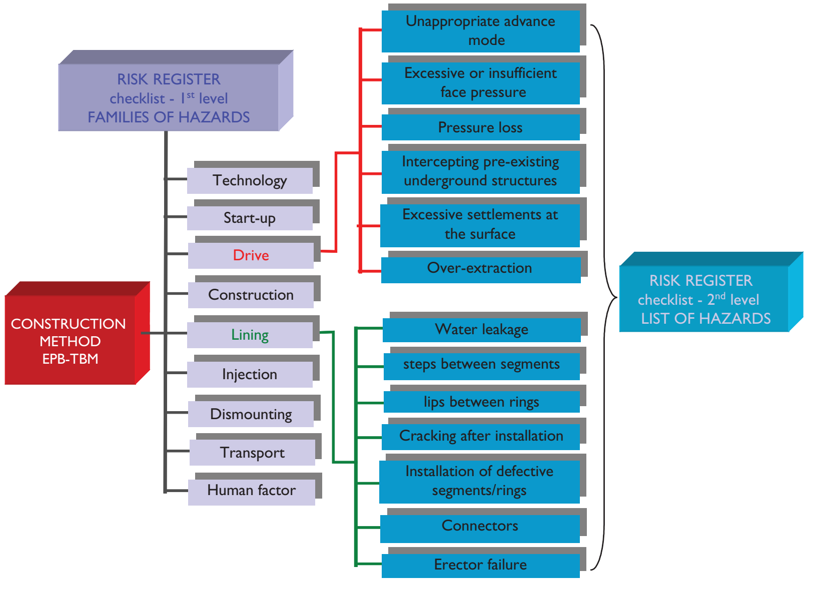


Figure 2.5 Example of the Risk Record for urban mechanized tunnelling: the 1st level (families of hazards) regards the construction method that leads to a second set of identified hazards (2nd level of the checklist).

Table 2.1 Simplified example of listing hazards and related causes referring to input information and construction method in a urban mechanized tunnelling project.

Family of hazards	*Causes*	
	Design	*Construction (mechanized)*
Factual data and input information GEOLOGY	– Insufficient data collection – Insufficient field work of experiences geologists – Local experts not involved – Lack of site investigations to reduce local uncertainties – Lack of geological model validation during pre-construction activities (boreholes to install in-ground monitoring equipments) – Lack of geological model validation during construction (probing ahead, face mapping)	– TBM is not equipped to probe ahead – Face mapping is not systematic during cutterhead maintenance and whenever possible – Lack of strict controls of face pressure to maintain face stability – Recruited personnel without enough experience
Factual data and input information HYDRO-GEOLOGY	– Insufficient data collection – Late start in the collection of data – Groups of available data statistically not meaningful – Insufficient tests both on site and in labs – Ground failure mechanisms related to tunnelling not fully understood	– Insufficient piezometers – Insufficient correlation studies of rainfall data vs. piezometric readings – Late start in the installation of monitoring instruments – Monitoring instruments not installed
Factual data and input information GEOTECHNICS	– Insufficient data collection – Groups of available data statistically not meaningful – Insufficient tests both on site and in labs – Tests are not adequate to define all the peculiar behaviour of the ground – Design using just average parameters – Ground failure mechanisms related to tunnelling not fully understood	– Lack of controls on the excavated material – Lack of correlation of TBM parameters and geotechnical conditions at the tunnel face
Construction method (EPB-TBM) DRIVE	– Operational face pressure ranges not prescribed – Operational face pressure ranges calculated by using inadequate methods – Lack of prescriptions for the advance mode – Lack of prescriptions for validating advance mode and face pressures during construction according to the encountered conditions – Insufficient collection of data on potentially interfering structures	– Lack of automatic controls of key driving parameters – Face pressure out of prescribed ranges – Advance mode not consistent with the prescribed one – Lack of procedures for validating advance mode and face pressures during construction – Unexpected underground feature causing pressure loss – Recruited personnel without enough experience

(Continued)

Table 2.1 Continued

Family of hazards	*Causes*	
	Design	*Construction (mechanized)*
Construction method (EPB-TBM) LINING	– Insufficient reinforcement in the segments – Wrong or low-pressure resistance gaskets – Insufficient load conditions considered during the dimensioning of the segments – Critical scenarios not considered in the dimensioning of the segments	– Lack of quality controls during segment production – Lack of quality controls during construction – Lack of maintenance – Defective installation – Defective storage and handling of the segments during transportation and on site – Recruited personnel without enough experience

data cannot identify the specific problems (such as location of the faults or anomalous ground).

The process of qualitative risk analysis starts with the identification of risk and aims to give an initial risk assessment.

The Risk Register is used to associate with each identified hazard the probability of occurrence and an impact in terms of likely consequences, including a first approximation of their potential effect on health and safety, estimates of cost and time, and exploitation.

Probability (P) and impact (I) are assigned using the qualitative scales that are prepared to suit the requirements and constraints of the typical project. An example used for the Porto Metro Project is given in Figure 2.6. For both the definition of the qualitative scales and the assignment of P and I to hazards, engineering judgment is used, again through interviews with key project participants and experts and through brain-storming sessions with the project team and experts.

The qualitative description given to probability has to be relevant to the project duration and conditions. The impact can be assessed in terms of: health, safety, and environment impact during construction; construction delay; foreseeable extra-costs; health, safety and environment impact during exploitation. The criterion for qualitatively assessing the impact has to suit the peculiar characteristics of the Project and can also consist of a combination of criteria.

In the case of Figure 2.6, three criteria were used for assessing the impact : (1) health, safety, and environmental impact during construction; (2) commercial impact (extra-cost for additional safety measures); and (3) health, safety and environmental impact during exploitation. By defining P and I, the risk, R, is defined as their product. Hence, construction, commercial, and operational risks are estimated separately (see Fig. 2.7). The resulting scale of risks (or risk matrix), with score rates from 1 to 25, is associated with a level of estimated risk (from irrelevant to unacceptable, see Fig. 2.7) and, more important, with the project-specific acceptability criteria ('low' = accepted; 'medium' = to be further analyzed in order to decide whether to accept or reduce it; 'high' = to be reduced). Risks can then be prioritized, singling out those that need to be mitigated and those that can be accepted.

PROBABILITY	SCORE	DESCRIPTION	DESCRIPTION
	1	Improbable	About 1 in 1000
	2	Remote	About 1 in 100
	3	Occasional	About 1 in 10
	4	Probable	More likely to happen than not
	5	Frequent	Expect it to happen

IMPACT	SCORE	HEALTH, SAFETY AND ENVIRONMENT Risk during construction	COMMERCIAL RISK Cost of implementing safety measures	H, S AND E RISKS DURING OPERATION (assuming 100yr life)
	1	Minor injuries/inconveniences. Operative can continue work. Short term local damage	ʃ10k extracost	Minor injuries/inconveniences. Operative can continue work. Short term local damage
	2	Minor injuries. Operatives require first aid treatment. Stop work. Medium term local/ regional damage	ʃ100k extracost	Minor injuries. Operatives require first aid treatment. Stop work. Medium term local/ regional damage
	3	Reportable / lost time injury or illness. Long term local/ regional damage	Delay in project of several weeks. Cost to project ?1Ms	Reportable / lost time injury or illness. Long term local/ regional damage
	4	Major injury or illness with long term effects. Long term widespread damage	Delay in project of several months. Cost to project ʃ1oMs	Major effect to city. Closure of a railway for at least 24 hours
	5	Fatalities. Widespread permanent damage	Potential close down project	Fatalities. Widespread permanent damage

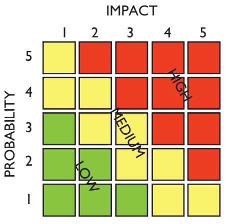
Figure 2.6 Examples of the qualitative scale of the probability and the impact of an event.

Alternatively, the impacts from the various categories of risk-generating aspects (referred to in Fig. 2.9): 'construction', $C1$; 'commercial', $C2$; and 'exploitation and long-term safety', $C3$ can be added to obtain the total impact $I = I(C1) + I(C2) + I(C3)$. In this case, a unique risk matrix will be obtained, as shown in Figure 2.8.

Figure 2.9 provides an example of Risk Register Form. After identifying the hazards, the Risk Register Form is used to track the specific risk analysis of each hazard. The hazard, for example, 'tunnelling - TBM operation - ground loss', is given a code. Then it is described by listing its causes and describing qualitatively estimated consequences in terms (for example) of construction, commercial, and exploitation impacts. Based on the specific qualitative scales of P and I, the initial risk is quantified and, in case it is unacceptable, mitigation measures are listed to be implemented both at the design and at the construction stages, in order to reduce the probability and/or the impact. By assessing again, both P and I, and assuming that the mitigation measures are in place, a qualitative estimate of the residual risks can be obtained.

The Risk Register Form can be completed with additional information such as the Owner of the initial and of the residual risks (sometimes, after implementing the mitigation measures, the risk owner can change) or the estimate of the residual additional cost, in case residual risk materializes. The latter information, even if it is produced through engineering judgment, allows the uncertainty in the cost to be declared, as recommended in Figure 2.3.

The main advantage of the qualitative risk analysis is to allow the proper tuning of the Reference Design Scenario in a clear, consistent, and shared way. The qualitative risk analysis is usually based on the rich experience of the designer and his experts who systematically analyze every single detail to create the comprehensive list of actions that can make the Reference Design Scenario become the Most Safe Reference Design Scenario.



SCALE	LEVEL OF RISK	REQUIRED ACTIONS
17-25	unacceptable	The construction is not initiated before the risk has been reduced. If the risk cannot be mitigated, there is a risk of the project being stopped.
13-16	substantial	The construction is not initiated before the risk has been reduced. The solutions for risk exist, but require the use of additional resources.
9-12	significant	The construction can be started and continued until such time as the counter measures need to be implemented.
5-8	minor	The work is generally subject to delays. Need to consider the solutions for optimizing the costs.
1-4	irrelevant	None.

RISK	SCORE	HEALTH & SAFETY RISK	COMMERCIAL RISK	OPERATIONAL RISK
	Low	Check that no further risks can be eliminated by modifications of design. Proceed with design.	Seek alternative, and assess cost to benefit ofmitigation measures in relation to severity of risk.	
	Medium	Consider alternative design or construction method. If alternatives are not available, specify precautions to be adopted. List residual hazards in risk register.	Disseminate risk assessment information to senior management, affected stake holders and third parties as appropriate.	
	High	Seek alternative solutions. If alternatives are not available, specify precautions to be adopted, and advise senior management and Planning Supervisor (where applicable). List Residual hazards in risk register.	List residual hazards in risk register.	

Figure 2.7. Example of a qualitative scale of risk associated with an event.

PROBABILITY						
	5	0 - 15	20 - 30	35 - 45	50 - 60	65 - 75
	4	0 - 12	16 - 24	28 - 36	40 - 48	52 - 60
	3	0 - 9	12 - 18	21 - 27	30 - 36	39 - 45
	2	0 - 6	8 - 12	14 - 18	20 - 24	26 - 30
	1	0 - 3	4 - 6	7 - 9	10 - 12	13 - 15
		0 - 3	**4 - 6**	**7 - 9**	**10 - 12**	**13 - 15**
		IMPACT				

Figure 2.8. Risk Matrix and acceptablility criteria for $I = I(C1) + I(C2) + I(C3)$ (see Fig. 2.9 for additional illustration).

However, the qualitative risk analysis is not sufficient for reaching the goal of RMP. In fact, the qualitative risk analysis is not able to provide answers to the following questions:

- Based on the identified risks, what is the reliability of the estimated project cost and duration?
- How much impact do residual risks have on project cost and duration?
- How can different project alternatives be quantitatively compared, from the perspective of (1) their effectiveness in managing the identified risks, and (2) reducing their impacts on potential cost and time overruns?

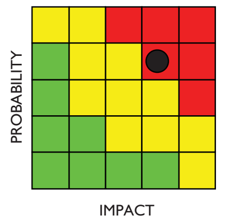
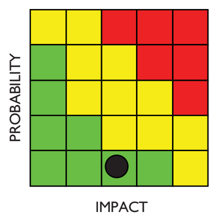
REGISTRY OF RISK IN MECHANIZED TUNNELLING	
CODE: TUN01	HAZARD: Ground loss
DESCRIPTION	
Ground loss leading to excessive surface settlements	
CAUSES	CONSEQUENCES (components of impact I)
❑ Over-excavation ❑ Insufficient face pressure ❑ Incorrect cutterhead speed ❑ Unexpected ground feature ❑ Slow response of operator to changing conditions intersected	*C1 – Construction (schedule, health and safety)* Major construction problems related to potential long-term stoppages of the works and damage to third parties. *C2 – Commercial (cost)* Medium cost impact for repairs; severe cost impacts in case of evacuation, induced damages and/or fatalities. *C3 – Exploitation and Long-term Safety* Minor issue during exploitation.
ASSESSMENT OF INITIAL RISK	
C1 3 + C2 4 + C3 1 = I 8 x P 5 = R 40 NOTE: Initial Risk UNACCEPTABLE OWNER: Contractor	
MITIGATION MEASURES	
DESIGN PHASE	CONSTRUCTION PHASE
❑ Calculate the operational ranges of the TBM key parameters along the tunnel profile, to be used by the TBM-operating staff. ❑ Prescribe TBM procedures. ❑ Predict ground settlements and damage to buildings. ❑ Design an adequate geotechnical and structural monitoring system (works, ground and buildings). ❑ Update design predictions during construction.	❑ Implementation of TBM construction procedures ❑ Trained TBM personnel ❑ Implementation of automatic alarms on key-parameters governing the TBM drive. ❑ Provide the TBM with automatic systems to maintain the face pressure during stoppages for segmental lining installation ❑ Remote TBM monitoring ❑ Implement GIS system for collecting and cross-checking monitoring and TBM data ❑ Systematic interpretation of construction and monitoring data.
ASSESSMENT OF RESIDUAL RISK	
C1 6 + C2 6 + C3 1 = I 3 x P 2 = R 9 COUNTERMEASURES: Prepare Emergency Plan. OWNER: Contractor	
Cost of residual impact:	< 1,000,000 ?
Residual probability:	< 10%

Figure 2.9 Example of Risk Register Form for tracking the identification and qualitative analysis of risks.

To give effective answers to the above questions, a further step has to be taken in the RPM: the quantitative risk analysis.

2.3.3 The initial risk: quantitative risk analysis

Quantitative risk analysis means substituting the qualitative risk judgment with a quantitative estimate of the probability and impact of a hazard or risk event.

Statistics allow assigning a probabilistic distribution to various events, both discrete (e.g. through Poisson distribution) and continuous (e.g. through Gaussian, logarithmic, or exponential distributions).

Probability describes the level of uncertainty associated with a variable. In the field of urban mechanized tunnelling the concept of probability can be applied to most of the project input variables, such as the geotechnical parameters, the spatial sequence of the state of a parameter (e.g. lithology changing from A to B to C along the tunnel profile), the duration of a construction cycle, and discrete events such as the rise of an adverse situation (e.g. unknown ground feature, tunnel face instability, accidents, etc.).

Hence, data regarding the ground characteristics, the construction variables, and the unpredictable events can be treated statistically to identify the most appropriate probability distribution function for each variable. For example, the unconfined compressive strength of the ground and its modulus of deformation can be represented through a Gaussian distribution; joint spacing is well described by a negative exponential distribution, and a simple triangular distribution can be used to represent the duration or cost of a single construction cycle under predefined conditions.

From the geological and geotechnical/geomechanical points of view, the concept of probabilistic characterization allows for the definition of a probabilistic geological profile, hence permitting visualization of the level of uncertainty associated with the 'geo' aspects. The main advantage is that the geological uncertainty is considered explicitly, in comparison with the classical engineering geological profile that is just a best-guess, representing the most likely conditions along the tunnel alignment (i.e. one of the possible predictions based on the geologist's experience). In urban tunnelling, the geological profile does not influence heavily the calculation of the tunnel segmental lining since, being in a situation of shallow cover, the load condition to be considered for the dimensioning is generally the overall overburden. However, the geologic profile does influence many other aspects: the confining pressure to be applied at the tunnel face, the injection pressure of the annular tail-void, the potential of settlements, the width of the settlement trough, the wearing of the cutters, the advancing speed, and the potential for over-excavations and collapses if the excavation procedures are not timely adapted to sudden geological changes at the tunnel face (see Section 6). Finally, the geological profile also influences the time and costs. Therefore, it is possible to prepare a probabilistic profile to introduce the concept of 'variations' in terms of time and cost impacts.

Quantifying the impact of a hazard is mainly done to quantify its consequences in terms of project time and cost from different perspectives (e.g. construction, maintenance, exploitation). That is, to quantify how a potentially critical event, E, whose likelihood of occurrence is described by a probability, P, can impact the cost of the project in reference to a best-estimate base cost. Because the estimate of a future project cost or schedule involves substantial uncertainties (risks), the uncertainty must also be included in the cost-estimating process.

Considering that there are natural fluctuations in the project time and schedule due to the likely range of variation of some of the input parameters (e.g. geology and construction aspects), and considering that risk events, if they occur, all produce impacts which add cost and/or time to the project, the cost estimation must include both the expected (or foreseen) variation and the risks (i.e. account for uncertainty), using a logical and structured process. A range of 'probable costs' can then be defined (Figure 2.10).

The 'range of probable costs' consists of three components (Grasso *et al.*, 2006):

- The 'normal cost', that is the best-estimate of the basic cost, is calculated on the basis of a "bill of quantities" with reference to the Most Safe Reference Design Scenario.
- A 'variance' corresponding to the foreseen variation in the ground and construction parameters.
- The cumulative cost of each identified residual risk.

In urban mechanized tunnelling projects, unacceptable residual risks can be due to the occurrence of face instability when operating by a combination of open and closed modes when the closed mode is not timely activated in response to the changes in geology at the tunnel face or when, due to human error, an inadequate face pressure is applied.

The 'variance in cost' is not calculated directly. It is assessed by subtracting the normal cost from the range of total cost obtained by simulating (a relevant number of times) the process of construction along the probabilistic profiles. A Monte Carlo sampling procedure can be implemented to extract from the distribution of the various parameters the values to be used in the particular simulation. The simulation also accounts for the probabilistic distribution of duration and costs of the tunnel construction in each particular geotechnical context identified along the tunnel layout.

Additionally, it is important to choose the correct way of summing up the individual residual risks. There are two possible methods:

- To analyze any risk independently of others, with no attempt to estimate its probability of occurrence, and to accumulate the estimated effects of each risk, thus providing the maximum and minimum project-outcome values. Clearly, this is a simplified method which may exaggerate the total project risk.
- To apply probabilities to the risks and consider the inter-dependencies between the risks. Especially in urban areas, 'risk inter-dependency' should also include an evaluation of the negative, evolving consequences (impacts) of a recurring, discrete event in terms of increase in the political and social impact. In these cases, exponential functions can express the increasing time and cost impact of a recurring event due to an increasingly negative opinion of the public.

2.4 DESIGNING FOR THE IDENTIFIED RISK SCENARIO

The Designer should be able to manage all kind of design risks related to geology, hydrogeology, load conditions, construction method and all physical and environmental impacts both surface and subsurface, from the conceptual design to the follow-up of the construction.

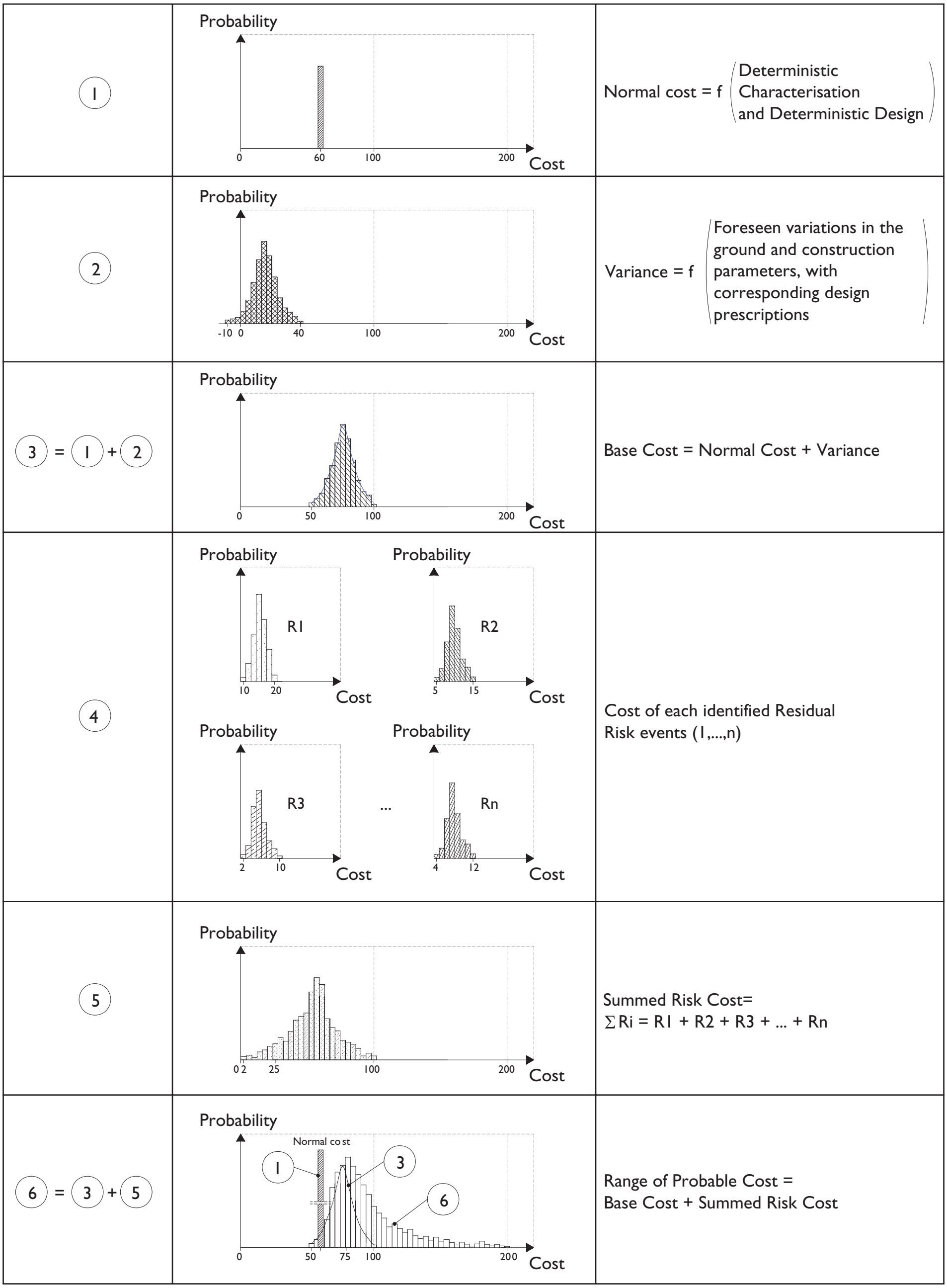


Figure 2.10 Cost model showing the breakdown of the "Ranges of Probable Costs" of constructing a tunnel under conditions of uncertainty (Grasso *et al.*, 2006).

In the previous subsections, attention was drawn to the identification and quantification of risks affecting an urban-transformation project in which the use of underground space and the application of mechanized tunnelling are prominent.

Once the Reference Design Scenario has been established and the sources of risks have been properly characterized, a consistent method needs to be developed to give response to risks through the design process.

The RMP for an urban mechanized tunnelling project generally requires the following design components:

- Planning and conducting investigations that are adequate for the level of geotechnical and environmental risks of the project.
- Recognizing that the deterministic approach is generally less appropriate and giving preference to sensitivity and risk scenarios analysis and to probabilistic approaches. This applies in particular to the definition of the geological profile, to the calculation of any factor of safety and to the prediction of project time and costs.
- Design through risk scenarios not only considering the most likely load condition or the average geotechnical parameters, but also analyzing the consequence of encountering - even locally - the most unfavourable situations.
- Adopting a transparent risk mitigation policy throughout the design process.
- Adopting a flexible design, in order to be prepared to face unfavourable situations, giving attention to the following critical elements:
  - defining countermeasures to manage and reverse adverse trends during construction;
  - identifying key-parameters and/or key-events to be controlled and monitored during construction in order to timely detect adverse trends;
  - defining the relevant operational ranges or threshold limits of the key-parameters, and
  - predefining the triggering criteria for activating the countermeasures, if ranges/limits are exceeded.
- Quantifying (at least using qualitative methods) and communicating residual risks.
- Whenever possible, using probabilistic methods to assess the design reliability: average factors of safety (for the support and the prescribed face pressure) are replaced by the probability of occurrence of a negative event (e.g. face instability), and the probability has to be reasonably low.
- Adopting special care for establishing a Building Protection Strategy which requires the following actions:
  - survey all identified buildings in the construction-influence zone before construction starts (BCS - Building Condition Survey);
  - use the BCS results for assessing the vulnerability of the buildings to damage;
  - establish a specific damage classification system for the project;
  - perform settlement-sensitivity analysis for each identified building and define its tolerance to tunnelling-induced movements;
  - classify all identified buildings into different risk categories;

- single out the buildings at risk that require protection and design the relevant mitigation measures;
- identify the buildings requiring surveys and special monitoring during construction;
- develop an effective monitoring plan;
- perform a post-construction BCS, independent of whether the damage has occurred or not, and
- archive and maintain all relevant data in a dynamic-and-relational database for use by all Parties.

The optimum effectiveness of the RMP is achieved if the appropriate excavation method is accompanied by the competence and training of personnel and by the establishment and implementation of procedures to guide all the relevant construction processes, to govern the key events, and to address all the potential anomalies with a proper and predefined action plan. Within the RMP framework, the Designer can assume two important roles in the construction phase:

- Interact with the TBM Manufacturer and the Contractor in order to contribute new ideas for technological innovations, with the aim to assure safety and improve the production in terms of advancement in all kinds of geological context.
- Validate the design hypothesis by observation and monitoring during construction and make use of the construction feedback to match the design assumptions to the actually encountered conditions and to optimize the design, also in terms of costs.

An example of designing through risk scenarios and of tracking and communicating the initial and residual risks, is given in Figures 2.11–2.16. The example refers to the design of the segmental lining of a TBM tunnel.

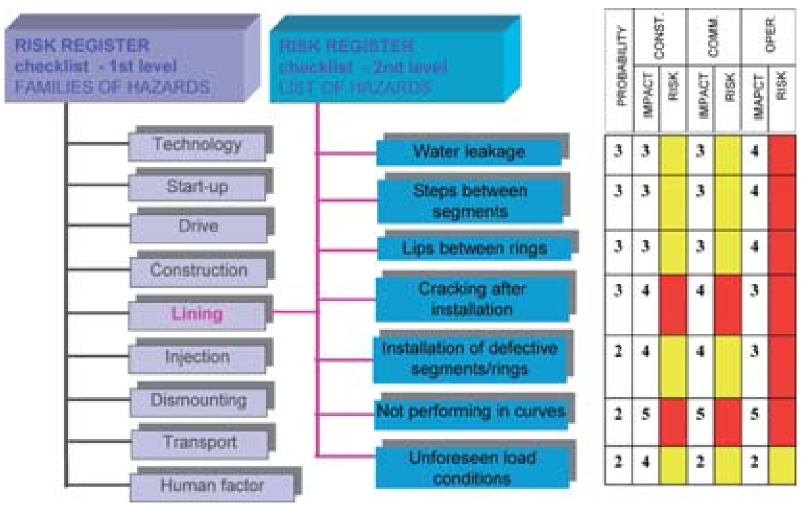


Figure 2.11 Design of tunnel lining in response to risks: assessment of initial risks.

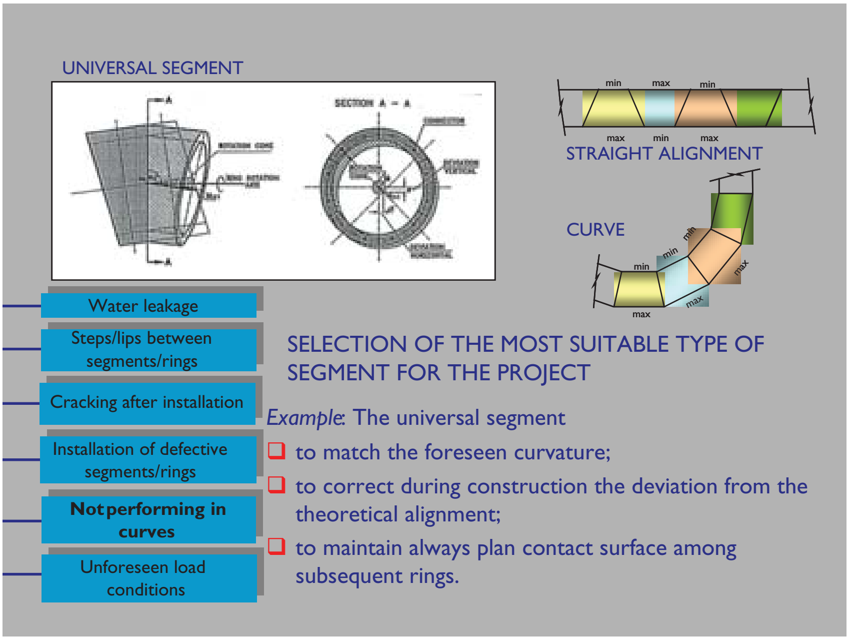


Figure 2.12 Design of tunnel lining in response to risks: measures for reducing the geometrical risks.

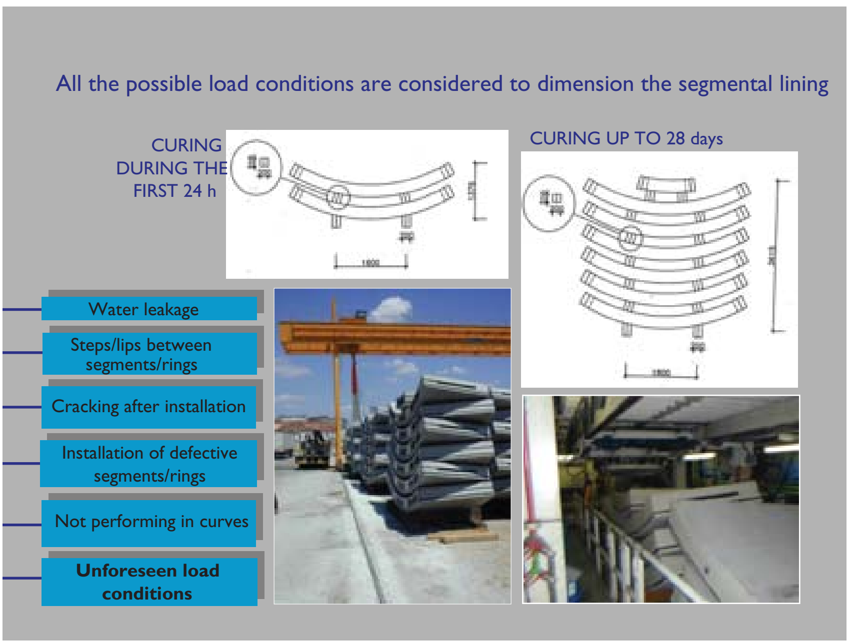


Figure 2.13 Design of tunnel lining in response to risks: measures for reducing the risk of unforeseen, ordinary load conditions.

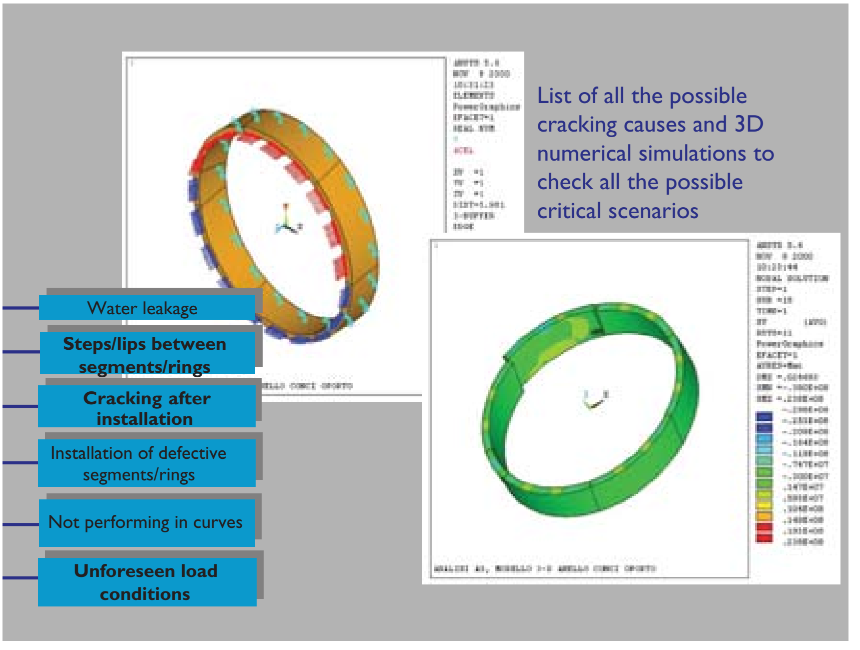


Figure 2.14 Design of tunnel lining in response to risks: measures for reducing the risk of critical scenarios leading to anomalous load conditions, cracking and defects.

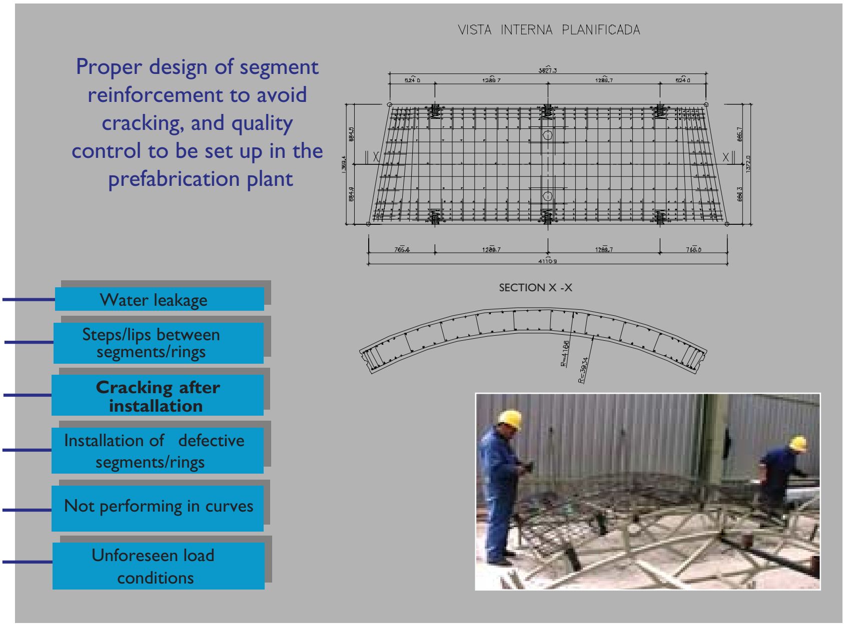


Figure 2.15 Design of tunnel lining in response to risks: measures for reducing the risk of cracking.

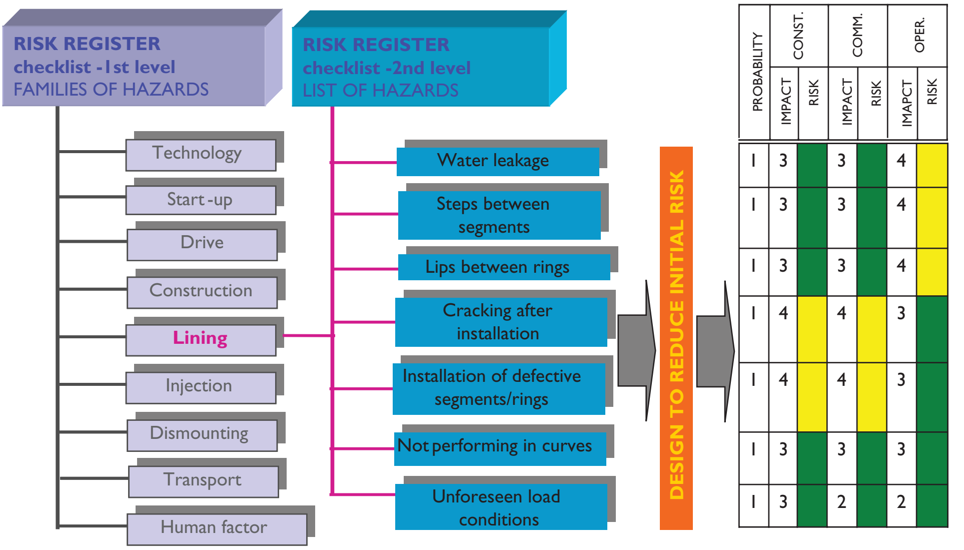


Figure 2.16 Design of tunnel lining in response to risks, assessment of residual risks and evaluation of its acceptability.

The following observations can be made regarding the design and performance of the segment lining in this example (with due regard to the risk register of Fig. 2.5): Figure 2.11 shows the list of potential hazards related to the segmental lining design and the associated qualitative risk assessment, obtained according to the criteria described in the previous section.

From an analysis of the results, it is clear that the majority of the risks will have to be reduced at the design stage through different kinds of actions:

- selecting the most suitable segmental ring (for the project alignment) and its geometrical characteristics (Fig. 2.12);
- considering all the possible load conditions to dimension the segments, including the ordinary loads (different stages of curing, see Fig. 2.13, de-moulding, storage, transportation, handling, etc.), and the critical scenarios leading to cracking and anomalies, such as asymmetric loads by the hydraulic jacks or failure of connectors (see Fig. 2.14);
- dimensioning an adequate steel reinforcement and setting appropriate quality control procedures in the prefabrication plant (Fig. 2.15).

Once all the actions to mitigate potential initial risks have been implemented, the residual risk can be estimated (Fig. 2.16). This is a simple and fundamental step of the design process since it allows one to communicate, to the Project Actors, the risks that are still present and to decide whether accepting or further reducing them (for example, by increasing the steel reinforcement or improving the ring erection technology and procedures).

2.5 QUANTIFYING THE RISK OF TIME AND COST OVERRUNS AT THE DESIGN STAGE WITH THE USE OF DAT

The system DAT (Decision Aids in Tunnelling) is a software for making probabilistic estimates of the range of probable time and cost of constructing a tunnel, or a network of tunnels, taking into account the variability and uncertainty in the geologic and construction variables and the impacts of residual risks, which could lead to deceleration or even stoppages of the works along certain portions of the tunnel alignment.

DAT was developed in the 1980s by MIT (Massachusetts Institute of Technology) with the subsequent participation of EPFL (École Polytéchnique Fédérale de Lausanne); it has been applied to various projects by Geodata since the early 1990s (Xu *et al*., 1996, Einstein *et al*., 1998a,b).

DAT simulates the construction cycles of a tunnel by following a proposed construction sequence along a probabilistic geological profile (as defined in Section 2.3.3) that stochastically changes for each complete simulation process, for a probabilistically-significant number of runs.

A DAT run is essentially a computer simulation of several random processes. Simulation of the construction process generates statistical information about the total time and cost. This information gives a good idea on the average, minimum and maximum expected values. By definition, the simulation of a random process uses a random number generator.

A unique feature of DAT is its capability to make a comparative evaluation of the performance of project alternatives (different construction schemes, in terms of alignment and methods of construction), with respect to the potential of these alternatives in managing geotechnical and construction uncertainties within prescribed, or acceptable, values of time and cost.

DAT has two interrelated simulation modules: *Geology and Construction*.

In the *Geology Module*, all relevant 'geo-variables' (geological, hydrogeological, and geotechnical/geomechanical) that have an impact on the tunnel construction and whose 'parameter-state' combinations define "ground classes" are input into the programme in a probabilistic form.

The user's (designer's) task is are to identify and define those parameters and what their possible states are. Uncertainty in this definition is either represented by indicating the variability in the assigned value of the parameter, and/or reflected in the probability of occurrence of the possible states of that parameter at a given location interval.

The process of a probabilistic profile generation, in terms of allocation of ground classes along the tunnel alignment, consists of the following steps:

- Subdivision of the tunnel alignment into homogeneous zones defined by similar 'geo' conditions. The variable length of these zones may be defined using a triangular distribution.
- For each homogeneous zone, the 'geo' parameters, which determine the excavation method and the support measures, are defined in terms of their possible (parameter) states. The 'geo-variables' are organized in various input matrices following an

approach similar to that of defining a geotechnical profile, i.e. defining, chainage by chainage, all the 'geo' conditions that have an impact on the tunnel construction.

- The variability of conditions inside a homogeneous zone is modelled using a Markov process. For each parameter, the average state-extent and a transitional matrix are provided (parameter states along the tunnel alignment can also be assigned deterministic values).
- In a manner similar to defining the geomechanical classification, different parameter states are combined to define homogeneous ground classes that are subsequently associated with different construction methods.

For example, if the groundwater pressure and the presence of high-strength abrasive rocks or unstable incoherent soils are identified as impacting parameters, their possible states have to be defined, together with the influence of their possible state combinations on every excavation phase modelled in the subsequent construction module.

The *Construction Module* consists of two principal components:

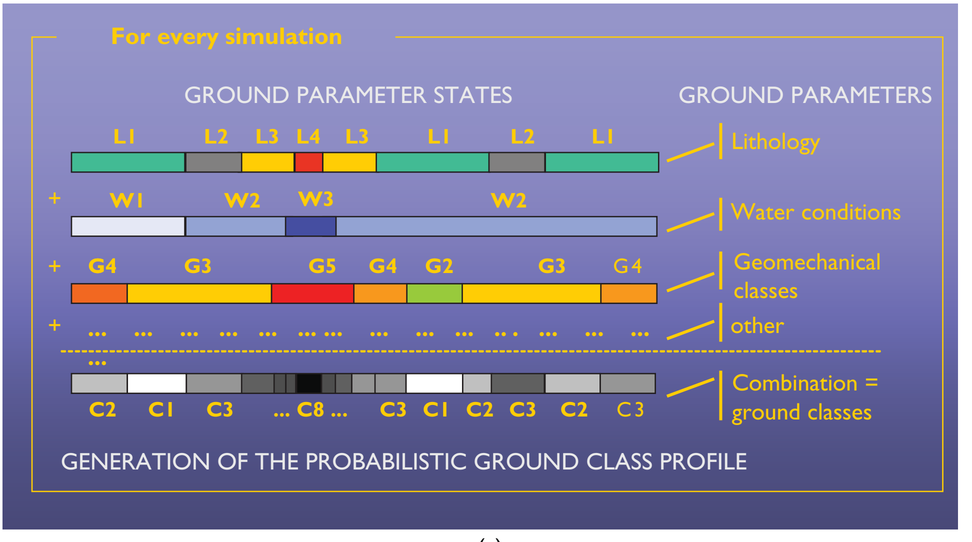
- The first refers to the construction methods, where the construction cycle can be simulated activity by activity. In this case, variability is introduced into the model by statistical distributions of basic construction indices, e.g. advance rate and unit cost, usually derived from practical case histories and price analysis.
- The second, referred to as the tunnel network, permits the definition of the sequence of realization of the various tunnel stretches comprising a project, e.g. two opposite fronts for a tunnel advancing from two adjacent stations of a metro by traditional excavation.

In both the geology and the construction modules, variability of the parameters is described through a user-defined distribution function that can be chosen from among Uniform, Triangular, and Bounded Triangular distributions. In the uniform distribution the variable always has the same probability of taking on any value. In the Triangular distribution a minimum value, a most likely value (the mode), and a maximum value have to be provided, recognizing that the total area under the triangle must equal one (as the total probability of occurrence of the parameter must be 100%). In the Bounded Triangular distribution, the probabilities on the minimum and maximum boundaries of the triangle are greater than zero.

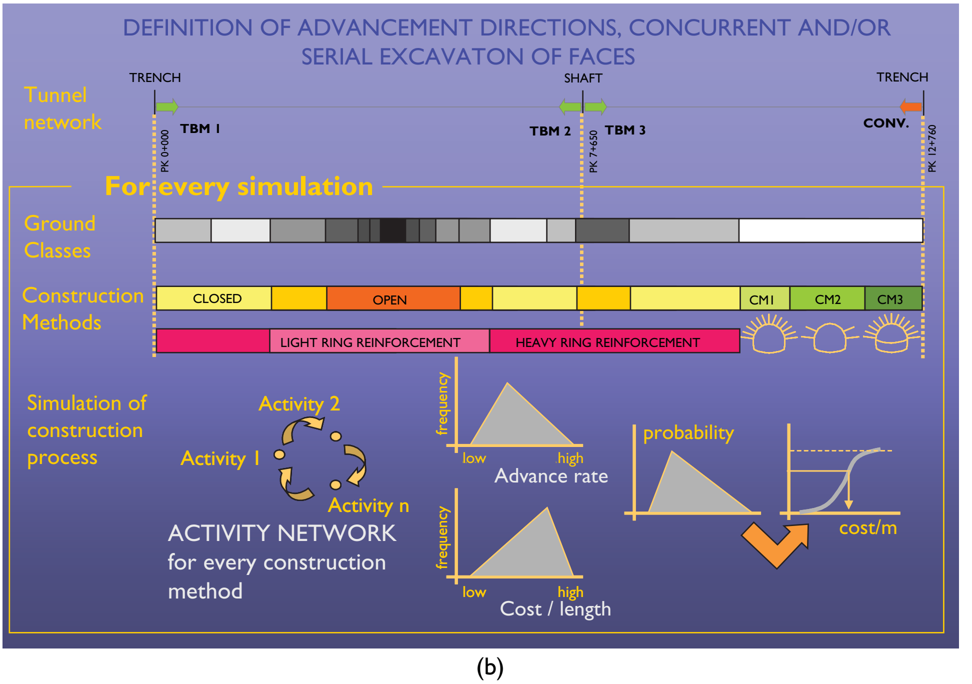
The schematic process of a DAT simulation is described in Figure 2.17.

The construction simulation is based on the Monte Carlo method of random sampling and follows, round-by-round, the already probabilistically-defined ground class profile. The procedure is repeated for all zones of a profile, adding up to a final cost-and-time value corresponding to that profile and to a point in the time vs. cost scatter diagram (Fig. 2.18). This procedure is repeated for each probabilistic profile generated by the Geology module. To achieve a statistically significant result, it is usually necessary to do no less than 200 and up to 1000 simulations.

Hence, the output of DAT is a probabilistic cost and time distribution, shown in terms of a scattered time-cost diagram. The distribution of points in the dispersion graph expresses, in an explicit form, the impact of uncertainties and/or residual risks on cost and time of construction.



(a)



(b)

Figure 2.17 Schematic process of a DAT simulation: the Geology Module (a) generates the ground class profiles and the Construction Module (b) simulates the construction process in that particular ground class profile by associating the construction method with the ground classes, for every simulation.

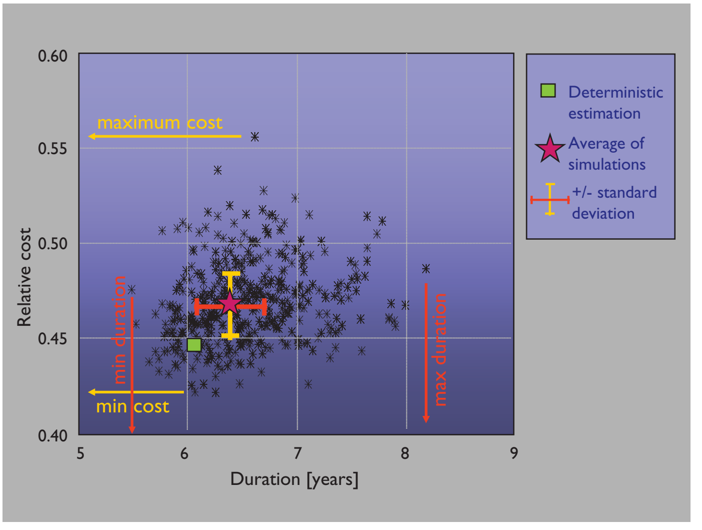


Figure 2.18 Comparison between the deterministic duration and cost assessment of the project Reference Design Scenario and the time and cost assessment introducing the variance of the geological and construction variables.

The probability of occurrence of a potential hazard leading to tunnel stoppages and/or other big accident, anticipated by the designer, can be expressed by a Poisson distribution, and its impact (on time and cost) can be described using an exponential function. The accumulated cost of all the identified hazards can be simulated and added (Fig. 2.19).

The fields of DAT application in urban tunnelling include:

- Evaluation of the total duration of the project and the associated costs considering the variance due to geological and construction uncertainties.
- Simulation of crisis scenarios (residual risks) and their impact on time and cost of the project.
- Definition of the degree of accuracy in the exploratory data required with respect to the predefined risk tolerance (see Fig. 2.20a).
- Comparison of project alternatives (see Fig. 2.20b).

In the case of urban tunnelling, DAT can be used both at the early stages of the design path and during construction. Two case histories are summarized below as examples of the use of DAT.

DAT was successfully used in 2000–2001 for the Porto Metro Project (for details of the project see Section 8.2, and Chiriotti *et al.*, 2003) in order to support decision-making in identifying the best ‘acceleration’ solution to recover an accumulated delay of 8 months (out of a total of 33 months) in the civil works construction of the running tunnel of Line C (2.3 km) and that of Line S (4.0 km). Initially it was planned to

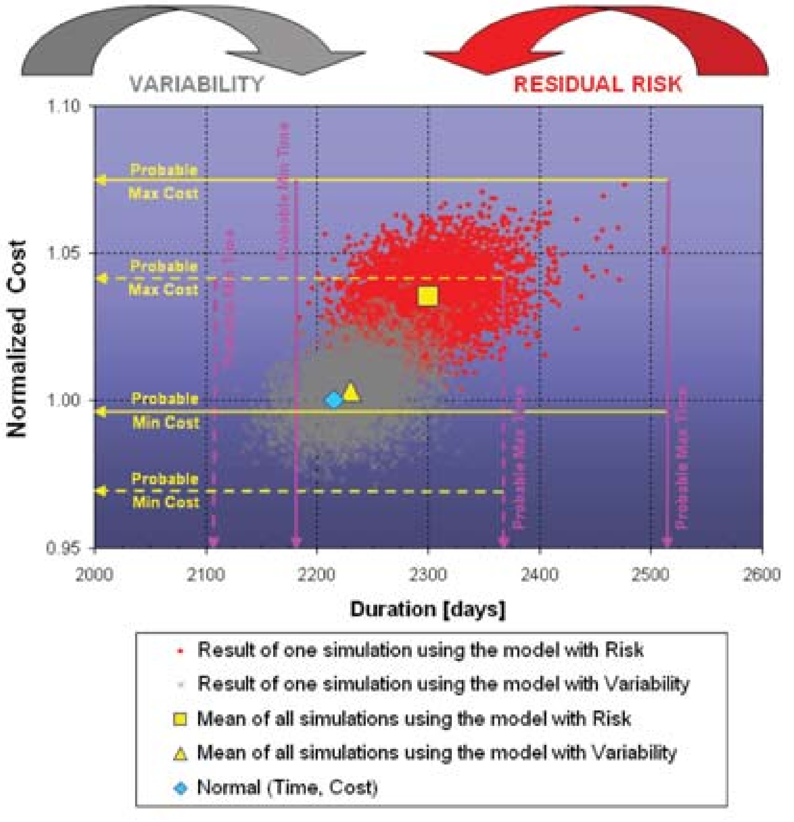
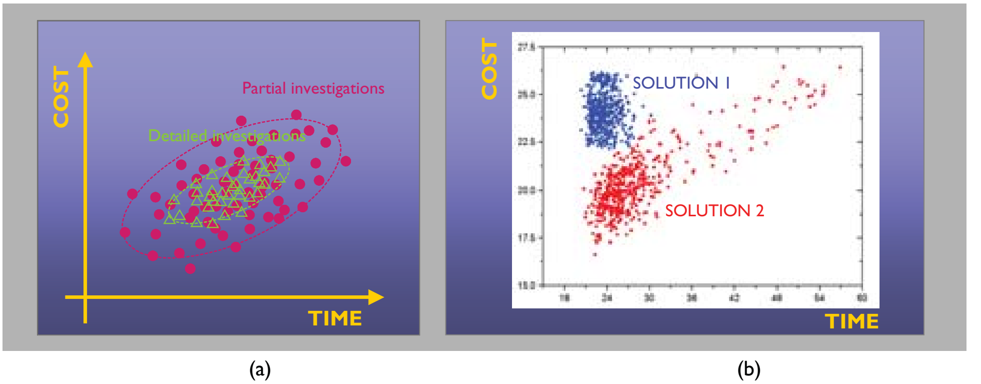


Figure 2.19 Example of the combined model of Cost and Time for a Reference Design Scenario.



(a) (b)

Figure 2.20 Use of DAT (a) to asssess the impact of insufficient geological investigations on time and cost, an d (b) to compare the effectiveness of two alternative design solutions in terms of reliability of their time and cost estimates (Chiriotti *et al.*, 2003).

excavate both tunnels using the same EPB-TBM which would start excavation of Line C first, proceeding East-West, from Campanhã Station to Trindade Station. Then, the TBM would have been removed and reassembled at the Salgueiros access ramp to excavate the entire Line S down to São Bento Station, proceeding North-South.

The first few hundred metres of Line C were excavated and the initial delay accumulated by the Contractor in the preparation works, together with some serious technical difficulties encountered by the TBM and TBM operators during the learning-curve phase were all pointing at the impossibility of meeting the contractual deadline for completion of the tunnels, putting tunnelling on the critical path for the success of the entire Project. The choice of an effective project alternative was required quickly.

Three project alternatives were identified in addition to the base solution, 'do nothing' (Alternative 0.1, Fig. 2.21): (1) Pre-consolidation of ground to be done at some locations along the alignment to allow a rapid advance rate of the tunnel boring machine in open-face mode. The consolidation would be effective in 80% of the cases (no accidents); on the other hand, accidents would be possible in the remaining 20% (Alternative 1.1, Fig. 2.21). (2) Reduction of the length to be excavated by the TBM, by introducing two sections to be excavated by conventional drill-and-blast method (Alternative 2.1, Fig. 2.21). (3) Acquiring a second TBM, and both TBMs would be always operated in close-face mode for excavating the two tunnels (Alternative 3.1, Fig. 2.21).

The alignments of both Line C and Line S tunnels were divided into homogeneous zones from the point of view of the advance mode (open, closed but not-pressurized face, and closed and pressurized face), on the basis of the geomechanical model. A triangular probabilistic distribution was used to represent the advance speed for each of the advance modes.

To realistically represent the excavation process, the DAT simulation also included instances of inadequate face support pressure for short periods (i.e. delayed reaction of the Construction Team to sudden geological changes at the tunnel face, lack of boreholes ahead of the face, inadequate pressure, over-excavations, etc.). This simulation was obtained by associating:

- to each advancing mode within a homogeneous zone the probability to drive the TBM in a less conservative mode;
- to each less conservative mode, the probability to cause an accident and the distribution of occurrence of the typology of the accident (long, medium, and short, in terms of the time required to overcome the accident);
- to each accident, a triangular distribution of the duration of the delay it would have caused.

Also, the duration of the different types of TBM ancillary works was expressed through a triangular distribution. Using the above procedure, a residual risk of instability of variable duration was introduced towards the possibility of decreasing the advance rate.

The results of DAT analyses showed Alternative 3 to be the best technical solution in terms of risks, time and cost (Fig. 2.22), provided that the residual risk of collapses was minimized by always operating the two TBMs in closed mode (i.e. with the plenum full of well conditioned excavated material, adequately pressurized). If an open-face mode is used in a situation where the use of close-face mode was foreseen

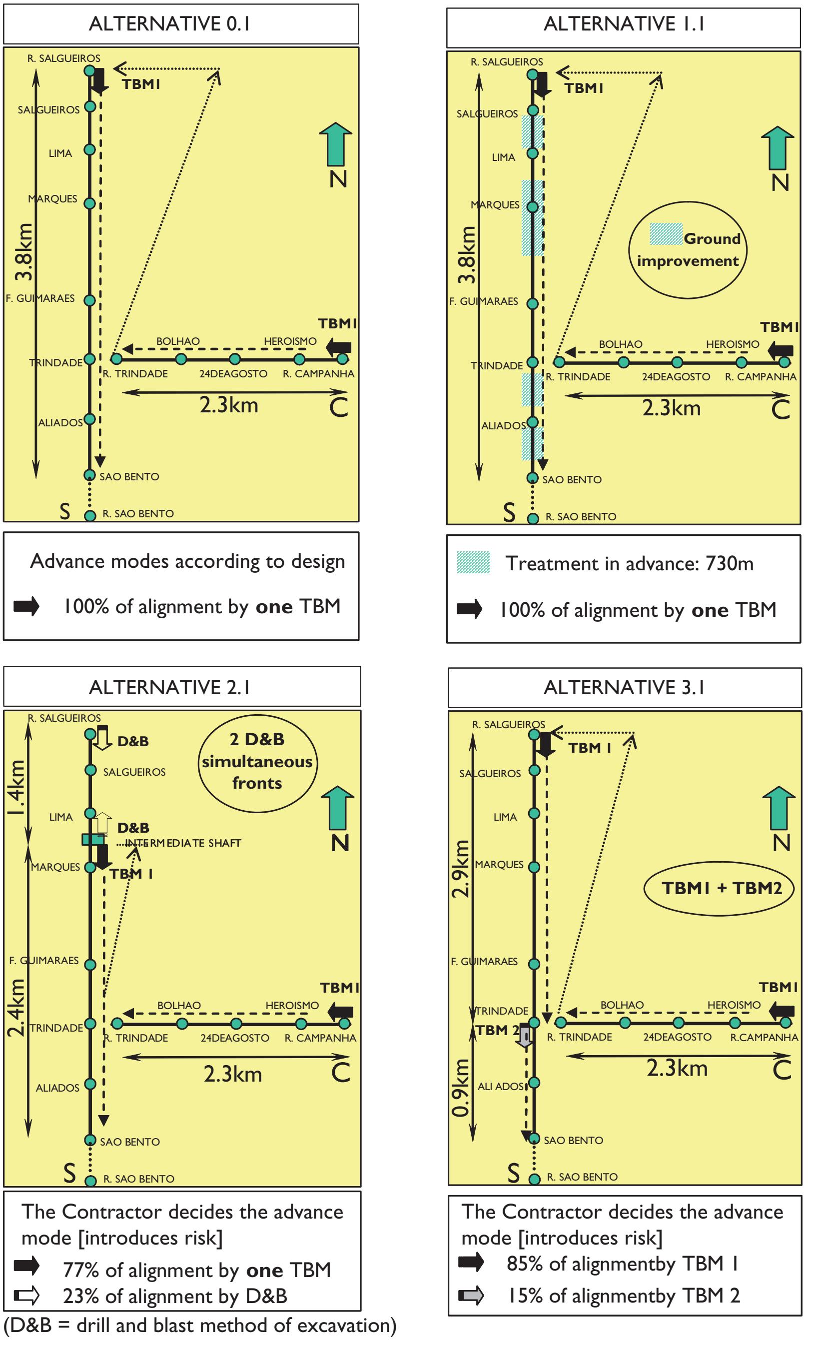


Figure 2.21 Schematic description of the tunnelling alternatives considered to identify the best solution to recover an 8-month delay on the schedule of the civil works of the Porto Metro Project (Geodata, 2000–2001).

(e.g. the contractor decides to change the advance mode foreseen by the design to improve production, or a sudden change of the geology is not followed by a quick operator response adapting the advance mode), there is a risk of excessive settlement and collapse, which translates into a delay in the construction. The risks of inadequate operation of the TBM and their impacts are simulated by means of DAT, introducing the exponential delay effect of successive accidents (Fig. 2.23).

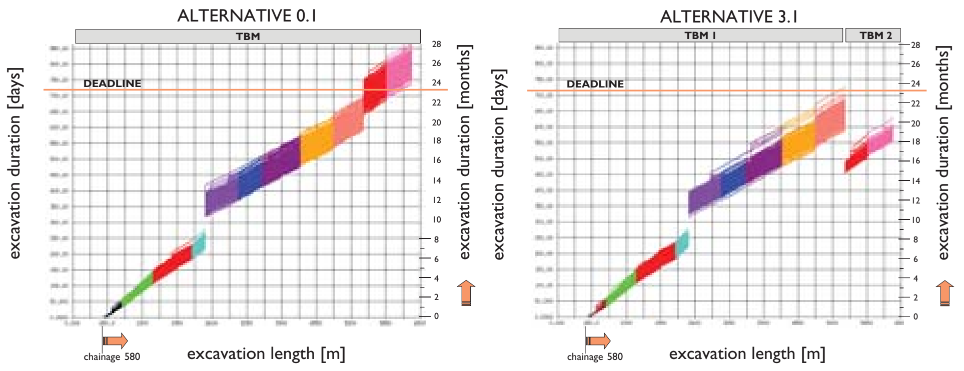


Figure 2.22 Example of DAT application for urban tunnelling in Porto Metro (by Geodata during 2000–2001), including simulation of the effects of successive accidents or residual risks.

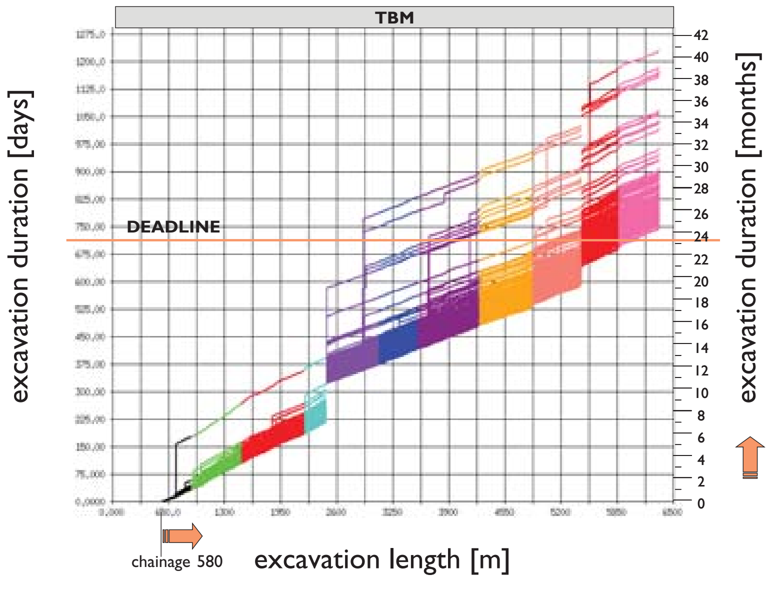


Figure 2.23 Example of DAT application for urban tunnelling in Porto Metro, showing the impact on the final deadline of accidents due to a wrong EPB mode selection by the operator or to a series of late reactions to sudden changes of geological conditions at the tunnel face (open instead of close mode).

2.6 USE OF A PLAN FOR ADVANCE OF TUNNEL (PAT) FOR CONTROLLING THE RESIDUAL RISKS

During the construction, the RMP, and the role of the Designer within the RMP, are of paramount importance. The tools (like Risk Register) and methods (such as risk identification and quantification, design through risk scenarios, etc.) need to remain active and dynamic to ensure that the design is always the most appropriate and is based on the "best-estimate" knowledge of the ground and of the inherent situation.

The concept of iterative design through the use of a Plan for Advance of Tunnel (PAT) was first introduced by Geodata in 2001 for the Porto Metro, Portugal (Grasso *et al.*, 2002b, Chiriotti *et al.*, 2004). The PAT is a live document that provides a dynamic link between design and construction and facilitates the management of residual risks. In fact, the PAT is a low-cost, easy-to-implement, practical procedure for the designer-contractor-engineer team to continuously update the risk scenarios and the corresponding mitigation plans as construction proceeds.

A PAT is produced (or updated) in advance of the excavation for each 200 to 500 m-long stretch of the tunnel. It summarizes both the design and construction requirements in order to achieve a safe performance; and it is based on the content of the initial design documents, on the construction feedback from the previous PAT stretch(es), and on new input data, if any.

A multi-disciplinary approach is used to update the identification of the initial risks and to keep under control the residual risks by:

- collecting, analyzing, and processing the TBM and the monitoring data relating to the previously excavated section;
- collecting, analyzing, and processing new data that can affect the local geological-hydrogeological reference model;
- collecting, analyzing, and processing the piezometric and the rainfall data in order to determine the need for adapting the face-pressure operational ranges defined in the design documents;
- reviewing the results of recent condition-surveys of the buildings and the information on pre-existing interferences;
- reviewing the need for monitoring instruments or the frequency of the readings, and
- reviewing the requirements in terms of TBM performances.

This information is then used to obtain an optimum prediction of the reference model and to summarize in a drawing, and in a short report, the following operational instructions:

- need for additional consolidation works or for reduction of foreseen consolidation works;
- most likely geological conditions at the tunnel face and in the overburden;
- most likely hydrogeological conditions and piezometric levels;
- position of the monitoring instruments (in the tunnels, in-ground, at the surface, on the buildings and utilities);
- summary table of the monitoring thresholds;

- frequency of readings for all foreseen monitoring instruments,
- operational ranges for the key parameters of the TBM: weight of the extracted material per ring, apparent density of the extracted material, face-support pressure, and injection pressure of the tail void,
- frequency and position of the probing-ahead holes,
- particular requirements related to the TBM drive: make no stoppages beneath sensitive buildings, inject bentonite around the shield to reduce the geometrical volume loss in sensitive grounds beneath sensitive buildings,
- requirements related to visual inspections of sensitive buildings starting from when the TBM is approaching the building, and until the stabilization of settlements is reached, and
- requirements for temporary evacuations in emergency situations.

The PAT facilitates the operators and technicians on the site since all the relevant information is updated and synthesized in short and synoptic documents, instead of being spread into different design documents.

The construction team is provided with the PAT documents, after the contents have been discussed and agreed with the Owner. At this point, the PAT becomes a live guide for driving the tunnel. It is used to further update the key parameters on a daily basis as a function of the real-time-monitoring data and to support the decision-making process.

All the Parties are, therefore, assured that the construction is proceeding as a controlled process.

3

Selection of tunnel alignment with low-level risks

3.1 INTRODUCTION

The construction of a new tunnel in any city environment is an immense task, often involving the investment of hundreds of millions of dollars over a few years. The task can be very difficult and challenging, especially where there is also the lack of experience, on the part of both the Owner and the local construction industry, in implementing such a large and complex project, in addition to the many uncertainties and risks involved in the project discussed previously in Section 2.

Selection of the alignment with low-level risks is a decision to be taken at an early stage, usually the Preliminary Design stage, of development of an urban tunnel project. This is indeed the second key step, after identifying the risks and assessing the initial-risk levels, of the mandatory approach advocated in this book for tunnel construction in urban areas (see Fig. 1.1).

Indeed, instead of waiting for troubles that will very likely emerge later on during construction, the early selection of the alignment with low-level risks follows the ancient Indian philosophy of "Be wise a priori". It is true that we have modern tools like an RMP to deal with hazards and manage the consequent impacts, if they do manifest themselves during construction (see Section 2), but it is definitely more convenient and economical if most of the high-level risks can be totally avoided through the correct choice of the alignment. It is believed that Clients should qualify their Consultants and Contractors on the basis of not only their ability to manage risks in the construction stage, but, more importantly, also their capability in choosing the right design-construction solutions that avoid medium- to high-level risks for the project. Selecting the alignment with only low-level risks, whenever possible, is laying down the solid foundation for, and preparing a smooth path to, the success of the project.

Specifically, the scope of work of selecting the alignment with low-levels risks involves: (1) carrying out a comprehensive desk study of the *study corridor* identified as part of the alternative alignments study, (2) identifying and assessing the initial-risk levels associated with each alignment option in the study corridor, (3) establishing a structured framework to compare the various alignment options in a manner consistent with the Preliminary Design, and (4) taking the decision on the optimum alignment with inherently low-level risks, thus eliminating all those alignment options that will likely not produce a positive outcome. On completion of this selection process, it will be demonstrated that the project based on the selected alignment is both a technically feasible project and an economically productive investment for the project Owner and, ultimately, for the taxpayers who are also the end users of the final product.

A recommended study corridor, which encompasses all feasible alignments considered worthy of further investigation, is normally defined in one of the previous project stages, i.e. before the Preliminary Design stage. The spatial and time-dependent parameters of the study corridor should be wide enough to allow the evaluation of all direct impacts, likely to arise from the construction and operation of the tunnel, and should facilitate a definition of the study area sufficiently robust for the subsequent, more detailed investigations of environmental, and socio-economic impacts of the project's implementation.

The basic information relevant to the selection of the alignment with low-level risks usually includes:

- mapping of the study area, especially the corridor recommended by the previous studies inside which all alignment options lie;
- aerial and satellite photography;
- geological maps, soils data, and reports;
- meteorological and rainfall data and hydrological reports;
- previous designs and reports;
- utilities records, including locations of existing or planned utility lines;
- information about buildings;
- land use information;
- planning proposals, including strategic and regional development plans;
- data on current infrastructure projects;
- socio-economic data such as population, tourism, income, vehicle availability, and ownership;
- traffic data (and passenger data in the case of metro projects);
- construction cost;
- design standards or other relevant standards for the works;
- international data for benchmarking purposes;
- environmental standards and data;
- historical and archaeological data;
- experience of the project promoter and/or owner as well as local construction industry in implementing similar projects.

A list of the available data and data sources for the data categories listed above should be compiled by the project owner, with the help of consultants if required, to facilitate both the choice of the optimum alignment and the subsequent project-development studies.

The specific technique for evaluating the various alignment options is "brainstorming". When doing the brainstorming exercise, mostly through a series of workshops, one should pay particular attention to the following aspects:

- The participation of all Stakeholders should be secured.
- The starting point for investigating the various alignment options will be the conceptual design and/or the feasibility study developed previously, but in selecting

the alignment with level-risks, one shall not be constrained by any of the parameters adopted in earlier studies.

- Initially, one shall not constrain his/her thinking by considering detailed issues of cost, impact, and feasibility that can be expected to come into the final selection process later on, i.e. the analysis of the initial-risk levels can be made mainly in qualitative terms, to be efficient.
- For easy analysis and comparison of the options, the alternative alignments should be drawn using a suitable CAD system, and ideally geo-referenced, with plans and sections being produced at 1:5000 scale or better.
- The advice of a tunnelling expert and system-design expert will be essential during the investigation of the alternative alignments; and the experts can help to ensure that sufficient information is available during the geotechnical desk study to make a meaningful contribution in terms of alignment, depth, station locations (for metro projects), tunnelling methods, outline costing, and value-engineering considerations.
- The presence of historical remains is likely to be a high risk factor for the project during its implementation. Particular emphasis should be placed on this aspect from the earliest stage; therefore, the advice of an archaeological expert will be also essential to the successful selection of the alignment.
- Sufficient environmental information should be available to make a meaningful contribution to the investigation of alternative alignments, and to ensure that subsequent, more detailed studies does not call in to question the basis of the alternative alignments study or the decision to pursue the selected alignment option.

The following subsections shall examine in detail the typical key factors that can strongly influence the selection of the preferred alignment for an urban tunnel, from the perspective of risk analysis and management.

3.2 THE GENERAL LAYOUT OF AN URBAN TUNNEL

The design of an underground infrastructure in a city environment is an activity that requires not only strategic urban planning and urbanisation choices, but also an in-depth risk analysis and economic analysis that could involve years of work and debate.

It is important to decide, during the feasibility studies, the limits within which the basic choices should fall, which, as far as the design of tunnel structures is concerned, can be traced back to the following aspects:

- The choice of the horizontal alignment of reference, understood as the corridor within which the infrastructure is to be placed together with its service utilities and/or connections to different parts of the city.
- The choice of the vertical alignment, related to the intended use of the infrastructure to be constructed, taking into consideration also the geological and geotechnical constraints.
- The integration of the infrastructure with respect to the urban planning and upgrading of the city, which defines the relationship with the inhabited centre.

- The choice, from among the different available technologies, of the most suitable system in relation to the needs for the end users.
- The choice of the configuration of the infrastructure that best responds to the requested functional and safety requirements (e.g. single or double tubes).

Starting from these basic choices, which can be made with the aid of multicriteria type of comparative analysis, the engineering design of the civil structures begins to take shape with the definition of the elements of a general nature, like:

- geological, geotechnical, and hydrogeological studies;
- environmental studies;
- geometry of the alignment;
- design of the characteristic sections;
- assessment of the construction impacts on the urban context, and
- study of solutions to mitigate the consequences.

3.3 ALIGNMENT CONSTRAINTS AND SPECIFIC FUNCTIONAL REQUIREMENTS

3.3.1 General aspects

The environmental context and the tunnel-excavation method influence the definition of the (horizontal and vertical) alignment to a great extent and impose various constraints that are rather different and often more important than those connected to a structure on the surface in an area outside the town. Therefore, the tunnel often becomes the centre of the Project.

The particular aspects that are valid for the different types of infrastructures are examined here and some simple guidelines are given. Some characteristics are, however, common to all alignments of any linear infrastructure constructed in urban areas through mechanized excavation, in particular:

1. The vertical corridor in which the planned infrastructure will be sited is often up to a depth of 30 to 40 m and it is necessary to understand well, and with sufficient reliability, what is present in this corridor as well as in the few meters below it. This understanding can be achieved through a detailed characterization of the underground in archaeological, geological, geotechnical, and structural terms.
2. Large urban centres are often situated in alluvial plains with quite high water tables. Therefore, the routes are almost always destined to be excavated in difficult situations.
3. Regardless of the geological conditions, it is important that the tunnel overburden is chosen to avoid interference with the archaeological layers, the utilities, and foundations of adjacent and overlying facilities. The overburden should, in any case, be at least 1.5–2 times the tunnel excavation diameter to allow an effective control and management of the excavation-face stability. Clearly in this sense, a good geotechnical characterization of the surrounding ground can allow the optimization of the choice of the tunnel vertical alignment, looking for strata with better geotechnical parameters.

4. Irrespective of the construction method adopted, there will inevitably be some degree of disturbance to the normal daily activities of the city. Therefore, one of the key issues in the chain of decisions to be made is to choose the design construction solution(s) that can not only reduce the level of the undesirable disturbance, but also minimise the duration of the avoidable disturbance. In other words, the project should be conceived in a way that will ensure the continuity of the construction production process, minimising all possible interferences that can slow it down.
5. Consequently, priority should be given to using mechanized construction right from the early planning stage of a project and, in particular, to the use of TBMs to construct tunnels, considering that mechanized tunnelling as an industrialised excavation process can achieve remarkable advance rates, thus minimising the duration of construction.
6. However, mechanized excavation, though the safest and most reliable excavation technique, also has remarkable rigidity (geometry of the sections, radius of curvature, programming and site work constraints, etc.) that could limit design and construction choices. Furthermore, the resolution of these problematic situations is a critical response, which is often necessary for functional requirement purposes such as cross connections, enlargements for lay-bys and platforms, large connection chambers, etc.
7. The work sites for mechanized tunnelling are complicated and even cumbersome, representing a very important constraint, which would suggest moving these sites from the city centre to the suburbs. The start-up of the TBM excavation should be made as easy as possible, and, if possible, in a straight, horizontal line, and in areas that are easy to excavate. This will provide an opportunity, with virtually no risks, to calibrate the performance of the TBM using the complex instrumentation on-board, thus facilitating the “learning curve” necessary for a reliable setting up of the excavation procedures to ensure the desired production rates.
8. The rigidity of the TBM imposes lower limits for the radius of curvature of the horizontal and vertical curves, which are a function of: (a) the type of excavation machine, (b) the characteristics of the precast final lining, and (c) the excavation diameter. Such limit-values should be compared with the corresponding limit-values connected to the functionality of the system that has to go through the tunnel. In most cases, the construction limits (for the radius of curvature) imposed by TBMs are the strictest. In general the smaller the curve radius, the bigger the relevant over cutting, i.e. the bigger the risk level for instability.
9. The longitudinal slope of a route can also constitute a design constraint, inducing various problems in the logistic working methods (muck and segments transport, for example), although the slope values in urban infrastructures are not particularly high (normally lower than 3%, exceptionally 5 to 6%).

3.3.2 Transport infrastructures

3.3.2.1 Characteristic-sections

A correct design of the tunnel alignment is clearly the first important step in the development of an underground infrastructure project in an urban environment, as it is the result of the deliberations on a set of very different decision variables, which are sometimes in conflict with each other. For example, the following 3 types of infrastructure will generate different design requirements:

1. A functional and transport type, imposed by the technological system to be adopted.
2. A town-planning type, which is represented by the final location of the stations and/or the entrance and exit points of the tunnel, connections with both existing and other future infrastructures, etc.
3. A construction and technological type, imposed by the excavation method and by the effectiveness of the machines in relation to the dimensions of the excavation.

The parameters of the alignment and the geometry of the sections are defined in conformance with the constraints of a geometric nature, which are generated by the three types of infrastructures.

Figure 3.1 gives a summary of the main functional characteristics of some selected types of transport systems, including the relative range of tunnel diameters.

In defining the gauge section, i.e. the "envelope" of the functionally-required, typical, internal section, the system designer will focus on avoiding any potential intrusions into the "envelope". To pass from the gauge section to the typical excavation section (necessary for choosing the diameter of the TBM), the tunnel designer needs to determine the thickness of the final lining and the annular void between the extrados of the lining and the excavation profile (see Section 5.3 for more details).

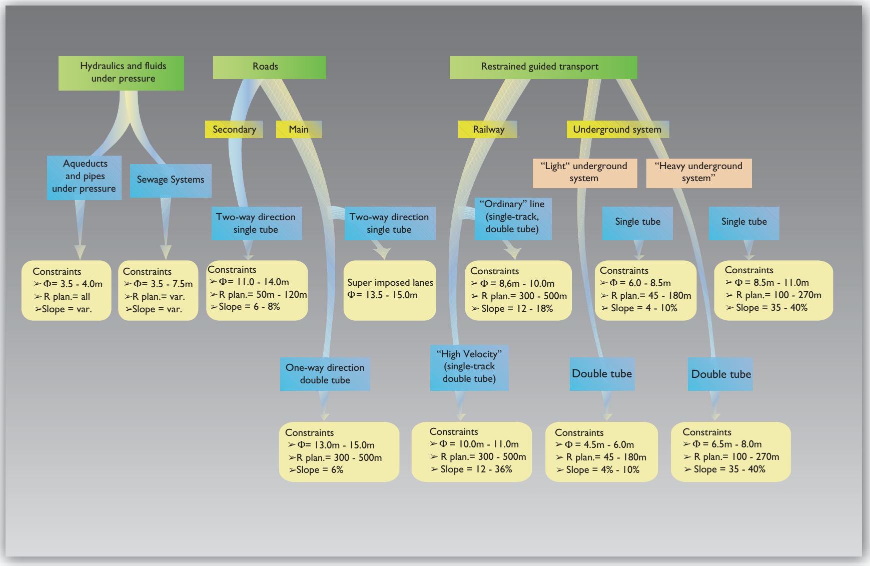


Figure 3.1 Typical transport system and corresponding range of tunnel diameters (current constraints as of 2006).

The thickness of the lining depends on the local conditions and the excavation diameter itself and should never be less than 25–30 cm, while the annular void is a technically unavoidable space that should be minimized in urban tunnelling (10–15 cm).

Mechanized excavation imposes a circular shape, with the only variable being the dimension of the diameter. As shown in Figure 3.1, the dimension of the circular section is extremely variable and depends on the use of the tunnel in the intended transport system (road, railway, or underground metro). In general, the minimum dimensions of the tunnel sections depend on:

- the overall transverse dimensions of the vehicles in the curves;
- construction and loading dissymmetry;
- the inscription modality of the vehicles in the curves;
- the geometrical configuration of the transit or railway tracks in the curves;
- functional arrangements of the furnishings, system, safety, and maintenance;
- clearance and safety margins, and
- possible, future modifications of the track base.

For the *restricted guidance transport systems* the sections also depend on:

- the structure of the vehicles, including the interactions between the rail track and the vehicle, and
- the distance between the rails, which is a function of the velocity (for double track tunnels).

For *roads*, in general, the sections also depend on:

- the width and number of lanes;
- possible enlargement of the radius of curvature, and
- width of the lateral or emergency platforms.

It is, therefore, necessary to guarantee a space that is free of obstacles, from the following points of view:

- Design.
- Maintenance.
- Characteristics of the vehicles.
- Load and distribution of the load on the vehicles.

3.3.2.2 Urban roads and highways

Road tunnels have much larger excavation diameters than railway tunnels because the roads always have at least two lanes and the sidewalks (see Fig. 3.2). The use of a TBM implies the excavation of a larger volume of the ground per meter of tunnel compared to that excavated by the conventional method (and this difference grows with an increase in the diameter, as a function of the square of the diameter). These factors have made mechanized excavation less competitive because of the higher costs

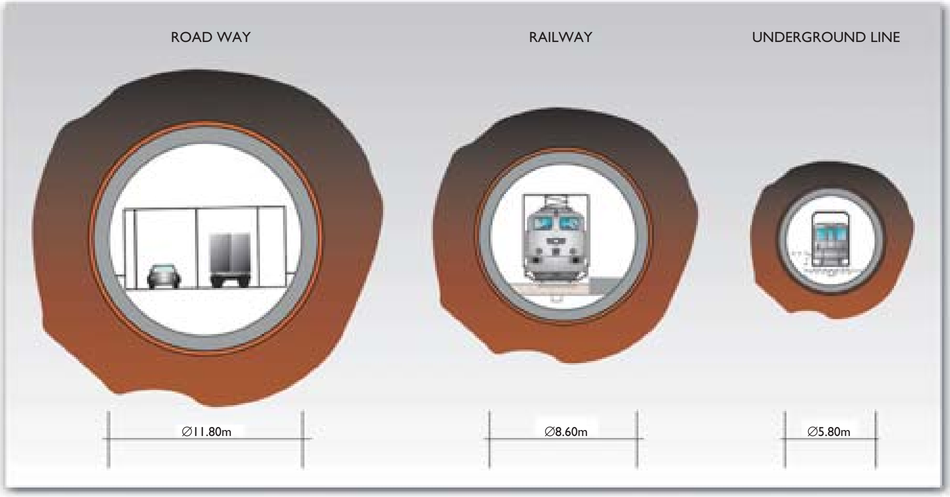


Figure 3.2 Comparative dimensions of urban road tunnel and railway tunnels.

and the difficulty in constructing machines with adequate diameters for the functional requirements for road circulation.

As already mentioned in Section 1, nowadays the ever-increasing requirement of mobility and flow of road traffic in large cities makes it even more necessary to construct the long urban tunnels that act as by-passes to the intense traffic areas or as urban penetrations of large motorway arteries.

The effective application of the new reference norms by the various road network administrations, which have been introduced in almost all countries to increase: (1) performance requirements, (2) safety levels, and (3) need for standardisation of the various types of roads (in the same way as already happened with the railways). These three factors, together with the continual technological improvements of the tunnel boring machines (even for very large sections), ensure that a great number of tunnels can be realized using TBM.

Another aspect to be considered is that, unlike the case of rail transport, the circular section can also be successfully utilized (both at the top and at the bottom) for the installation of the necessary plant and safety works. These works, traditionally requiring less space for road tunnels, are today demanding greater spaces with the improvement in the standards of safety and the necessity of having more plant that are not closely connected to the road: cable ducts, escape routes, ventilation, signals, illumination, safety apparatus, etc.

All this allows a better utilization of the circular section excavated by a TBM, compensating for the greater costs derived from the otherwise larger-than strictly necessary excavation section. The unit costs for the use of TBMs are gradually reducing, even for road tunnels of large diameter in urban environments. For example, it is possible to insert an underground passage for utilities and for maintenance service under the road level, which is easily accessed from the road, thus contributing, through the use of the underground road artery, to the realization of a real "technology corridor".

Even from the point of view of minimum radius of curvature, the need of less and less tortuous routes to render the traffic more fluid and to satisfy visibility requirements, increasingly favours the use of mechanized excavation technologies for urban roads.

In conclusion, it is correct to state that, apart from the remarkable technical difficulties and the high initial investment costs, a road tunnel is more demanding for urban underground space than any other underground infrastructure; and its construction represents a prime mover of great technology innovation and urban transformation, not only in the road traffic field, but also in an urban environment and implementation of the services in the city.

3.3.2.3 Ordinary and high velocity railway lines

The use of mechanized excavation is also becoming more common for the construction of railway tunnels in urban areas for many reasons: to move the existing lines underground, to create direct underground fright shipping lines and rapid passenger railway-links, to double the already existing tracks and, more generally, to improve the already existing lines, and to realize new and direct High Speed links to the urban centres.

One of the main constraints in selecting a mechanized solution is the radius of curvature of the tunnel alignment; and the minimum radius varies according to the type of line, the design speed, and the vehicle-fitting conditions at the curve. Even for low design speeds (60–80 km/h, the curvature radii do not usually fall below 250 m. This value is approaching the limit for the construction of single-track tunnels realized by a TBM (8 to 9-m diameter) and becomes difficult to obtain for double-track tunnels (13 to 14-m diameter).

As far as the railway environment is concerned, each country has standardized the criteria for defining the geometrical and functional characteristics of the possible, typical sections for the different types of railway systems.

In any case, for the definition of the internal typical section, it is necessary to take into consideration the aerodynamic, maintenance, and safety criteria. Another important aspect to be considered is the railway limit profile itself. Finally, it is necessary to consider construction tolerances of a tunnel, more importantly for mechanized excavation than for conventional excavation, because the rail transport system is a rigid-guided transport system.

A typical section for a low-speed, single-track, railway tunnel to be constructed by mechanized excavation is shown in Figure 3.3. The internal diameter of the section is about 8 m. Today, the dimension of this simple typical section is not constant across various countries and, in some situations, the difference may be quite significant. Consequently, the international rail organisations are making great efforts towards standardization, and there is an increasing tendency to adopt single-track parallel twin-tunnels for safety reasons, avoiding the involvement of both tracks in the case of mishaps, accidents, or fires.

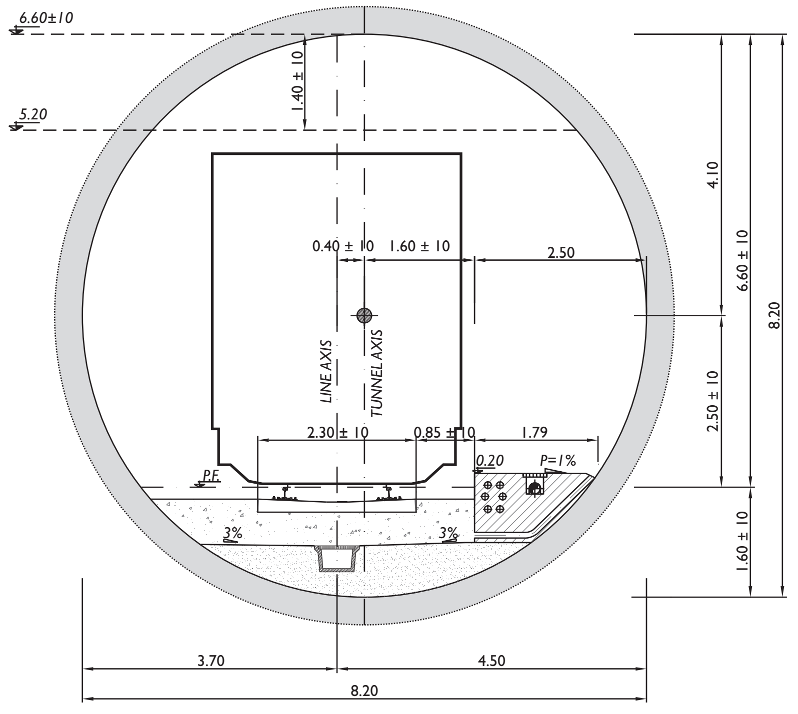


Figure 3.3 Example of a typical internal section of a tunnel for a single-track railway (Italferr, 2004).

3.3.2.4 Underground railway lines

The first underground lines were built in the second half of the nineteenth century as continuations of out-of-town railway lines. These transport systems had the role of:

- shortening the distance between the suburban areas and the city centres, according to the concept that distance should not be seen in "spatial" terms but rather in "temporal" terms;
- boosting the transport capacity along already particularly loaded corridors, and
- creating an integration between the railway networks (at a national level) and the urban transport networks.

According to the functional and dimensional standards, underground railway systems, also called metro system, are usually classified as "*heavy*" or "*light*".

The typical set of system-design parameters, and the relative gauge values characterizing the two systems, are given in Table 3.1, while the rolling stock characteristics of the two systems are compared in Table 3.2.

Unlike the road and railway infrastructures for which codified geometrical configurations have been established by the different managing bodies, the internal section dimensions of the metro systems basically depend on the transversal dimensions of the vehicles adopted, which can vary greatly according to the types and models that exist

Table 3.1 Summary of typical system-design parameters.

System characteristics	*Light railway*	*Traditional underground system on rails*
Capacity of the vehicle/train (passengers)	250–500	500–2000
Typical capacity of the system (passengers/h)	1000–30,000	10,000–60,000
Commercial speed (km/h)	15–45	25–60
Maximum speed (km/h)	70–90	70–100
Width (double track) (m)	5–7.5	5–8.0
Maximum frequency (vehicles/h)	40–60	20–40
Distance between stations (m)	400–600	900–1800
Minimum radius (m)	10–50	150–300
Maximum surmountable slope (%)	5.0–15.0	3.0–5.0

Table 3.2 Comparison of rolling stock characteristics (Malavasi, 2005).

Category of underground line	*Level of automation*	*Rolling surface*	*Example*
LIGHT			
Capacity: <=15–20,000 passengers/h per directions	Partially Automatic	Rail	Hannover (all lines)
Vehicle capacity: <=300–400 places/train		Wheel	–
Length of train <50–60 m	Completely Automatic	Rail	Vancouver (Sky Train)
		Wheel	Rennes (VAL)
HEAVY			
Capacity:>15–20,000 passengers/h per directions Vehicle capacity:>300–400 places/train Length of train>50–60 m	Partially Automatic	Rail	Athens (lines 2 & 3) Helsinki (all lines) Osaka (line 7) Washington DC (all lines)
		Wheel	–
	Completely Automatic	Rail	Nurenberg (line 3)
		Wheel	Paris (METEOR)

on the market. The worldwide mean value of vehicle dimensions for the traditional, heavy, underground systems is about 2.65 m, while for the light underground systems, the mean dimension is generally smaller, down to 2.08 m for the VAL system (Orly, Lille and Rennes in France and Turin in Italy).

3.3.2.5 Configuration of the railway line and the stations

The choice of line configuration for a metro system is indispensably connected to the choice of the station platforms. This is of strategic importance for the overall costs of the structure due to the important reciprocal constraints between the construction of the tunnel and the construction of the stations, in terms of the design contents and the planning of the construction works.

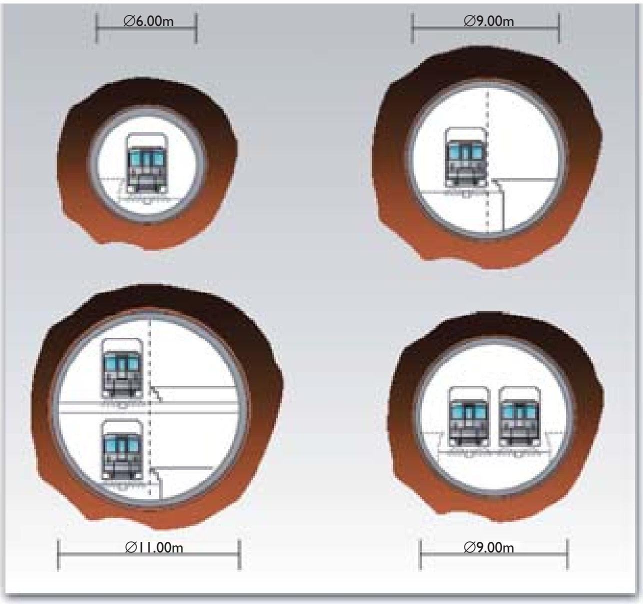


Figure 3.4 Configuration of a running tunnel and station platforms.

Depending on the choice of the running tunnel configuration (double-track, single tunnel vs. single-track, twin tunnels), there can be six types of station configurations. Four examples of such configurations are presented in Figure 3.4, which have been employed successfully in different parts of the world. It is also noted that sometimes, even for the same metro line, different configurations have been adopted for various reasons. As can be seen in the same figure, the final diameter of the tunnel (thus also that of the TBM) is a function of the choice of the platform configuration.

The selection of a suitable running tunnel-station configuration is no doubt a basic design decision that has to be made in the early stages (usually in the technical feasibility) of development of a new metro line, and it will subsequently condition the detailed definition of the entire alignment. This decision can be made with the application of a set of decision tools like risk analysis, multi-criteria analysis, decision trees, and cost-benefit analysis. In any case, the decision criteria should include, among the other aspects, the least occupation of the underground space and least disturbance to the environment.

3.3.2.6 The double-track, single-tube configuration: a typical solution with relatively low-level risks

The single-tube configuration, under a given set of boundary conditions (geology, archaeology, surface constraints, etc.), can offer certain advantages for both the infrastructure and the railway installations:

- The greater freedom of constructing just one tube makes it possible to better choose the route under the city streets.

- With a single tube, the communication between the tracks is immediate, without the need of connecting tunnels which are difficult to construct and can have a negative influence on the regular ventilation flows and on the handling of emergency situations.
- The double-track, single-tube solution, compared to the twin-tube solution, will likely disturb a smaller area of the ground surface. Furthermore, the risk connected to the possibility of geotechnical interference between the two tubes of the line is removed.
- It is easier to select the route in a way 50 as to facilitate the location of the stations in areas which are less critical for the city, for example, at the main squares, thus providing space for worksites for underground construction.
- The number of buildings that are passed under by the running tunnel is generally reduced.
- It can allow for the running-tunnel continuity as the tunnel can cross the stations directly.

However, this solution is not a perfect one and it has also some disadvantages:

- The tunnel diameter shall be relatively large and thus may impose a deeper vertical alignment for stability reasons.
- It will be necessary to construct deeper and, possibly, more expensive stations.
- An accident of a train on one track may cause the entire line to stop its service.

Consequently it is necessary to conduct a risk analysis for each feasible solution, even though such analysis may be preliminary, and to determine a system-wise optimal solution or combination of solutions with decision aids, as for example the DAT (see Section 2.5).

3.4 CONSTRAINTS AND PECULIAR CHARACTERISTICS OF THE URBAN ENVIRONMENT RELEVANT TO THE SELECTION OF A TUNNEL ROUTE

When selecting the tunnel route for any urban tunnel project and subsequently defining the horizontal and vertical alignment, the constraints may cause hazards for the tunnel design and construction. If these are not handled properly at the early project development stage, there is a series of constraints that should be considered for a correct and accurate design. The final route design is usually the product of a compromise between meeting the functional and technological requirements and mitigating or avoiding the potential interferences related to the constraints.

The following subsections discuss the characteristics of a selected group of typical urban constraints and provide some comments on how to deal with them.

3.4.1 Buildings and infrastructures

Buildings are surely the common type of interferences that can give the greatest problems when defining the horizontal and vertical alignments of an urban tunnel.

To minimize the potential risk-levels associated with the buildings, it is necessary to conduct a specific Building Condition Survey (BCS) and subsequently a Building Risk Assessment (BRA). The detailed procedures for BCS and BRA are discussed in Section 5.1.

At the early alignment definition stage, it is usually sufficient to gather all relevant information through a careful desktop study. The key information to be gathered about each building in the corridor should include:

- the destined use;
- the structural characteristics and its state of conservation;
- the type and the depth of the foundations;
- the presence or not of any floors in the basement;
- the geotechnical characteristics of the ground below the building foundations.

Such data should be presented on thematic maps and/or profiles both for easy visualisation of the data themselves and for quick appreciation of their interferences with the tunnel alignment, including the various alternatives.

There are elements, such as deep foundations and floors in the basement, representing physical obstacles that some times could be avoided only through modifications of the vertical alignment. The special use and the historical/architectural importance of some particular buildings can become difficult obstacles in terms of forcing the Owner, or the public, to accept that the tunnel has to underpass these particular structures. In this sense, it could be more complex, from a social point of view, to cross under a hospital or a thirteenth-century church than to pass under a building of ten floors.

The main danger of direct interference is related to the presence of deep foundations on piles. In this case, it is important to determine the geometric configuration of the piled foundation as precisely as possible and to find in the first instance a horizontal-vertical alignment.

Other important interferences are frequently linked by existing infrastructures both above and below the ground surface, such as:

- underpasses and large road arteries;
- underground system lines;
- railway lines;
- car parks;
- water supply pipes/tunnels;
- sewers.

Such interferences constitute a group of rigid constraints for the definition of the tunnel route and they can be resolved by shifting the new tunnel to a different level (see, for example, Fig. 3.5).

The traffic congestion of large cities has led to an ever-increasing use of underground car parks, which can create remarkable problems for the excavation of an urban tunnel both because of their depth (for example, car parks with more than three floors or silos-shaped car parks) and because of the retaining walls that are often anchored by many rows of tiebacks. The latter can effectively enlarge the area of interference of the existing structure (see Fig. 3.6).

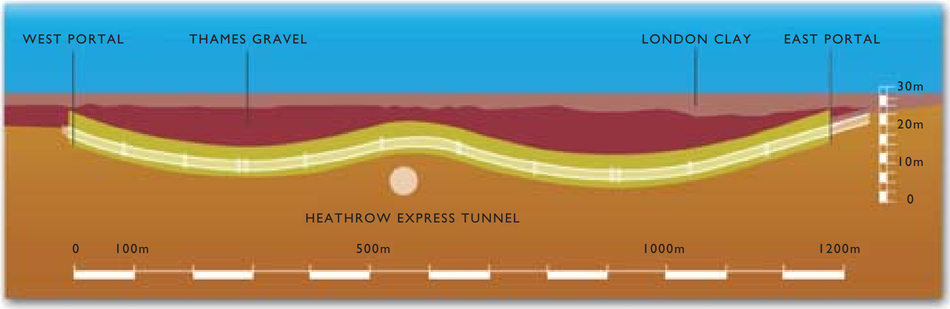


Figure 3.5 The Road Tunnel in Heathrow: elevation variation foreseen to pass over the Heathrow Express Tunnel.

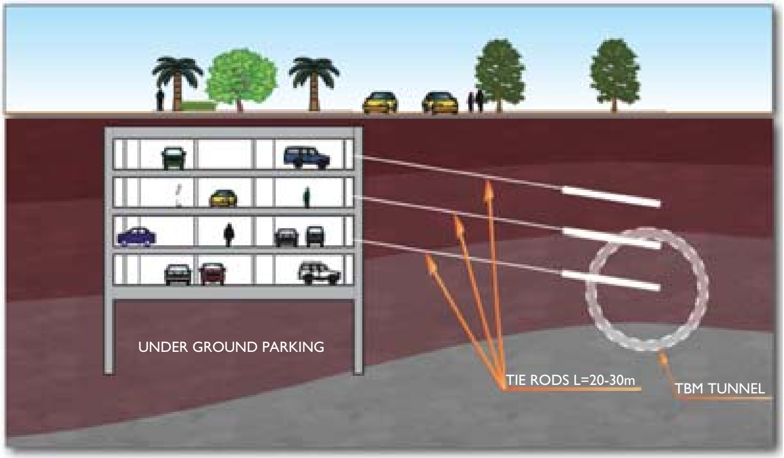


Figure 3.6 Possible interference between an underground car park and a tunnel to be constructed.

In this case, it is of fundamental importance to identify and avoid these interferences in the early design stage, because mechanized excavation can not easily deal with an unforeseen obstacle. Facing this kind of interferences during construction will inevitably lead to construction delays, a risk that should be avoided at the planning stage.

Very often, the requirements of the route impose the necessity for the tunnel alignment to pass under infrastructures or underground car parks with reduced clearance: in these cases it is only possible to pass under the structures if measures for protecting the existing structures have been implemented using, for example, ground treatment (Fig. 3.7) or underpinning of the structures themselves (Fig. 3.8).

3.4.2 Utilities

Utilities are the public service networks that have been placed underground in an urban environment (see Section 5.1). Significant utilities include: free-surface channels

Figure 3.7 Ground treatment through grouting to allow a TBM to pass under an underground car park during the construction of a Metro Line.

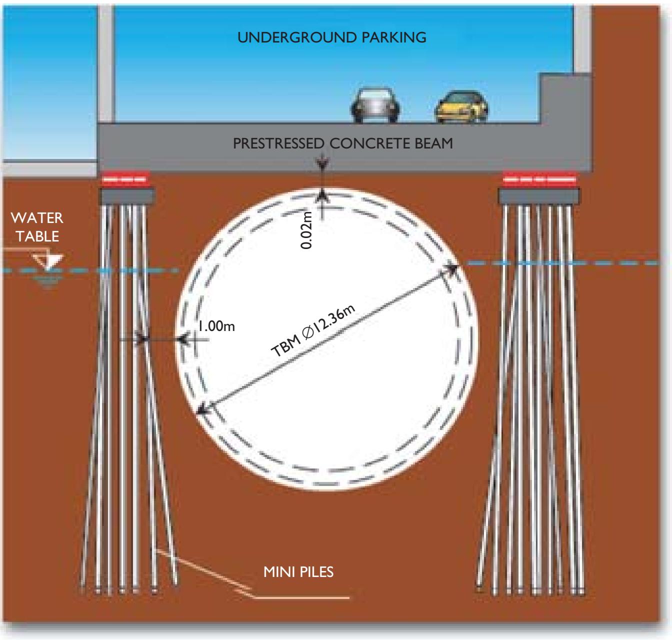


Figure 3.8 The Zimmerberg Tunnel in Zurich – Reinforcement through underpinning of an underground car park underpassed by the TBM excavation (Kovari *et al.*, 2004).

(sewage and water ducts), pressure pipes (gas, district heating, aqueducts), electricity networks, and telephone lines.

The construction of a tunnel and the related access to the stations (in the case of a metro line) may directly interfere with, or indirectly impact on, the utilities. In some cases, even the drilling of site investigation boreholes or ground treatment holes may directly hit the utilities, and cause damage. Thus, as in the case of buildings, it is necessary to make specific utility surveys and conduct subsequent utility-risk analysis, right from the early alignment selection stage, in order to minimize the potential risk levels associated with the selected alignment.

To gather the necessary, basic information, the first step is to do cartographic research and record information about all the networks that are present in the area of interest. Notably, many cities are becoming equipped with centralised and computerized archives on updated and easily accessible WEB/GIS platforms, which are able to very quickly supply the basic information necessary for developing the first design hypothesis for an urban tunnel.

3.4.3 Existing historic structures

Interference with the pre-existing important structures of historic value is a problem that not only concerns large archaeological cities like Rome or Athens, but also those cities where the sub-soils have not been studied well or are not well-known from an archaeological point of view.

When referring to historical-archaeological finds, it is necessary to consider not only the remains of ancient vestige, but also the underground passages, cisterns, and abandoned wells. The last three are potentially the most critical for constructing a tunnel by TBM, similar to the case of a sewage system which can cause a rapid and uncontrolled emptying of the plenum of the TBM. This hazard can impact the stability not only of the tunnel, but also of the surrounding ground as well as the structures on the surface.

Where the interferences are assessed before construction, such as in the case of the Cittadella Tunnels during the excavation of the underground system in Turin (see Fig. 3.9), it is possible to intervene even in a simple, but very efficient way, by reinforcing, or temporarily filling-in all the underground passages in order to guarantee safety for both the existing structure and the tunnel under construction.

However, in the early planning stage of an urban project, it is most efficient to work closely with the authorities for conservation of the archaeology and structures of historic value and to select the tunnel alignment, both horizontal and vertical, that generates potentially the least interference with such features.

3.4.4 Involvement of the citizens in development of the project

Construction of a large tunnel in an urban environment can change the very structure of a city, at least during the execution of the works. In this case, it is common for many local inhabitants to take the “NIMBY” (Not In My Back Yard) attitude, which is largely due to a lack of information. To prevent this attitude, widespread

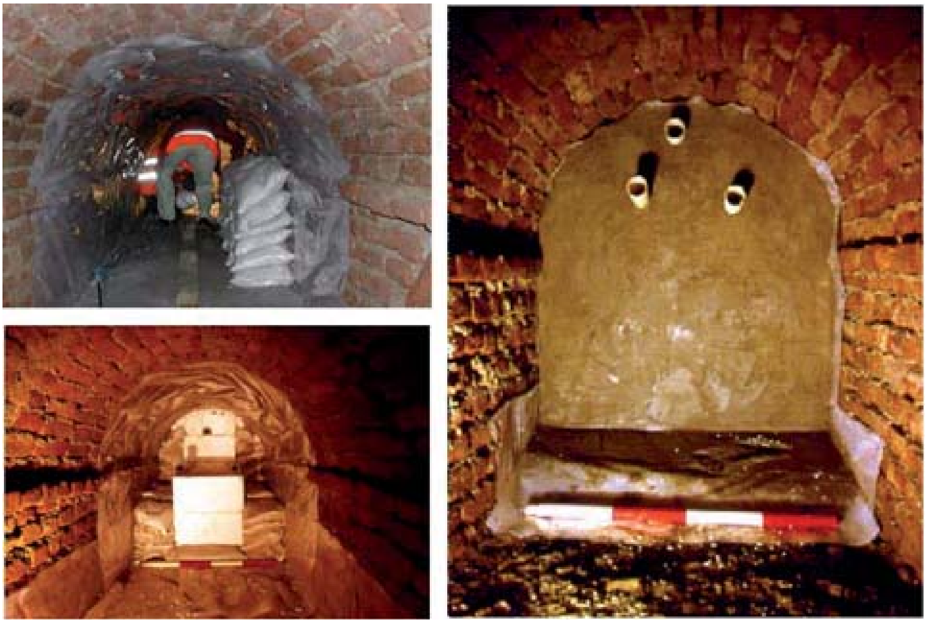


Figure 3.9 Protection intervention on the Cittadella Tunnel during the construction of the underground system in Turin.

and timely communication with the citizens, both during the design stage and later on during the construction stage, are necessary. Failure to provide this communication can be a potential hazard to the project and might cause delays and cost overruns.

Nowadays, it is no longer possible to construct structures in densely populated areas without listening to the requirements of those who live in the areas that can be potentially impacted by the work. It is obviously not possible to satisfy each single requirement, but it is surely possible to reach a correct compromise between those who have to perform the work and those who have to put up with the disturbance that this work causes. Two clear examples of damage to the existing structures, caused by tunnel excavation in an urban area, are provided in Figure 3.10.

Indeed, the definition of a tunnel route is often the sum of the contributions of the local communities and of the requirements of each neighbourhood, which are not always known in the first design stages. Best efforts should be made to ensure that the number of people inevitably, and negatively, affected is minimum.

Once an optimum alignment has been determined, it is necessary to assess the associated level of residual risks and to adopt consequently a Risk Management Plan, (RMP - see Section 2), to manage properly the identified residual risks. The relevant aspects of the RMP should be communicated to the public concerned, who should be assured that adequate countermeasures have been prepared to safeguard even those properties subjected to low levels of risks.

Figure 3.10 Damage to existing structures during tunnel excavation in an urban area (Left: Lane Cove Tunnel, Sydney; Right: Heathrow Express, London).

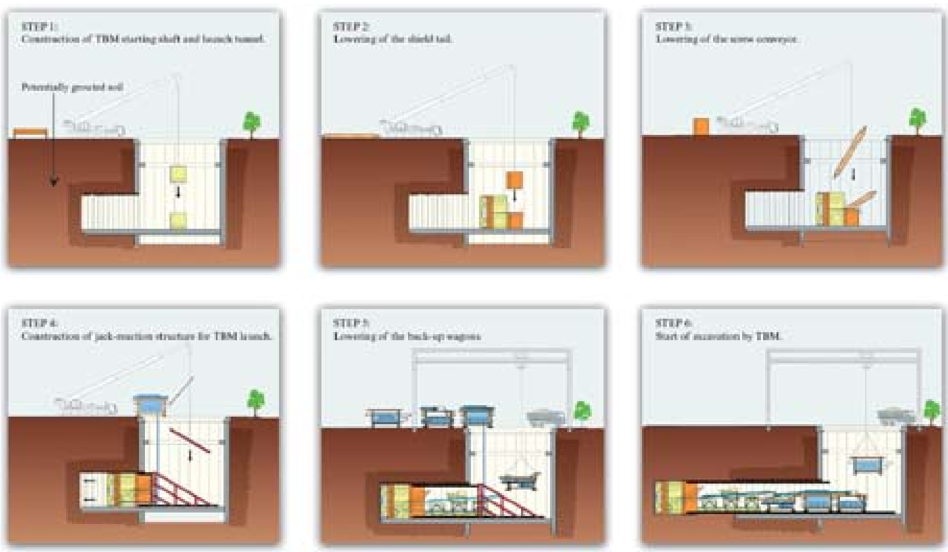


Figure 3.11 Typical steps for starting the TBM excavation from a launching shaft.

3.5 STARTING AND ARRIVAL POINTS, SHIELD LAUNCHING AND RECEIVING SHAFTS, LOGISTIC WORK SITES

The selection of an optimal urban tunnel alignment should consider the location of the tunnel starting and arrival points, the location for the TBM-launching and receiving shafts, as well as the huge space required for organization of logistic worksites to support the industrialized construction process. In fact, it is always necessary to consider the location of the TBM launching shafts in well defined areas, outside the historic centre and connected to a good road system.

The size of the areas necessary for the logistic sites can vary considerably according to the selected type of TBM. The differences are connected with the different methods used for the management of mucking for the two types of city machines, EPB or Slurry Shield (SS)/Hydroshield (HS).

In the case where an SS or HS is used, the principal constraint is to find the necessary spaces for the installation of the slurry-separation plant. The dimensions of this plant vary in function of the required capacity (for example, for a mean capacity of 1000–1500 m^3/h, the necessary area for this kind of plant could be 2500–3000 m^2).

In the case of an EPB, the problem of slurry separation, for reuse as bentonite, does not exist. However, in order to avoid the problem of the muck being too fluid, which makes it difficult to transport to the disposal site, it could be sometime useful to foresee a muck-washing plant at the job site.

In the case of a metro line, the TBM-launching and dismantling chambers are very often incorporated inside the head stations (of the stretch to be excavated by the TBM) in order to minimize occupation of the surface space. In other situations, it is necessary to construct specific shafts, whose internal geometries are obviously conditioned by the dimensions of the TBM and the spaces necessary for the assembly and dismantling operations.

Finally and also most importantly, the selection of an optimum tunnel alignment should consider the special needs for the design of the very first stretch to be excavated by the chosen type of TBM. This initial stretch is usually 200 to 250-m long and constitutes the so-called "learning curve" section, where all the TBM crew will obtain a practical learning of the excavation process and control of the TBM.

The excavation start usually occurs in minimum overburden conditions and often without the preventive measures and complete mounting of the technological trains, because of the limited space for manoeuvring. The initial stretch is, therefore, subject to intrinsic operational difficulties, which, together with the lack of specific experience and the inevitable initial experimentation of the machine, can be the cause of a series of risk, and possibly of a long slow down or hold up in the production programme of the excavation.

In fact, the experience gained has shown that most accidents (over-excavation, collapses, and damage to pre-existing structures) and delays connected to mechanized excavation in cities, are concentrated in the learning-curve section.

Therefore, particular attention should be paid to the selection of the alignment of the initial stretch, opting for the alternative with inherently low-level risks, and especially avoiding to pass under any sensitive points of interference (buildings, utilities, etc.). Wherever surface constraints limit the potential for a change in the design of the route, it becomes indispensable to carry out systematic ground-treatment beforehand, which should encompass the entire length of the "learning curve" section, that is, till the TBM reaches the "regime excavation conditions" in which all the procedures have been tested and made operatively efficient.

3.6 THE RELATIONSHIP BETWEEN THE EXCAVATION OF THE RUNNING TUNNEL BY TBM AND THE CONSTRUCTION OF THE STATIONS

In urban railway tunnels, the relationship and reciprocal interactions between the construction of the stations and the excavation of the running tunnel by a TBM, takes on a particular relevance. A station can be the starting point or finishing end (or shaft) of a TBM and, therefore, it becomes important to respect the considerations expressed in the previous section concerning the logistic sites for TBMs.

Apart from the terminal stations that can act as starting and arrival points of the mechanized excavation, all the intermediate stations are also subjected to the passage of the TBM. It is, therefore, important to pay particular attention to the aspects connected to work planning, in relation to the two different ways of passing the stations by the TBM:

- "void" crossing, where the station has already been built, at least as far as the excavation and the main structures are concerned;
- "full" crossing, where the station volume has yet to be excavated.

In general, the most common method for TBM crossing of stations is that of void crossing and it can usually be foreseen in the design stage of an underground line. However, the experience and the technological progress of recent years in mechanized excavation have shown that, once the excavation operations are successfully under way and regime conditions have been reached with full activation of all the inspection procedures, average tunnel advance that can be obtained, in any geological context, can be very high. In some instances, the construction of the stations can become more critical than the excavation of the tunnel for an underground line.

In fact, it can happen that, during the construction of a line, the tunnel construction is ahead of construction of the next station. In this case, whenever the design has specifically foreseen void crossing of the station, slowing down is necessary. In the worst case, the TBM advance has to be stopped in front of a station because it is not completely equipped for the TBM to pass through.

Such an eventuality should be avoided at all costs, not only because of the risks connected to the consequent delays in the work programme, but also because of more general problems of making the tunnel safe: a stopped TBM, perhaps under a building or even just under a road subject to road traffic, is evidently more dangerous than a functioning and operative TBM.

Experience has shown that the risk of delay in constructing a station by a conventional excavation method can be significantly higher than that by the cut & cover method and thus a mined station is more likely to cause delays for the TBM to pass through it. Therefore, in selecting the alignment, in terms of the running tunnel and stations type combinations, priority should always be given to cut and cover stations in order to minimise the related construction risks.

3.7 CONCLUSIONS

The comprehensive analysis of the typical key factors, which could have a significant influence on the choice of the preferred alignment, may serve as a checklist; and by adapting this checklist to the specific project in hand, it is possible to effectively establish the necessary basis for making the preferred choice with (virtually) only low-level risks.

Subsequently, a broadly based multi-criteria filtering exercise is normally carried out, to eliminate the alignment options that have been identified as not meriting further detailed evaluation and investigation, because they are dominated by hazards entailing high-level risks. As briefly mentioned in Section 2, the semi-quantitative multi-criteria analysis can be further substantiated using quantitative, decision tools such as DAT. In any case, the main reasons for the exclusion of each excluded option should be well documented, transparent and easily traceable. Acceptable reasons for exclusion may include excessive negative impacts of hazards associated with high-level risks in terms of policy, environment, construction cost and time, life-cycle costs, and operational factors.

In any case, it is essential to carry out a comprehensive desk study of the study corridor recommended by previous stages of the project development for the study of alternative alignments. The objectives of this desk study will be:

- To provide a preliminary indication of soil, rock, hydrology, hydrogeological and seismic characteristics of the corridor based on available information, aerial or satellite photography, and site inspection.
- To provide geotechnical input to the process of refining, evaluating and costing alternative alignments.
- To establish both the strategy and the required scope of the subsequent site investigations, aimed at understanding fully the urban environment in which the tunnel will be built.

All these objectives will help to ensure that the level of investigations undertaken for the alignment-option study are adequate and that a subsequent, more detailed investigation, does not question the conceptual basis for the decision regarding the selected alignment.

Finally, with the comprehensive analysis presented in this section, it is possible to confirm that "Be wise a priori" is not merely a philosophy, but also a viable and a "must-be-adopted" technical approach for developing a tunnelling project in a city.

4

The primary responses to the initial risks: a "city machine" and its essential characteristics

4.1 PRINCIPLES FOR MAKING THE MACRO CHOICES

As mentioned in the Executive Summary, in Section 1.5, and deeply analyzed in Section 2, one of the main focuses of this book is on the use of a design and construction approach that minimizes the risks during tunnelling in urban areas. According to this approach, the first step to developing an urban tunnel is to begin with the risk analysis for the project in question, starting from the early planning and design phases and considering all potential hazards, especially those relevant to the construction.

It is believed that the correct selection of the construction method shall constitute a primary measure for reducing the identified initial-risk levels.

It has now become a mandatory requirement that the construction of underground facilities must protect the environment through which they pass by assuring that the adjacent ground, facilities, and surrounding environment are not adversely affected. Thus, excellent construction techniques that result in a minimum of disruption to the public are essential to the positive image of the underground industry (Parker, 2006b). This is especially true for tunnel excavation in the urban environment, where particular methods and procedures are required to face the main problems, constraints, and challenges listed below:

- Subsidence phenomena on the surface (roads or other facilities) and their limitations.
- Interference with utilities.
- Interference with foundations of pre-existing structures.
- Muck disposal, with regard to its consistency (e.g. excessively liquid) and its content of polluting substances (derived from the face-conditioning products).
- Use of external monitoring (on buildings and on the ground surface and in the ground) to be related with the control parameters of the excavation (face pressure, weight of extracted material, and volume and pressure of the backfill grouting):
  - type of instruments to be installed in relation to the expected deformation response of the ground to the tunnel excavation;
  - positioning, spacing, setting the warning and alarm levels, and defining the reading-frequency of the instruments;
  - type of data acquisition (manual and/or automatic) and feedback time of monitoring data, to enable cross-controls with the TBM performance data and the back-analysis.

- Need of local improvement (if any) of the ground, prior to TBM excavation.
- Need of further local improvement (if any) of the ground, concurrent with the TBM advance (local ground treatment and/or secondary grouting of the backfill, from the pre-cast lining). Such treatments executed from the surface could be preferable to those from inside the tunnel, in case of low cover, to avoid interfering with the production cycle. But the surface treatments have the drawback of creating interference and nuisance to the urban environment.
- Need of spaces for the worksites, particularly in relation to the dimensions of the TBM to be brought into, and/or extracted from, the tunnel.

Mechanized excavation, which is discussed in this Section, face the above-mentioned challenges and minimise the risks, both theoretically and practically, and thus should be the preferred choice for tunnelling in city environments.

If the mechanized solution is selected, the next step from the perspective of Risk Management, would be the choice of a "special" or "ideal" machine from the numerous types of available machines, to cope with the potential problems in the given project. This could be, again, regarded as an effective "primary mitigation measure". However, it is evident that the choice of a particular machine for a given project has serious implications in respect of the resulting (or residual) risk levels. Obviously the risk of instability is highest when excavating without particular precautions. The risk-level can be reduced by choosing a TBM with face support, but the reduction may not always achieve an acceptable level of risk. It is important to remember that no excavation method is inherently risk-free, and the mechanized method is not an exception. Thus, as for the initial risks, the residual risks must also be assessed and managed.

Furthermore, it should be recognized that in choosing to use a mechanized solution there is also a "price to pay" because of several disadvantages, including:

- The assembly and start-up of the machine, requiring the availability of large spaces on the surface and underground, which are not always easy to find in a city environment.
- The fixed, circular shape of the excavating section of the machine is not always optimum with respect to the requirements and/or constraints of the underground space.
- The potential over-dimensioning of the tunnel lining, responding to the necessity that it should also support the forces derived from the construction (for example, the longitudinal cylinders' thrust on the lining segments to propel a shielded TBM).
- The strict requirement for a detailed knowledge of the geotechnical characteristics of the ground to be excavated: location, geometry, and structural characteristics of the potential interferences (either on the surface or underground) and in particular, all the formations present at the tunnel level, which might have historically caused accidents with serious consequences.

The collective experience of the tunnelling industry (see the recent report by the Closed-Face Working Group of BTS, 2005, and the ITA Open Session on Risk Management, 2006, in Seoul) has demonstrated that selecting the correct method, including the correct and specific machine, is necessary, but not always sufficient, condition for success. Many other elements are still required to guarantee the full success of a mechanized excavation; and the additional key elements include:

- A detailed design of the machine itself is necessary, together with correct dimensioning of the lining and careful attention to the overall operational logistics of the machine. A wrong approach to design or construction may produce unacceptable conditions such as excessive surface settlements, limited advance velocity, damage to the prefabricated concrete segments, and inadequate water-tightness of the structure.
- Experience and know-how are crucial for an efficient, technically valid, and economically effective application of the machine. In fact, it is necessary that all of the stakeholders participate in the project to the best of their capabilities:
  - The Owner needs to develop and use a list of criteria for the selection of the Contractor and the Designer;
  - The Designer must arrange all the known elements necessary for a correct work development and for doing adequate investigations. The Designer must also be aware of the state-of-the-art machines for tunnel excavation, and collaborate with the constructors in selecting the best solutions for the identified problems;
  - The machine Manufacturer must efficiently combine the needs of mechanical engineering with the aspects related to tunnel construction. A constant exchange of experience between civil and mechanical engineers is needed, together with the continuously maturing field experience;
  - The Contractor must always operate the selected machine in a careful and rigorous manner, not missing the details of any situation that must be managed through a continuous excavation-control for achieving a correct and secure advance of the tunnel;
  - The Contractor must utilize only skilled and well-trained personnel.
- It should be highlighted that, since methods of excavation or TBMs endowed with "magic" powers do not exist, any type of TBM requires a strict system to control its use, which has to respect relevant procedures and work instructions. Such a "method", i.e. the application of a strict control system in the TBM use, employed as a "secondary mitigation measure", shall enable an actual "minimization" of the residual construction risks to the point of making them acceptable. Thus, there can be an alternative definition of the "true" residual risk (see Sections 3 and 6), as the first level of remaining risk acceptable by those who have the decisional powers to absorb and/or manage it (Employer, Contractor, Designer, or a "risk management committee" purposely instituted and comprised of all the Stakeholders previously mentioned).
- If the residual risk-level is still too high (i.e. non acceptable), additional mitigation measures must be implemented (for example, ground treatments).

To sum up the fundamental concepts stated above, the principles for making the macro choice of the primary responses to the identified risks should include:

- The human presence from all possible sources should guide the design-construction choice when tunnelling in a city environment.

- If the potential damage to persons and/or properties involving the life of people has such a high impact on the risk evaluation that, even if the probability of their occurrence should be low, the related risk level could remain unacceptable, thus secondary and/or additional mitigation measures should be foreseen.
- The knowledge about the ground (being always the main source of hazards) should guide the preliminary design-construction choice, but this knowledge should always be checked and confirmed on site, through exploration and geotechnical monitoring during construction and interpretation of the data obtained.
- The choice of the correct excavation method, based on the information derived from the site investigations and interpretation of the results in the light of Risk Management, constitutes a "Primary Response" to the identified initial risks.
- If the method selected is a mechanized solution, it is usually necessary to foresee the use of machines capable of applying a pressurized support to the excavation face.
- The remaining risks after adopting the primary response should be analyzed in the same manner employed for the initial risks and managed by implementing secondary mitigation measures. In particular, a rigorous control-system of the excavation process should be adopted.

4.2 THE COMMON SOLUTION: A "CITY MACHINE"

Following the principles discussed in the previous section, it is useful to consider the tunnel boring machines for application in city environments as a special category of machines, the so-called "City Machine", with particular attention being paid to the corresponding requirements (or specifications).

In fact, there is, inevitably and always, a certain degree of uncertainty involved in a given urban tunnel project, no matter what the geological, hydrogeological, geotechnical context is. The accuracy of the preliminary site investigations and the constant update of these investigations, just like the review of the geotechnical characterization of the ground during the course of construction, shall never be sufficient for resetting to zero the levels of risks linked to possible excessive settlements on the ground surface and/or to collapses of the excavation front (see Section 2). Consequently, in order to operate under the conditions of maximum safety, minimizing the principal risks, through a rigorous management of the project risks, it is necessary that (1) a mechanized excavation in a city environment be done with, only and exclusively, a machine capable of providing the necessary face-support pressure, and (2) the machine be utilized in "close mode" for all tunnel stretches involving pre-existing structures which could, in any way, be related to the presence of human beings.

It should be pointed out that the choice between open and closed mode is not restricted to EPB shielded machines (see Sections 4.3 and 4.4 for more details). As a matter of fact, the definition of "open mode" refers to the use of a machine equipped with "face-support" facilities, but actually operated without applying a support pressure to the excavation face, for the excavation of a certain stretch of tunnel. With an EPB shield this excavation mode is obtained through maintaining the plenum either totally, or partially empty, without applying pressure and controlling it at the upper section below the tunnel roof. In an analogous manner, the same mode can be obtained with a Hydroshield through maintaining the level of bentonite slurry at the same height in both the front and the rear compartments of the plenum, without pressurizing the air in the upper part of the plenum. In this case the bentonite slurry that is present in the lower part of the plenum has only the function of conditioning the muck and facilitating its removal.

It is obvious that such a way of operating a TBM does not offer any possible interventions in terms of either face support or control of surface settlements. Moreover, the information obtained under the "open mode" conditions does not provide any useful indication of the ultimate control of the extracted quantities, being impossible to evaluate the quantity of the material that (theoretically) enters the plenum, by only measuring the quantity of the material exiting from a chamber whose degree of fullness is unknown.

Not surprisingly, looking back at the tunnelling projects already realized in city environments, one can observe how a serious accident or a series of minor accidents always led to a subsequent period of no-production. However, by just rethinking and studying the whole construction process, no further accidents occurred in the following period of completing the works. To obtain such good results it was not necessary to make radical changes in the design or the construction methodology. Instead, a rigorous application of the operating procedures designed for safety was the optimum solution. From this observation a spontaneous question arises: was it impossible to operate the machine in this correct way right from the beginning, thus avoiding the occurence of painful and costly accidents?

The authors certainly do not intend to affirm that the methodology shown in this book will provide a kind of guarantee for avoiding accidents. On the contrary, it has been stated several times that risk-free methodologies do not exist. Instead, the authors would like only to emphasize that by operating a machine correctly, there will be, at any moment, the *documented* certainty of having put in action all the available means aimed at avoiding accidents.

Consequently, the primary solution can be stated as follows:

- the choice of a city machine, followed by
- a rigorous application of the excavation-control and safety procedures (explained in detail later in Section 6).

A city machine is, first of all, a machine equipped with face-support means (Hydroshield or EPB Shield), but it is also a machine satisfying a certain number of minimum requirements as those described in the next subsection.

4.3 ESSENTIAL COMPONENTS OF A "CITY MACHINE"

The scope of this section is to provide a sort of checklist for preparation of the minimum technical requirements for contractors and manufacturers, to be completed with the details pertinent to the actual project.

The special requirements for a city machine are related to:

- Excavation process (including maintenance and muck handling).
- Face-support-pressure control facilities.

- Parameters cross-control system.
- Probe-drilling ahead of the face.
- Ground treatment.
- Guidance.
- Safety of workers.

These requirements, in turn, shall determine the design of the essential, common components of the machine.

It is important to emphasize that, in comparison with the other machines used in other environments, the components to meet the above requirements should be specially designed for the project, manufactured, and used with particular care. In the following subsections, some of these components are described in detail, leaving the functional principles to Section 4.3 (where the two types of commonly used machines are described) and to Sections 6.2 and 6.3 (where the machines' operational control is shown).

4.3.1 Components necessary for the excavation process

4.3.1.1 Cutterhead (rotation, torque, opening ratio)

Depending on the type of rock and/or soil to be excavated, the rotation speed of the cutterhead should be adjusted, in order to adjust the torque consequently (the softer the ground, the lower the rotation speed and the higher the torque, for a certain available power), thus using the maximum available torque.

It is necessary to have a TBM with cutterhead rotation speed and torque (on the main drive) continuously variable through hydraulic or variable-frequency electric motorization. This is a common practice for TBM excavation in non-homogeneous ground conditions, but in an urban environment, it is essential in any case.

Some zones of a given tunnel alignment may be characterized by soils with a high percentage of clays and silts which can be highly cohesive and plastic. When a TBM excavates through such zones, the soils may exhibit the so-called "sticky behaviour", reducing significantly the production and, in some cases, even causing a complete stop of the advancement of the tunnel. Whenever such a doubt arises, it is opportune to investigate deeply the question right from the initial stages of selecting and designing the TBM for the project. This is because the sticky behaviour may strongly influence the configuration of the cutterhead design (the percent opening ratio) and, in general, also the mucking path from the plenum to the first conveyor belt (in the case of an EBP shield). Furthermore, it can substantially affect the level of torque to be supplied to the cutterhead, especially for EPB shields.

A simple and efficient method exists for assessing the "stickiness" of the ground to be excavated, based on the Natural Water content, W_n, Plastic limit, W_p, and Plasticity Index, I_p.

The ground should have a sticky behaviour if

$W_n/W_p \geq 1.0$

and

$I_p \geq 0.25$

This kind of behaviour (very dangerous for the TBM operations) is evident in the presence of bentonite slurry, always used in Slurry Shield SS, Hydroshield (HS) and very often also in Earth Pressure Balance Shield (EPBS). In any case, the hazard of a clogging in the muck circulation is so dangerous, that the plenum (see Fig. 4.4) should be designed to favour the circulation of muck from its upper part to its lower part. Thus, the openings in the centre part of the chamber shall be bigger than in the external part. This feature is very important because in the central part of the plenum the speed is the lowest. In almost all experiences, and in particular with sticky materials, it was noted that the clogging of the cutterhead starts in this central part. So, this part of the cutterhead, is another important area to be designed to avoid the clogging when excavating in sticky materials. In this area, the tangential speed of the cutterhead is rather low and, consequently, the excavated material is, relatively speaking, moving very slowly. This slow flow of material induces the initiation of front cutterhead clogging, which increases until complete blockage of advance. A wider-open-centre design also limits the wear on the cutterhead structure, in addition to increasing the flow of the material.

These phenomena increase the need of cutterhead torque and thrust and, eventually, put severe limits on the TBM advance rate. The recommendation that emerges is that, together with an appropriate design of cutterhead, injection of additives to the front or in the cutter chamber should be seriously investigated and implemented.

4.3.1.2 Main drive (seals)

The main drive is one of the most important mechanical parts of a TBM. In Closed-Face Machines the main bearing can potentially be polluted by pressurized slurry contained in the plenum. Therefore, it is essential to provide special devices for sealing the main drive in case of pressurized plenum:

- The main bearing has to be protected by lip seals. It is important to verify that the bearing is equipped with 2 rows of lip seals (4 or 5 each), respectively, at the inner and outer part of the rings.
- The lip-seal's design must foresee a special oil/grease lubrication system which ensures the full protection of the bearing.
- The system for automatically greasing the seals provides a guarantee for their performance during excavation. The system protects the seals with a continuous flow of oil/grease, providing a total reliability of the lubrication.

4.3.1.3 Muck extraction system

The muck transport system (pumping system for SS and screw conveyor plus belt conveyor(s) for EPBS) must be properly designed and dimensioned to avoid clogging the suction area.

In fact, when excavating sticky soil with the contemporary presence of viscous slurry, there is the tendency to close the suction area and/or the cutterhead openings, thus choking the muck circulation ways.

On the contrary, in coarse alluvial ground or in weak rock, the presence of boulders can create problems for the circulation of muck. If the quantity and dimensions of boulders are important, the use of an SS allows the installation of a stone crusher before the muck-entrance gate to the pipes of the pumping system. Of course, this is not possible in an EPBS, due to the density of the material that would immediately choke the crusher. The boulders in an EPB will pass through the screw and thus the maximum allowable dimensions shall depend on the diameter of screw cage, the pitch, and the diameter of the central shaft. Otherwise, the boulder should be maintained outside of the plenum by installing some grids in between the arms of the cutterhead.

4.3.1.4 Maintenance

The most sensitive maintenance works are related to checking the conditions of, and changing, the cutting tools on the cutterhead; these operations are executed in the plenum and under air pressure.

For this reason, every "city machine" has to be provided with one or more "hyperbaric chambers" (or "man locks") that allow maintenance works to be performed (even under the water table or in the case of risk of face instability), and with one "material lock" that allows men and material to pass through sealed and pressurized doors.

The man lock must be a "twin chamber" type and is located at the upper part of the main shield. One chamber is the main man lock and allows personnel to pass from the shield to the plenum. The second chamber is the emergency man lock that allows safety personnel to reach the plenum or the main man lock, in case of injuries of the personnel working inside the plenum. The man locks shall be equipped with all the supply circuits and standard devices required by the local regulations for operation under compressed air.

4.3.2 Measures for face and crown stability control

4.3.2.1 Pressure sensors

As a general rule, Closed-Face TBMs are equipped with pressure sensors, installed on the bulkhead, following the configuration shown in Figure 4.1 for an EPB:

- 2 in the upper part (crown pressure).
- 2 at axis level (average pressure).
- 2 in the lower part (maximum pressure).

However, for the HS, air pressure sensors are installed in the air cushion; and two slurry-pressure sensors on the bulkhead are enough because the pressure distribution is similar to that of hydrostatic water pressure.

The pressure measurements are displayed to the operator on the screen in the cabin on-board the TBM to allow him to manage the face-confinement pressure, specifically for the crown-stability control, and to manage the water ingress.

Other earth pressure sensors should be installed along the casing of the screw conveyor (at least one at each end of the screw) to indicate (to the operator) the pressure drop along the screw and to detect the changes in the muck fluidity.

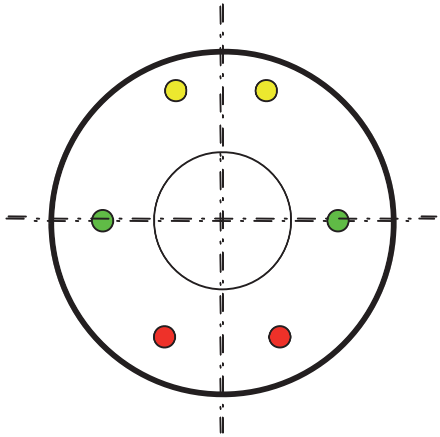


Figure 4.1 The position of pressure sensors in the bulkhead of an EPB machine.

4.3.2.2 Measurements of extracted dry material

The current technology allows us to measure, with sufficient precision, the quantity of extracted material (from the piping system, through the separation plant, in case of Hydroshield, and from the screw conveyor in case of EPBS) for comparing it with the actually excavated material.

In a “city machine”, these measurements and the relevant scales are mandatory. In the case of HS, flowmeters and densitometers have to be installed on both in- and out- slurry pipes; in the case of EPBS, scales installed on the belt conveyor can directly indicate the weight of the extracted material.

All these measuring instruments need be easily calibrated. The calibration should be done regularly and frequently (in case of scales for EPBS, even every day, or maybe every shift).

In the operator cabin, a special screen shall be dedicated to the comparison between the excavated and the extracted quantities (see Section 6, Figures 6.26a, b and c).

4.3.2.3 Foam and slurry injection

Application of the EPB technology requires the possibility to inject foam (generated by mixing water, foaming agent, and compressed air) for improving the fluidity and reducing the permeability of excavated materials inside the plenum.

The quantity (and the quality) of foam needed depends on the types of ground to be treated before excavation. Therefore, it is very important that the foaming system has a fine-quantity regulator, with also the possibility to add polymers and other additives (see Section 6.3).

The hydroshield technology is based on the use of bentonite slurry in which it is now possible to also add polymers in order to improve the quality of the slurry and its behaviour into the ground.

4.3.2.4 Bentonite slurry injection in the plenum and around the shield

The face-support pressure must be maintained above the minimum level (the calculated "attention level," see Section 5.2) without any interruption. In the case of Hydroshield, this is ensured by the combination of inflow of slurry in the plenum, and the air cushion pressurization that can be maintained both during normal excavation and stoppage (lining assembly, maintenance, break-downs etc.). In the case of EPB Shield, it is noted that, when an excavation phase is finished, and during the entire consequent stop phase, the face-support pressure has the tendency to decrease (see Section 6.3), to even below the attention level. For this reason it is recommended to install an auxiliary device, called the Secondary Face Support System (SFSS), able to inject bentonite slurry into the plenum, bringing back the pressure to the desired value (see Section 6.3.2.2). Installation of an automatic SFSS (see Porto Metro, Section 8.3) is also shown to be useful to avoid the manual intervention by the operator.

The sector of the "annular void" around the EPBS body cannot be backfilled with mortar as it would increase the friction forces to unacceptable values. In the case of excavation with Hydroshield, bentonite slurry normally fills also this void, providing some support also on the tunnel roof, but just as a "passive" support. For the EPB Shield, this passive effect is totally missing. In this case, some recent experiences (Madrid, Barcelona, Porto) have demonstrated that injecting bentonite slurry all around the shield and maintaining it at a certain pressure (more or less the same as that in the plenum), it is possible to obtain a certain "support effect", together with a lubrication effect, which can reduce the frictional forces.

But, in order to utilize this injection as an "active" support on the roof, the topic needs to be studied deeply. In 2002, a Research and Development project was presented to the European Commission for funding, with the name of "self supporting TBM", but without results. The principles of this TBM were presented during the ITA Congress in Amsterdam 2003 (see Xu *et al.*, 2003).

4.3.2.5 Safety gate

The lower part of the bulkhead in the front shield of an EPBS must be equipped with a safety gate, which can be closed when the screw conveyor is retracted for maintenance. This allows the complete insulation of the plenum, avoiding water/material inflow during maintenance.

In an HS, the by-pass valve in the slurry circuit serves the same purpose, i.e. to insulate the plenum from the rest of the tunnel, and if necessary, for example, during the "slurry circulation" phases to reduce the slurry density, without disturbing the equilibrium in the plenum.

4.3.2.6 Tail skin sealing system

The tail shield should be equipped with at least 3 rows of wire-brush type of tail seal, with injection of tail-seal grease between adjacent rows to ensure high-level water tightness. As a matter of fact, with only two rows of wire-brushes, only one grease chamber is available, which cannot ensure sufficient protection from the possible inflow of slurry and/or grouting mortar, and the corresponding risk level is too high for an urban project.

4.3.3 System for cross-control of excavation parameters

4.3.3.1 Data logger

It is absolutely necessary to install a data logger on the TBM to monitor, record, and control the excavation parameters; and in particular, the following sensors/devices have to be installed to allow for the relevant checks (see Sections 6.2 and 6.3):

- Pressure sensors in the plenum (and in the screw conveyor for EPBS). In the case of Hydroshield, it is not enough to measure the air pressure, and thus almost two slurry pressure sensors should be installed on the bulkhead. In the case of EPBS, by using the difference in the pressure values, the "apparent density" of the material in the plenum can be easily calculated. The "apparent density" can be used as an indicator of the degree of filling of the plenum by the excavated materials and thus it should be displayed in real time on the control panel in the TBM operator cabin (see Section 6.3.2).
- The weight and volume gauges for measuring the quantity of the extracted material (scales and/or volumetric scanners in the EPBS, or a combined of capacity/density measurements of the slurry for the Hydroshield).
- All the "mechanical" parameter gauges of the TBM: cutterhead rotation speed and penetration rate; rotation speed of the screw conveyor, in EPBS; in- and out-slurry flow and density, in Hydroshield; torque on the excavation cutterhead; and pressures in the thrust jacks for the TBM advancement.
- Injection-pressure sensors and flow metres to monitor the grouting of the filling mortar behind the segments.

4.3.3.2 Web connection

In order to give all the Actors of the project the possibility to know in real time what is happening, both in the tunnel and on the surface above the tunnel, a complete monitoring system for both the surface settlements and the ground movement at depth, integrated with the control of the excavation parameters, has to be implemented; all the data shall be available to all the Actors, through a computerized, Geographical Information System (GIS, see Section 6.4), connected to the WEB.

4.3.4 Probe drilling

It is considered important that a mechanized excavation in a city, carried out according to the criteria and principles described in this book, should be performed using the most complete knowledge of the geotechnical situation. It can also be stated that all the site investigations in an urban area should be carried out at a preliminary stage, where the sites to be investigated are usually easy to access (relatively low overburden, free and accessible areas). Otherwise, such investigations should be done during construction from inside the TBM. In the latter case, the ground can be investigated through investigating ahead of the excavation face by means of core-drilling and/or probing without sampling, but logging the drilling parameters, or alternatively through continuous prospecting using geophysical methods (seismic, electromagnetic, etc.). Investigations from the TBM are recommended as secondary investigations, when passing under buildings or other existing structures, to increase the knowledge about the ground to be excavated in particularly sensitive areas. For details on site investigations, reference can be made to Appendix 3 and BTS 2005.

For probing the surrounding ground ahead of the excavation, the TBM should be designed to permit the specified drilling operations, with compact drilling rig(s) equipped on board to allow for the realization of inclined holes diverging forward at 10–15°. In case of need, it should be possible to mount special probe(s) on the drilling rod for direct investigations of the in-situ material characteristics.

In certain situations, the knowledge of the surrounding ground alone might not be sufficient, for instance, in the presence of karstic phenomena (encountered in the projects in Paris and Kuala Lumpur, discussed in Section 8). In these cases, it is necessary to know the ground characteristics in the section to be excavated. Therefore, the machine should be designed to allow drilling of boreholes in the excavation section through the cutterhead. To do this, particular positions of the cutterhead must be identified and aligned with the holes arranged in the bulkhead, and the drilling rig should be equipped with a "blow out preventer" to avoid the loss of pressure and/or the loss of water.

There have been great developments in recent years as far as "continuous" reconnaissance systems are concerned, whether they are electric, magnetic, or seismic. Guidance on choosing the best technique for a given situation can be found in the literature (for example, the Closed-Face Working Group Report, BTS, 2005). It is only necessary to underline here that all of these systems need to be "calibrated" on site and in the ground in order to ensure that the interpretations are meaningful.

When drilling ahead of a section to be excavated, it is also necessary to be careful in order to avoid the risk of losing the drill fittings, which could damage the cutterhead as it passes through the section later. A safe solution, in this case, is to use aluminium rods.

4.3.5 Ground improvement

Clearly, ground improvement should be done from the surface if it can be done successfully without undue disturbance to the public. However, sometimes there are some areas or locations along the tunnel route, requiring risk mitigation interventions prior to construction to protect the already existing structures, which are not totally accessible from the surface. In these cases, it is necessary to carry out the interventions from inside the TBM. However, it is well understood that interventions from inside a TBM are not as effective as those from the surface, because of numerous constraints in the already congested space. Therefore, the number of boreholes that can be drilled is limited. This borehole umbrella will always be inclined and diverging rapidly into the ground surrounding the tunnel. In short, there are few possibilities during the design stage for the choice of the treatment "mesh" (or pattern); and significant interventions at the face can only be performed with particular devices; and the treatment operations are, however, always risky.

If systematic and effective treatments from the TBM are not possible, it is necessary to use other systems, such as especially prepared access shafts or galleries from which the drilling and grouting can be performed with relative ease. In some cases (for example, "Nodo di Bologna", see Section 8.5) recourse has successfully been made to guided, inclined drilling from surface.

The equipment foreseen to be used for drilling the probe holes is also useful for the ground treatment. The usual machines that are employed to inject cement or silicate mixes from the surface can also be used in the tunnels. In certain cases, it is necessary to inject particular grouts (for example, expandable products such as resin and/or polyurethane foam) in order to resolve waterproofing problems or to temporarily fill the voids that have been identified or caused during excavation.

4.3.6 Guidance of the TBM

The modern guidance systems are capable of operating also in urban environments, even though the radii of curvature of the tunnels in cities are often at a minimal value due to alignment constraints (see Section 3).

The so-called "universal rings" are commonly used to line tunnels excavated by a shielded TBM in city environment; and theoretically, these rings can be utilized for any tunnel radius from the minimum value in the design to infinity, i.e. a straight line (see Section 5.3). Modern guidance systems can help to determine the segment-ring assembly sequence in order to obtain the desired radius.

It is recommended that high precision guidance systems and the corresponding software be used, as these can be considered indispensable for the complete equipping of an "ideal city machine", in order to:

a. guarantee that a TBM follows the designed alignment of excavation, with the excavated tunnel to be lined in a precise way;
b. allow the correction of alignment errors (which cannot be completely avoided) to remain within the tolerance limits allowed for a given type of lining and by the shield/lining segment coupling, and
c. reduce the risk of failure and/or damage of the segments, during the assembly and thrust stages, to a minimum.

4.3.7 Safety

The term "safety" refers here to the safety of the personnel who work inside the machine and the tunnel. The details of health and safety issues are provided in Section 7.

In most cases, a machine that has an EC (European Commission) certification mark may satisfy the safety requirements (see Section 7). However, it is often necessary to make adjustments that are requested, from time to time, following the recommendations of the Inspection Commissions. The requested specific adjustments can change from country to country and, sometimes, even from region to region. However, what should never be missing are the following components.

On the machine:

- Control monitors in the operator cabin.
- Control panel for segment erector strictly at disposal of a unique operator.
- Protection for people from contact with the mobile parts such as conveyor belts.
- Equipment to recuperate an injured person in the plenum (special slings, stretcher attachment cables).
- Rescue containers.
- Emergency locomotive in the TBM backup.
- Fire extinguishing apparatus on the main mechanical parts (hydraulic and electric engines).
- Methane (CH_4) gas sensors at the excavation face, and meters for oxygen, CO, and CO_2 levels.
- End buffers (or dampers) for the segment carrier wagons, etc.

On the trains:

- Self-braking wagons.
- Radio-controlled cameras and monitors in the driver cabins for rear vision.
- Closed cabins.
- Special closed wagons for personnel transport.
- Locomotives with fire extinguishing equipment (at least of a manual type).

In the tunnel:

- Pedestrian platforms.
- Emergency stations (1 at each 500 m interval if the tunnel is longer than 1000 m).
- Fire extinguishers.
- Emergency lights.
- Track-switching points, or double-tracks, if the space inside the tunnel allows them.

At the portals:

- Access checks for both the personnel and visitors.
- Emergency hyperbaric chamber.
- Stretcher-lifting equipment.
- Security containers with emergency equipment (overalls, breathing equipment, etc.).
- Emergency wagons with stretcher carriers.
- Communication systems between the control station and the trains as well as the TBM.
- Buffers (or dampers) for trains at the tunnel entrance.

4.4 SLURRY AND EARTH-PRESSURE BALANCE SHIELDS: FUNCTIONING PRINCIPLES AND REVIEW OF THE STATE OF THE ART

There are currently two types of machine that can comply with the technical requirements described in the previous subsection: Slurry Shield or Hydroshield and EPB Shield.

The choice between a slurry machine and an EPB machine usually cannot be made "a priori" simply based on the type of ground, the particle size distribution of the various lithologies involved, the presence or absence of groundwater, the height of the water table (if present) with respect to depth of the tunnel, etc. All these parameters are important for making the selection, but they need to be checked, case-by-case, in the light of risk analysis for the specific project.

In any case, to make the correct choice, it is necessary to know and understand well the characteristics of both types of machines. A general description of various types of Tunnelling Machines, following the classification scheme of ITA WG 14, is shown in Appendix 1. The following paragraphs of this subsection, will illustrate in more detail the two types of machine that are particularly used in urban areas: Slurry Shields (SS, see Fig. 4.2) and Earth Pressure Balance Shields (EPBS).

4.4.1 Slurry shields

4.4.1.1 Operative principles

The Slurry Shield is a machine that is able to support the excavation face by a pressurized, bentonite slurry pumped into the excavation chamber. The slurry is substantially

Figure 4.2 The 14.2 m-diameter Herrenknecht Slurry Shield model S-108, used for excavating the Elbe tunnel in Hamburg.

composed of a bentonite suspension in water, with some additives if necessary. The excavation chamber, called the "plenum", is a space between the excavation face and a steel bulkhead (separating the plenum from the remaining part of the TBM), where the excavated material is collected and mixed with the slurry. A pumping system performs the functions of feeding the fresh slurry to, and removing the muck from, the plenum through a pipeline (see Fig. 4.3).

The balance between inflow and outflow involved in this cycle allows the slurry to be maintained under pressure in the plenum. By the variation of the inflow and/or outflow of the slurry, it is possible to control the face-support pressure value.

In the case of the Hydroshield a supplementary bulkhead, installed further behind the primary bulkhead, creates a room or an auxiliary chamber, which is divided into two functional compartments. The compressed air in the air cushion can push the slurry to the plenum in front, maintaining it under pressure. The air cushion pressure can be managed through an automatic regulation system. Consequently, it is possible to control the slurry pressure. The air bubble also acts as a compensative "shock absorber" to the unavoidable pressure fluctuations in the plenum (see Fig. 4.3).

4.4.1.2 Key parameters

The principle relating to the face and tunnel crown stabilization is the same for both "Slurry Shield" and "Hydroshield". But the Hydroshield is a more complex machine, due to the use of the "air cushion". Therefore, the detailed description will refer just to the Hydroshield, as to our knowledge it is the only type of "slurry shield" being used in Europe, in the form of its variants like "Mixed Shield" or "Benton'air", etc.

Actually the name "Hydroshield" historically refers to the machine invented by Weyss and Freitag and built by Bade and Theelen in the late 1970s. Afterwards, Voest Alpine of Austria bought from Bade both the name and the technology. Unfortunately, Voest Alpine quit the market in about 1995, after the construction of the two machines for the EOLE project in Paris and for Metro Rome (line A extension). However, the term 'hydroshield' has now become a common denomination.

The operating principles of the machine is described in Figure 4.4.

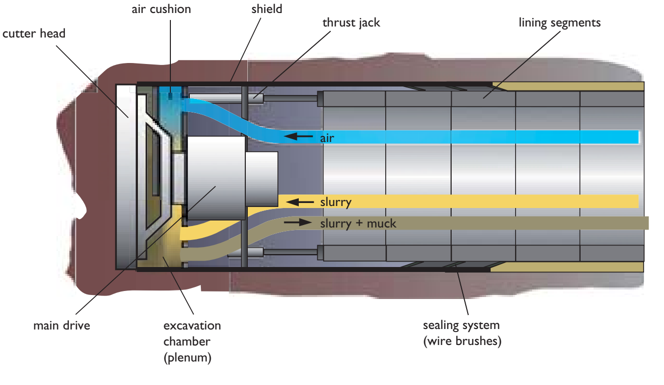


Figure 4.3 Functioning principle of a HydroShield.

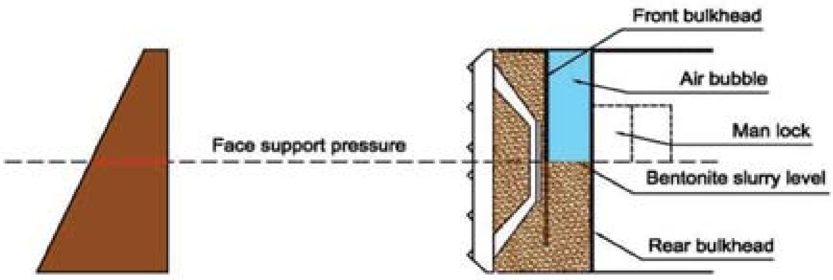


Figure 4.4 The scheme of a Hydroshield.

As indicated in Section 5.2, the pressure to be applied to the face depends on the ground pressure and the hydrostatic pressure (if any). The face-support pressure is in part supplied by the slurry (which creates a hydrostatic pressure as a function of its density) and in part by applying air pressure on the slurry through the overlying "air cushion".

The mechanism by which the pressure is applied is the following. The plenum is physically separated from the tunnel by a bulkhead (the rear bulkhead in Fig. 4.4) The chamber is divided into two compartments, by the front bulkhead. The front compartment, where the cutterhead is located, is kept full of slurry. The rear compartment and the front compartment are connected in the lower part of the chamber but they are separated by the front bulkhead in the upper part. The rear compartment is filled with slurry only in the lower part, while compressed air is fed into the upper part (forming the so called "air cushion").

In the rear bulkhead are located the inlets and outlets of the compressed air, regulated by a system of automatic valves, and the inlets and outlets of the bentonite slurry, which also function as conveyors for the excavated material.

In the classical "slurry shields" the pressure control is obtained by "balancing" the inlet and outlet flows. If the extracted volume is smaller than the injected one, more material will be accumulated inside the plenum and, therefore, the pressure increases.

In the Hydroshield, the pressure control in the chamber is obtained by controlling the pressure on the "air cushion": the compressed air "pushes" the slurry into the front chamber, thus applying the required pressure. Moreover, the air cushion exerts a compensating function, or acts as a dampener against pressure variations: an excessive slurry pressure is automatically reduced by reducing the air pressure in the air cushion via a release valve; and a pressure loss calls for new compressed air, which re-establishes the equilibrium in the system.

This system, apparently simple and effective, actually displays several operating difficulties, which are described hereafter to better understand their mechanism, to control, and safely manage them.

Using this type of TBM, the face-pressure calculation is fundamental, but it becomes even more important to take into account the rheological characteristics of the slurry, which in turn depend on the ground characteristics and the slurry material components: water and bentonite, but also on additives such as polymers. Another essential element for controlling the stabilization parameters is the slurry level in the rear chamber (or level of the separation line between air and slurry).

Beyond its primary function of exerting an active pressure on the excavation front, the slurry also gives benefits to the productive cycle, thanks to its cooling and lubrication effects.

4.4.1.3 Slurry characteristics

As stated earlier, the principle of the slurry-shielded TBM is to apply an adequate pressure on the slurry contained in the plenum. This pressure is thus transferred to the face to obtain a pressure value as close as possible to that calculated in relation to the in-situ stress.

The correct application of pressure is closely related to the correct reaction of the "slurry-ground" system. The pressure pushes the slurry into the ground pores, shedding its portion of solids and thus forming a film (called "cake"), which allows the correct distribution of the applied pressure to the entire face. The penetration distance and the cake thickness are functions of the applied pressure, of the grain size of the ground and of the slurry as well as of the electro-kinetic potential, which is related to the surface activity of the slurry particles, of the slurrys rheological characteristics, and of the hydro-geological conditions, principally the salt contents in the ground water. The confinement is of a hydrostatic and mechanical nature, the former is prevalent in ground with fine particle size and some cohesion, and the latter prevails when cohesion is low or nil, as individual particles then need separate support (independent from others).

The slurry cake also assists the maintenance operation in the excavation chamber, by enabling the compressed air pressure (instead of the liquid pressure) to act on the front when the plenum is emptied for maintenance, avoiding at the same time the leakage of air.

Since the excavated material disposal is through the pumped-out slurry, it is necessary to ensure that (1) the pipe diameter is compatible with the required volume to guarantee sufficient transport velocity, and (2) the slurry does not generate grain sedimentation along the conduit from the shield to the separation and treatment plant where the coarse particles are separated from the fines and the liquid to allow transport to muck disposal and recycling of the bentonite slurry. Particular attention should be paid to the slurry characteristics to permit the separation and treatment indicated above. Experience from several case histories suggests consequently adopting bentonite slurry with varying contents (type and/or quantity) of polymer additives, to resist better the polluting agents present both in the ground and in the water used to form the slurry mix. This will improve also the separation ability of the plant.

The principal control parameters of the slurry quality are known (Milligan, 2001). However, the control has to be particularly accurate, since the initial slurry, prepared from mixing water and bentonite, after appropriate hydration, is subsequently "polluted" by the excavated material (ground, water from underground water-tables, and, in urban areas, by chemical products present in the ground and in the water).

The apparent viscosity, V_a, is measured in centipoises (cp) in the FANN viscositymeter and is (Eq. 4.1):

$$V_a = F_{600}/2 \text{ [cp]} \tag{4.1}$$

where F_{600} is the viscosity at 600 rpm.

The plastic viscosity, V_p, is the difference between the viscosities at 600 and 300 rpm: (Eq. 4.2)

$$\mathrm{V}_p = \mathrm{F}_{600} - \mathrm{F}_{300} \text{ [cp]} \tag{4.2}$$

The "yield value", defined as (Eq. 4.3):

$$Yv = 0.96 \cdot (F_{300} \cdot V_p) \ [Pa] \tag{4.3}$$

provides a measure of the penetration resistance of the slurry into the ground (FPS, 2006).

The "filtrate", defined as the liquid that passes through a filter cake from a slurry, and measured by a specific lab test, indicates the penetration ability of the slurry into the ground.

It is also necessary to measure continuously the slurry density, both in and out. Proper functioning of the transportation plant, and particularly of the separation plant, is substantially dependent on this parameter. Good practice indicates that slurry density should not exceed 12.5 kN/m^3 in the outlet circuit, since the initial density is about 10.2–10.3 kN/m^3.

As stated, the reference to 'slurry' should be actually to a fluid whose characteristics lie in the range of the characteristics of pure bentonite slurry and a bentonite slurry and ground mix, which is present at the end of each excavation cycle. Hence, these measurements need to be repeated several times even within the same excavation cycle, if necessary, especially if substantial and sudden variations of the reference values are detected.

The procedure is rather complex and is the subject of specific studies by specialists: Suffice to note here that the slurry in contact with the face tends (under pressure) to penetrate the ground, which then acts as a filter. The slurry is able to penetrate to some distance, which is a function not only of the applied pressure and the interstitial pressure opposing the penetration, but also of the physical characteristics of the ground and the slurry. The solid portion deposited during the penetration constitutes the "cake", i.e. the active element transmitting to the face the support pressure, while the liquid portion is dispersed.

The delicate equilibrium between these components (which depends on the values of "Filtrate" and "Yield value") constitutes the essence of a proper functioning of the Slurry Shield, which, in turn, governs the face support.

4.4.1.4 The various types of cake

The knowledge of the type and function of the cake is extremely important. An excessively thick cake is not advantageous for stability, whereas too thin a film would make the face support less efficient. Moreover, too much water penetration into the ground has to be avoided, as it could favour possible swelling of clay (if present) and it would reduce the efficiency, for it would increase the interstitial pressure.

Two types of filter cake can be distinguished in relation to grain size: (1) membrane cake for ground with fine particles, and (2) impregnation film for coarser ground. In the first instance, inter-granular forces prevail, in particular the particle attraction due to the electro-kinetic potential is an important fraction of their weight. With a membrane film, increasing the pressure has practically no influence on slurry penetration, since the film becomes a 'continuous' obstacle on the face. However, an impregnation film is definitely more sensitive to pressure variations and, in most cases, an increase in pressure will lead to a restart of the slurry migration into the ground.

In either of these cases, the behaviour at the face will be considerably different. A "membrane" is definitely more favourable for the 'micro'-stability of the single particles, since it is capable of providing a more uniform pressure distribution and imparting a kind of artificial cohesion, extremely important in cohesionless ground. On the other hand, an "impregnation" film will have a greater thickness because the slurry finds less obstacle to penetration; it provides an important role because the face being "cut" is a system in evolution, the ground is continuously modified by the cutterhead rotation and consequently part of the film is continuously removed (Fig. 4.5).

4.4.1.5 Components and additives of the slurry

Bentonite is the fundamental component of the slurry. The terms "bentonite" and "smectite" are generally used to define the natural clay minerals which essentially belong to the sodium-, calcium-, or potassium-rich montmorillonite group. Because of their chemical composition and microstructure, these minerals have a strong ability to absorb water.

A useful tool for differentiating between the types of bentonite is the activity index, defined as the relation between plasticity index and clay percentage. The higher the activity index, the bigger the sodium montmorillonite percentage in the bentonite. The best bentonite for slurry is mainly composed of sodium montmorillonite (preferably more than 90%). The commercial sodium bentonite is generally obtained through

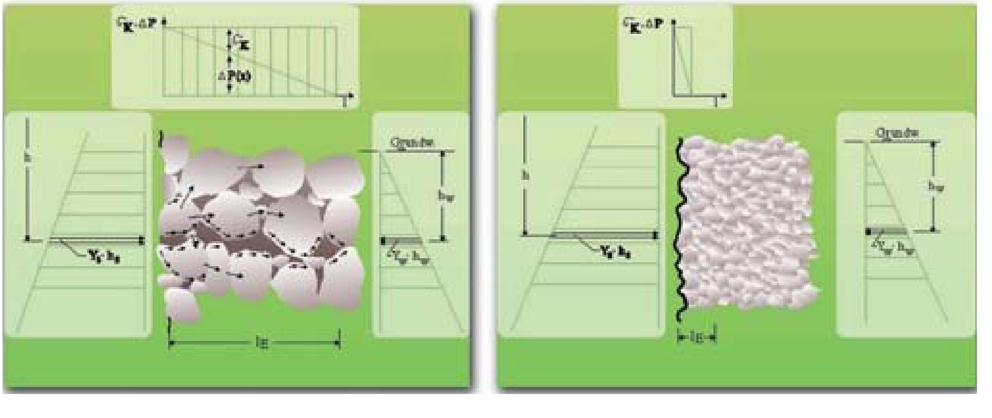


Figure 4.5 Slurry cake formation: the two types of cake.

the substitution of calcium ions with sodium ions. In any case, the bentonite in the market can present different qualities; consequently, its utilization in the mechanized excavation field with Slurry Shield may require the addition of small amounts of polymers, with the purpose of reaching optimum properties.

The bentonite slurry is prepared by mixing water and bentonite powder, in the desired proportions, to obtain the required rheological characteristics. The mix has to be "hydrated" in a tank for at least 12 hours, in order to reach the optimal characteristics. The proportioning typically varies from 30 kg/m^3 to 60 kg/m^3. The thixotropic properties of the bentonite allow the formation of a gel if the concentration of water (by weight) is higher than 5% (Milligan, 2001).

To improve the slurry quality, different types of polymers can be added. In natural grounds with high permeability, use of polymers is recommended. These are able to reduce the slurry penetration and, consequently, its dispersion, making the formation of the slurry cake easier and more efficient. It is possible to improve the rheological properties of the slurry through the introduction of long-chain molecules, which behave as reinforcement fibres shaped like a "net" that is able to retain the bentonite particles, acting as a "bridge" between the ground grains.

The slurry must have the required velocity in the pipeline, without requiring excessive power consumption, inducing wear to the pipe and pump, and causing sedimentation. However, in case of interruption of the flow, sedimentation of the material could possibly occur with a consequent clogging risk for the pipe system. To minimize this risk, it is possible to continue the slurry circulation through a by-pass valve, even when the excavation process is stopped, ensuring a certain velocity of the slurry inside the pipe. In addition, it is possible to add specific additives to the bentonite slurry in order to control its viscosity and to facilitate the flow inside the pipe.

Polymers can be also used for other purposes. For instance, in grounds with high salt content, it is recommended to use special types of polymers that reduce the sensitivity of the slurry to the contamination caused by the salt. In soil with heavy clay, the use of polymers reduces the clay dispersion and, therefore, maintains the slurry functions over a longer period. When the natural content of clay is high, only a mix of water and polymers could be sufficient to create the slurry.

4.4.1.6 The bentonite slurry treatment and separation plant

The treatment plant has the purpose to prepare, stock, and control the slurry. The mixture is prepared in a high shear mixer. Normally the mix is made denser; it is then hydrated and diluted to the desired concentration, before it is sent to the feeder tank.

The hydration, stocking and feeder tanks have to be appropriately dimensioned according to the plant output in order not to remain short of slurry; in the stocking tank will also be stored the slurry recuperated from the separation plant, after appropriate monitoring of the characteristics.

Each treatment plant has to have an annexed laboratory, to control the density, the viscosity, the yield value, the filtered material, and the thickness of the cake, that is all the parameters defining the "quality" of the bentonite slurry. The same laboratory will also service the separation plant, described below.

This part of the treatment plant is essentially the same as used for the preparation of the bentonite slurry for excavation of the diaphragm walls or for the perforations using bentonite, except for the size. The following cases provide practical examples of the characteristics of the slurry plants. The EOLE project in Paris and the S. Petersburg Metro (excavation diameter 7.40 m) were furnished with slurry pumping plants with a nominal output of 1200 m^3/h, while the SMART project in Kuala Lumpur with an excavation diameter of 11.30 m required a plant with the capacity of 2500 m^3/h. In the Paris and Kuala Lumpur projects it was "normal", due to the karst phenomena in the limestone, to incur slurry losses of the order of 800–1000 m^3/h, quantities which could spell a disaster for both the fresh slurry supply and the pressure control at the face!

The pumping system moving the slurry "in" and the liquid muck "out" represents an essential element of the "hydroshield" system because the performance of the entire system depends on its dimensioning.

Some typical quantities to be pumped have been mentioned above: the necessary plant power is determined by dimensioning the pump head necessary for the tunnel profile and for the position of the treatment and separation plant. For the SMART project (11.3 m diameter, see Section 8.5) the plant had 6 pumps of 900 kW (each) with a total electric power equal to what was necessary for operating the TBM.

The diameter of the pipelines must take into account the minimum velocity in order to avoid sedimentation of the solid in suspension, which would provoke choking of the system and its failure. Typically, the speed should not fall below 0.5–0.7 m/s and the plant should also be dimensioned for a nominal speed >1.0 m/s. The solid particle size and its quantity in the mix composition also plays a role in assigning the speed.

The separation plant is one of the most important components of the hydroshield system. Often it constitutes the determinant for the average rate of progress and, equally often, it can reduce the TBM advance rate, even to the point of stoppage, if it is not appropriately designed and dimensioned (see Fig. 4.6 and 4.7). The essential elements for its dimensioning are the quantity of material circulating in the transport system (as a function of the excavation diameter and maximum rate of progress required from the TBM) and the grain size of the material to be excavated, in particular its content of "super-fines", i.e. the particles of dimension <50 μ, which is a determinant for the dimensioning of the separation equipment.

Basically the plant is made of three sections in order to perform the following functions:

- to separate the coarser components (>4–6 mm), using vibrating screens;
- to separate the fine components (>0.3–0.5 mm), using of one or more stages of "cyclones", which separate solids from liquids by centrifugal effect;
- to separate the super-fines (>50 μ.) using special equipments, such as the "centrifuges", the press-belts, and the press-filters.

The remaining (<50 μ) can be re-utilized together with the water, by adding to the fresh bentonite.

In the last few years, centrifuges have been used less and less, due to their complexity and difficulty to manage and calibrate them, but the final choice of the components of this third part of the plant depends on the grain size of the excavated material and the super-fines in particular.

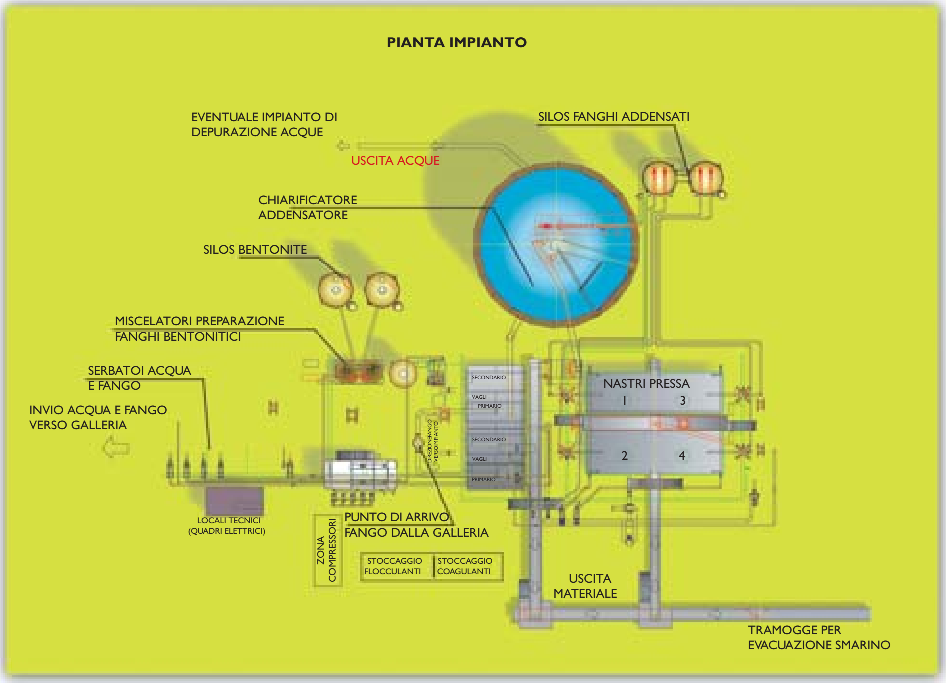


Figure 4.6 Separation plant layout of the EOLE project (see Section 8.1).

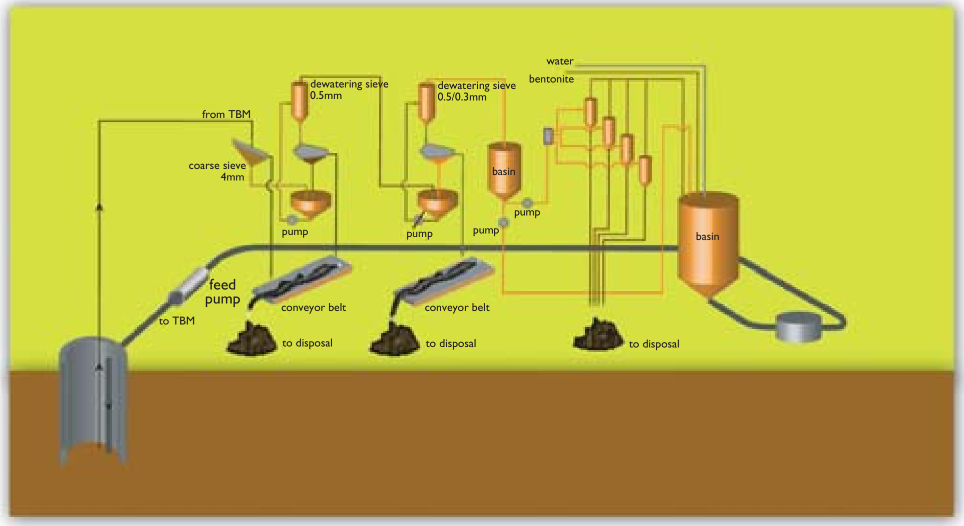


Figure 4.7 Typical components of a slurry treatment plant.

Even for a succinct analysis of the problem, it is recommended that one obtain the best possible knowledge of the excavated material to be separated in order to have all pertinent information (1) for dimensioning of the preparation, transport, and separation plants (the treatment plant), and (2) assigning safety margins in order to be able to face unforeseen conditions. An increase of just a few percentage points in the proportion of super-fines can disrupt the plant performance: it imposes the requirement to circulate the slurry in the piping without entering in the plenum (through the by-pass valve), just to separate the super-fines and lower the slurry density within acceptable limits. Because of this, the excavation is stopped and the average advance rate is reduced.

The "quality" of both the water used to prepare the slurry and the groundwater also has significant importance. If calcium and/or magnesium salts occur in solution, these could have flocculating effects on the bentonite suspension, either impeding or altering the tixotropic functions of the suspension. The chemical composition of water needs to be known and corrections need to be implemented, if necessary, using appropriate chemical additives.

Some treatments are often necessary to restore the physical characteristics of the slurry to acceptable values for recycling by adding some fresh bentonite or additives such as polymers, pH dispellers, or stabilizers. A control of the restored slurry is performed by examining its principal characteristics: density, pH, water loss, yield value, plastic viscosity, and solid content.

4.4.2 Earth pressure balance shield

4.4.2.1 Operative principles

The Earth Pressure Balance Shield, EPBS, is based on the principle of using the thrust and forward movements of the TBM to maintain a pressure on the face. The face-support pressure is applied by utilizing the ground just excavated, collected, and pressurizd in the plenum.

The openings in the TBM cutterhead, which is equipped with cutting tools such as discs or picks, permit collection and accumulation of the excavated ground in the plenum (which is very similar to the slurry shield chamber: i.e. a room between the cutterhead and the bulkhead). The muck extraction from the plenum is done through a rotating screw conveyor, or Archimedes (end-less) screw. The extracted quantity is proportional to the screw-rotation speed, whereas the excavated quantity is proportional to the TBM's penetration rate. A dynamic equilibrium based on the balance of excavated and extracted volume (volume balance) is created inside the plenum. Adjustment of this equilibrium, through variation of the screw-rotation speed, makes it possible to create accumulation and consequent pressurization of material into the plenum.

The face-support pressure is controlled by varying the screw-rotation speed, as a function of the TBM penetration rate.

In addition to the basic functions of muck extraction and control of face-support pressure, the screw conveyor (Fig. 4.8) allows the dissipation of the pressure in the plenum, from the maximum value (on the bottom level of the chamber) to the atmospheric level (at the discharge gate), through the formation of the so-called "plug" of material along the screw itself.

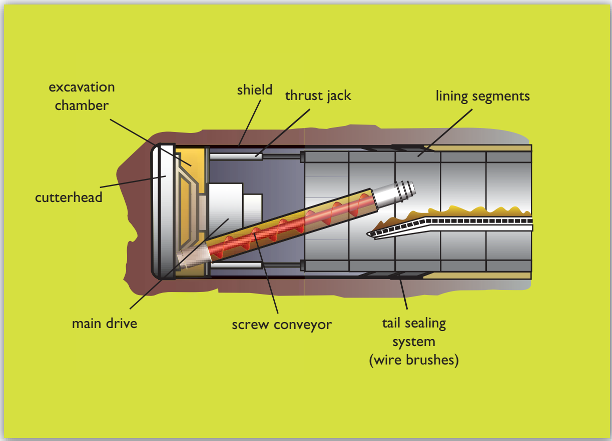


Figure 4.8 The functioning principle of EPB Shield.

The longitudinal thrust cylinders acting on the already positioned lining segments inside the rear shield exert a pushing force on the shield and bulkhead, which then transfers to the ground a pressure that must be adequate for excavating and counteracting the friction forces on the shield and for supplying the needed face-support pressure.

4.4.2.2 Geological and Geotechnical aspects

The question as to what grounds are most adaptable to EPBS excavation has been the subject of numerous technical debates, especially in relation to the ease of excavation and the optimal conditioning of the material. In particular, in the choice and management of the EPBS excavation, the following are the aspects to be analyzed in detail.

1. Type of ground (cohesive, frictional, friable rock, etc.). Once the applicability of an EPBS system has been verified, the type of ground influences the performances of the machine and the correct conditioning of the ground (in the plenum), which in turn has influence on the stability of the face. The use of EPBS becomes unfavourable in ground with fines content lower than 10%.
2. Permeability and location of the water table. Typically the use of EPBS is optimal in ground with permeability less than 10^{-5} m/s and water head less than 3 bars. If the permeability is higher, the type and quantity of the conditioning agent to be added to the plenum and the screw conveyor become relevant.
3. Heterogeneous materials, bedding, and discontinuities, in relation to the tunnel section (EPBS on mixed face).
4. Percentage, maximum dimension, hardness and abrasivity of boulders (if any). These aspect are important in relation to: the mechanical wear and tear, the possibility of entry of boulders into the plenum through the cutterhead, and in the muck disposal via the screw conveyor. The maximum width of the openings, as a

percentage of the cutterhead area, as well as the diameter and pitch of the helical screw have to be related to the maximum diameter of the expected boulders.

5. Percentage of minerals with high chemical reactivity (e.g. the heavy clay group) which can influence the conditioning of the excavated material, and cause a sticky behaviour.
6. Final use and disposal of the muck.

4.4.2.3 *Ground conditioning*

The muck resulting from the mix of natural ground and water (as a simple additive used in the first types of EPB Shield), is not always the optimum medium to transfer the desired support pressures to the front in a continuous and homogeneous way, without a high consumption of power. For these reasons, it is often necessary to add some conditioning agents to the muck in order to positively modify the physical characteristics of the ground. The main purposes of these conditioning agents are to guarantee the control of the face-support pressure, facilitate the "plug" formation inside the screw conveyor, and minimize the cutterhead torque and the wear of the cutting tools (Vinai *et al*., 2007).

There are different types of conditioning agents: slurries, foams, fillers, polymers, and others. The choice depends on their physical and chemical properties and on the type of ground to be excavated. Sometimes it is useful to use a mix of more than one type of the agents.

The conditioning agents can be introduced directly at the front, in the excavation chamber, or in the screw conveyor. In any case, they must be easy to handle and be completely non-toxic and biodegradable, i.e. environmentally friendly.

BENTONITE SLURRY

In the mechanized tunnelling sector, the bentonite slurry finds its maximum application in the Slurry Shield (SS) field. However, the use of slurry is possible or required also for EPBS, when its addition into the excavation chamber enhances the plasticity of the excavated material and reduces its permeability. The details of how the bentonite slurry carries out another role of primary importance, by actively controlling the face-support pressure during stops, are provided in Section 6.

FOAMS

Foam represents the physical state of a special liquid containing a surfactant (the foaming agent) in which air is dispersed so that it expands to enclose the air with a film (or membrane), thus forming bubbles of this liquid. The foam bubbles have an internal pressure higher than the atmospheric pressure; and the bubble pressure is related to the size of the bubble and the strength of the bubble membrane. Bubbles in a dry foam, in which the thickness of the layer has a relatively limited dimension, are not spherical, but are joined together in a polyhedral shape that is almost like a dodecahedron, with nearly planar membranes between the bubbles (Milligan, 2001, Fig. 4.9). The bubble's properties are governed by the Foam Expansion Ratio, FER, the relationship between the foam's and the original liquid's volumes, and also by the nature and concentration

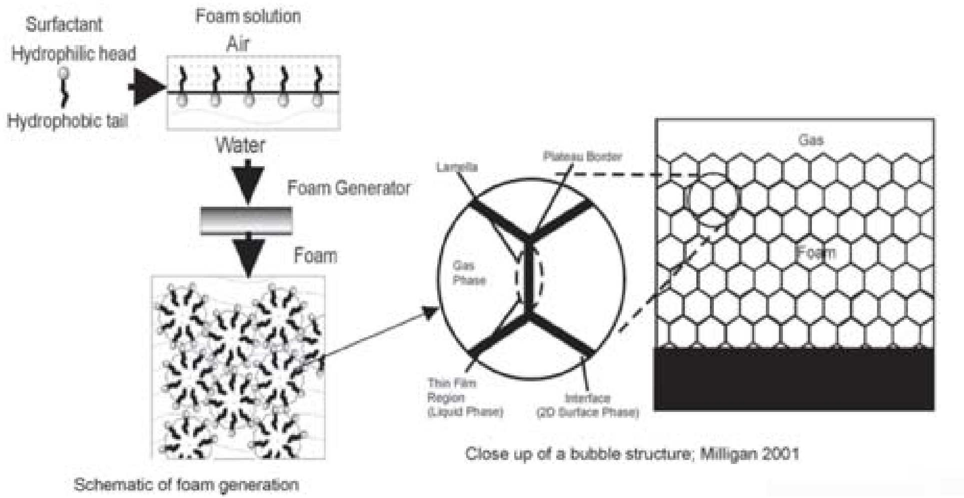


Figure 4.9 Functioning principle of foams (Milligan, 2001).

of the foaming agent in the liquid. Another important parameter controlling the effectiveness of the conditioning is the Foam Injection Rate, FIR, the ratio between the volume of the injected foam and the volume of the treated soil.

For the use of foam in EPBS excavations, it is important to know the total period during which the foam remains in the excavated ground-foam mixture inside the plenum and inside the screw conveyor. During this period, the eventual foam collapse would cause problems basically linked to both a decrease of the face-support pressure and a decrease in workability of the material to be extracted.

The stability of the foam is a function of the dimension and uniformity of the bubbles and the resistance of the membrane. The bubble dimensions should be as small and uniform as possible. In foams with variable bubble dimensions, the bigger ones tend to "capture" the smaller ones, causing a rapid collapse of the foam (Milligan, 2001).
There are different types of foam to be used according to the type of ground and the intended purpose. Three types of foam are listed below:

- Type A: highly dispersive, capable of loosening the strong molecular bonds, typical of clay minerals.
- Type B: used generally in sandy soils.
- Type C: provides high level of stability, keeping the ground cohesive and impermeable.

The principal factors that characterise the control and stability of the foam are:

- *The Concentration Factor* (*CF*): the foam concentration factor, which ranges between 0.5 and 5% is strongly governed by the total quantity of water in the ground, taking into account both the water injected during the excavation and the natural groundwater.

$CF = 100 * \mathrm{ms/mf}$

where CF indicates the foam (tension-active agent) concentration in the water
ms is the mass of the agent in solution
mf is the mass of the solution

- *The F.E.R. (Foam Expansion Ratio)*: is defined as the ratio between the volume of foam at work pressure (*Vf*) and the volume in the original solution (*VF*); the normal range is 10–30.

  $FER = Vf/VF$

- *The F.I.R. (Foam Injection Ratio)*: its range oscillates between 10 and 80%, although it is usually between 30 and 60%. This parameter is strongly controlled by the presence of ground water.

  $FIR = 100 * Vf/Vs$

where *Vs* indicates the in situ volume of the excavated ground (and *Vf* is defined above).

There are different types of tests to evaluate the foam characteristics. Although it is difficult to simulate in a laboratory test what will actually happen during the excavation, the laboratory results can be used as a starting point for further tuning the foams' characteristics to improve their functionality.

Most recently, the Tunnel and Underground Space Centre, TUSC, Politecnico di Torino, 2006, has undertaken a research program on the effects of foam conditioning on various soils, developing a screw conveyor experimental apparatus capable of applying a relatively high pressure in a chamber, which can be assimilated to an EPB plenum, to study the behaviour of conditioned material in EPB Machines (Vinai *et al.*, 2007).

POLYMERS

Polymers are suitable as conditioning agents in combination with foams. Depending on their concentration, polymers work as agents for the modification of the viscosity of the muck to facilitate its behaviour in the plenum, related to the viscosity. Other advantages of polymers include a reduction in the "stickiness" of an adhesive ground and an increased stability of the foam. When used in a concentration range of 1 to 3%, the polymer tends to bind larger particles into small chunks. Material handling devices can handle the chunks more easily than free-flowing fines (Babendererde, 1998), see Fig 4.10.

Polymers are substantially large and long molecule chains connected together with a high number of monomers. Homo-polymers are obtained from the polymerization of a single monomer base unit; copolymers are obtained from two or more different monomers. A polymeric material can exist in different forms as a function of: the length of the polymeric chains, the presence and nature of the connections among the polymeric chains, and the existence (or not) of structural molecular groups.

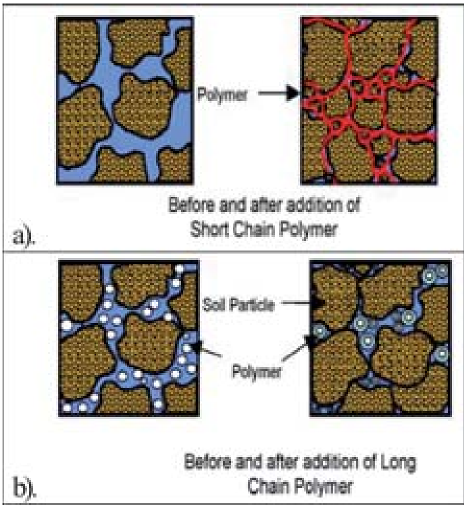


Figure 4.10 Functioning principle of polymers: a) short chain polymer, b) long chain polymer (after Babendererde, 1998).

Polymers able to absorb water can be added to the excavated material when it is necessary to dry it, to correctly handle the muck during the transport phase. The same result can be reached more economically with lime or cement, when the disposal conditions allow it.

FILLERS

Fillers are essentially fine-sand or fine-crushed limestone that have the effect of modifying the grain-size distribution which characterizes the natural ground, making the material in the excavation chamber more heterogeneous.

The face-support pressure is provided by the excavated ground which, during its entry into the plenum, is appropriately conditioned. Therefore, the important factor is the grain-size distribution of the ground *within* the plenum and not just that of the "natural" ground. This implies that, for a correct homogenised blending of the ground capable of transmitting the pressure to the face, the "crushing" effect of the excavating action has to be considered because it increases the fine's quantity in comparison with the in-situ situation, as well as the effect of the conditioning agents (for example, the use of polymer can vary the grain size distribution).

In some cases, the use of fillers (like crushed fine-size limestone) could become necessary to "improve" the ground quality and rectify the grain-size distribution in the plenum, by compensating a lack of such fines in the original ground. To obtain good grading of the material in the plenum and a good water-tightness in the screw conveyor, the desirable fines content (*into the plenum*) of very-fine grain size (<0.06 mm) has to be >10%. Also, the water content has to be kept between the plasticity and the liquidity limits.

Limestone filler was adopted, for instance, during the excavation of Lot 5 of Turin Metro, Line 1. Laboratory studies were carried out in relation to conditioning of the ground via foams and polymers, and the results indicated the need to use also limestone filler (which had the advantages of being both easy and cheap to procure) and to avoid environmental pollution. The final mix of additives composed of limestone filler and a polymer, prepared in a plant outside the tunnel, was transported via a transit mixer and injected directly at the face through some of the foam pipes; this phase was monitored using the PLC of the EPB Shield.

OTHER CONDITIONING AGENTS

A method to reduce the swelling potential of clayey ground involves the conversion of the existing ground into a less water-absorbing material, normally by changing specific mineral ions composing the clay. Swelling clay contains high portions of montmorillonite, while a less active clay is essentially composed of illite and kaolinite. The addition of potassium chloride is an efficient means of reducing the swelling effect. Additives such as lignosulphonates and complex phosphates can be used for dispelling or diluting the bentonite slurry. Various kinds of oil can also be used to increase the lubricating capacity of the slurry (Milligan, 2001).

4.4.2.4 Test methods for conditioned ground

It is important to understand the effects of the conditioning agents on the ground, for optimizing the treatment in relation to the required result. In that respect, various tests are available; these are summarized below.

- *Foam penetration test*: the purpose is to determine the depth, beyond the cutterhead, to which the foam can be injected. Reaching an optimal value is important, because an excessive penetration, besides a large consumption, would not warrant a pressure gradient that is required to support the excavation face. Conversely, too low a penetration could inadequately contrast the effects due to the presence of water. In this test the foam is pushed to penetrate, under pressure, the sample of ground contained in a cylinder to which a water counter-pressure is also applied (to simulate the presence of water in the ground). The capability to penetrate into the ground is thus measured.
- *Slump test*: the purpose of this test is to evaluate, through the slump, the plastic characteristics of the conditioned material present in the plenum. The test has the advantage to be used as a standard everywhere, in the concrete field. A slump of about 12 cm (Quebaud *et al*., 1998) is suggested to achieve good plastic characteristics in the plenum. Vinai *et al*. (2007) suggest the use of the slump value as a "marker" of the good conditioning.
- *Shearing test*: the test (Quebaud, 1998) consists in spreading on a steel dish a sample of the ground and to "shear" it via a series of blades and measuring at the same time the energy used in such an action. Afterwards, the test is repeated by adding foam (or other conditioning agent) and the corresponding reduction in the required energy is noted.
- *Compressibility test*: also the compressibility of the conditioned material is a factor to be considered, to accurately control the actual characteristics of the material transiting to the plenum. To ascertain it, a cylindrical instrument similar to that used for the penetrometer test is utilized.

- *Friction test:* the purpose of this test is to check the friction between the sample of a conditioned soil and a metallic surface, evaluating the wear and energy consumed at the interface according to the type of conditioning agent used.

4.4.2.5 *Mucking and muck treatment*

When using EPBS, the first phase of removing the excavated material consists in extracting it from the plenum, using the screw conveyor. The screw conveyor acts as a "plug" such that the pressure within the screw conveyor has been reduced to the atmospheric value by the time the muck is discharged. When the permeability of the ground in the excavation chamber exceeds the required limit and the tunnel alignment is under a water table, pressurized water could inflow into the tunnel, through the screw. The screw conveyor offers the possibility to inject conditioning agents for reducing the permeability and, therefore, avoiding the inflow of pressurized water. Because the volume of the muck along the screw conveyor is limited, the conditioning is a rapid and efficient answer for changing the excavated ground properties. Once out of the screw conveyor, the mucking material can be removed and transported to the surface (and ultimately to the disposal area) through various transport systems such as belt conveyor, trucks, or train.

4.5 SOME CONSIDERATIONS FOR THE CHOICE BETWEEN HS AND EPBS

It is now clear that a city machine could be only a Hydroshield or an Earth Pressure Balance Shield, the final choice between the two types of machines depending from the point of view of the peculiar, specific characteristic of the actual project to be faced.

But different manufacturers make different type of machines with different characteristics, even if they are Hydroshields and EPBS, making the choice more difficult.

The question of the final choice between the two types of machines has been discussed in so many articles, conferences, or congresses, that it should be the subject of a dedicated book (see, for example, "Boucliers Pressurisés – Boue ou Pression de Terre?" Proceedings CEIFICI, 1997).

In any case, the criteria for the choice are linked mainly to a series of elements which are summarized below and further discussed in the next subsections.

- Application field (types of ground).
- Excavation head or cutterhead.
- Grouting of the tail void.
- Control system.
- Muck treatment.

4.5.1 Field of application

The topic, "field of application" for Slurry Shield, SS, and EPBS Machines, has interested many authors, who tend to separate the ground into two categories: one that can be readily excavated using SS, and the other using EPBS, with some overlapping ranges of ground conditions.

Such a division is often made by comparing the particle size distributions of the various types of soils to be excavated in a standard diagram; and the discriminating factor is the content of the fine grains in the soil.

In choosing the right type of machine there are some theoretical aspects, related to the typical functioning principles of the two different types of machines, which need to be considered:

- SS applies the face-support pressure through the formation of a "cake", between the slurry and the ground. The higher the ground permeability, the more difficult is the formation of a cake. So there must be an upper limit to the particle size of the ground in the face to be excavated, which is strictly related to the permeability. But also, the finer the particle size, the more problematic the functioning of the separation plant. Therefore, it is also necessary to impose a lower limit to the particle size, just from an operational point of view.
- EPBS applies the support pressure directly to the face using the resulting muck, but it is clear that even the word "pressure" in this case is difficult to understand when excavating a stiff ground with a density higher than 14 kN/m3! The face-support pressure can be managed if the material in the plenum is or can be transformed into a sort of "paste" or "high density slurry", for example, by a mixture of clay and water. If the clay-water mix alone is not adequate, something else must be added to the excavated ground to modify its characteristics (see subsection 4.3.2.5 and the example in Section 8.4).

Furthermore, one of the most important things for functioning in EPB mode is to create a pressure gradient in the screw conveyor (the "plug"), in order to dissipate the pressure value at the bottom of the plenum from the actual value down to zero at the discharge gate, avoiding potential water inflow. This is possible only if the ground moving into the screw has an acceptable composition and permeability.

Without conditioning, the EPBS application is problematic in relatively loose grounds and in the presence of a small percentage (5–10%) of fine particles, while the SS could have some problems in material having high to very high permeability; both types of machines have problems in the presence of boulders with dimensions or quantities too large to be handled by the machines.

Recent examples (i.e. Turin metro) have demonstrated that, with the appropriate addition of fillers and/or additives, EPBS can be efficiently applied even in grounds outside its theoretically applicable field. For instance, in coarse soil, the addition of fillers, such as bentonite or very fine sand, can solve the problem of pressure dissipation along the screw conveyor. In other types of soil, a heavy use of other conditioning agents such as polymers, water, and foams may be required. Therefore, it is possible to state that, with appropriate additives, the EPBS can be used in a very large range of ground types (Fig. 4.11). However, there are economic limits linked to the cost of heavy use of agents, which need to be taken into consideration.

4.5.2 Cutterhead and excavation tools

In the last few years, the modern cutterheads have been the object of research and development, in order to optimize their performance and limit or simplify the operations for their maintenance.

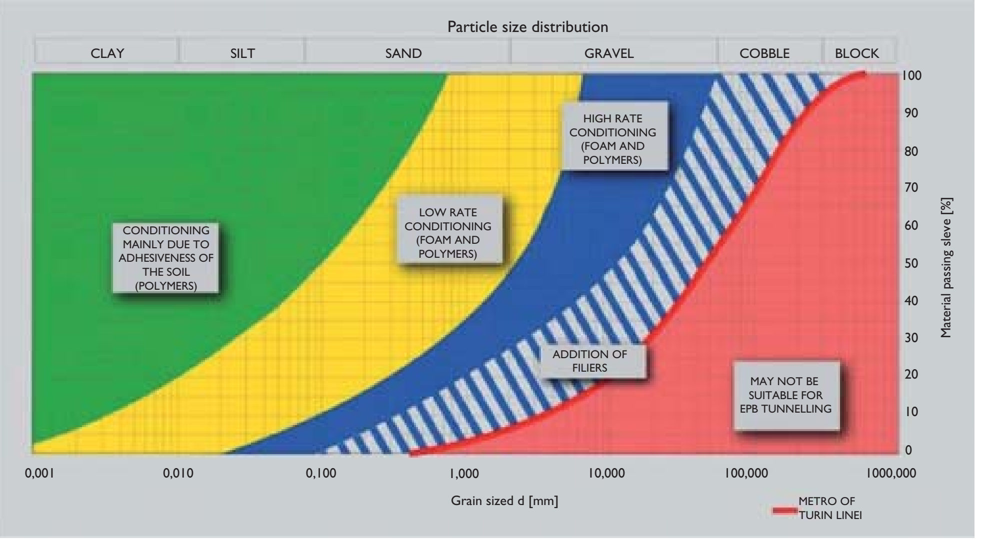


Figure 4.11 Fields of application for SS and EPBS machines and different conditioning techniques.

The position and structure of the excavating tools on the cutterhead influences the capacity for removing the on-site material by a given stroke and rotation speed. The ratio between the opening area in the cutterhead and the excavated section has a direct influence on the mechanical-support capacity of the front and on the face-support pressure control. Usually, for the SS or hydroshield, the cutterhead opening ratio is even more than 50% (see Fig. 4.12a) while for an EPBS it varies between 20 and 35% (Fig. 4.12b). In fact, the formation of cake in a Hydroshield needs a very intimate contact between the face and the slurry. On the contrary, the support effect of the EPB principle is based on the "mechanical" contrast of the muck, which has been accumulated inside the plenum. This implies that the structure of a cutterhead for SS should be lighter than the structure of the one for EPBS: the greater the opening ratio, the smaller the number of cutter tools which can be installed.

The capability for the cutterhead to excavate through boulders has been progressively improved, with the aim of avoiding the entry of the workers into the working chamber for manually removing the pieces that are too large to be removed by the machine (BTS, 2005). However, the situation is that in the presence of many boulders, whose dimensions are close to the maximum allowable for a certain machine, it is possible to install a "stone crusher" in an SS, but not in an EPBS.

4.5.3 Grouting of tail void (or annular gap)

The lining of SS- and EPBS-excavated tunnels in urban areas consists of pre-cast segments (see Section 5.3), which are installed at the end of every excavation cycle inside the rear shield, i.e. in the zone where the hydraulic cylinders apply the thrust for

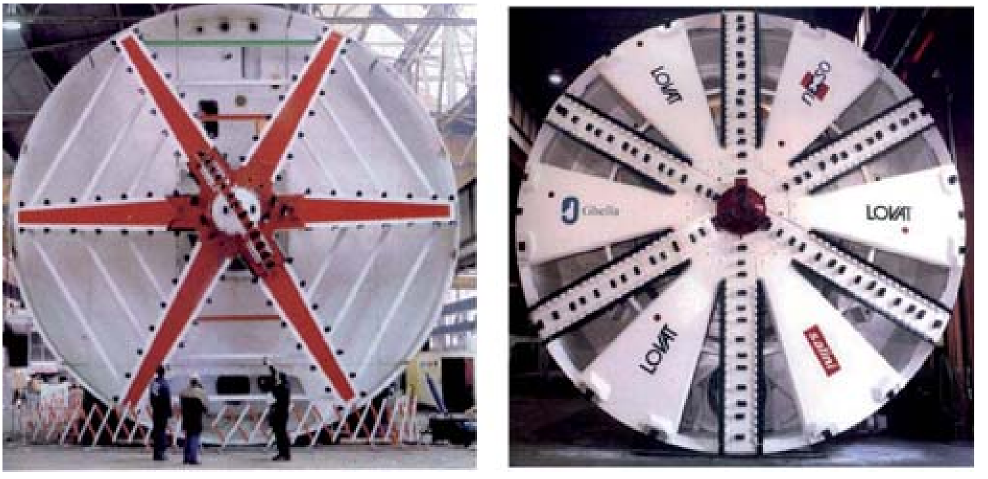


Figure 4.12 a) The Voest Alpine Slurry Shield HDS 925 which operated in Rome (see Appendix 7). b) The Lovat EPBS RME – 370SE which operated in Bologna (see Section 8.6).

advancement. Beyond the erected lining ring, an annular gap, or tail void is created by the following three, main factors:

- The shield's taper needed for an easier advancement (represented by the difference in diameter between the front and the rear part of the shield itself).
- The difference in diameter between the shield's extrados and that of the pre-cast segmental lining installed inside it (including the thickness of the shield and the space required for the tail-skin sealing system, as shown in Section 5.4).
- The over-cutting required for driving the machine through curved stretches.

In order to control the surface settlements, guarantee the correct installation of the segments, and allow an improved and uniform transfer of ground loads to the lining, the annular gap must be filled. This backfilling is commonly done by injecting pressurized cement grout directly from a series of tubes incorporated in the shield's tail through appropriate nozzles (Fig. 4.13). A gasket system (in general, steel brushes, in three rows) avoids the grout extrusion from the gap between the segments and the shield.

In addition, the grout can be injected directly through the segments (second phase injection). The grout is a mixture of cement, fines, water, and additives (plasticizers, retardants, etc.). These different components should be combined in appropriate proportions to achieve the required characteristics of the grout in accordance with the design specifications (see Section 5.4 for details).

In the mechanized tunnelling world, discussion is open on how to fill the space between the excavated profile and the shield extrados along the shield itself (see Fig. 4.13). Clearly, in an SS this space is "naturally" filled up by the pressurized bentonite slurry coming from the plenum. In an EPBS, instead, just some foam, air, and

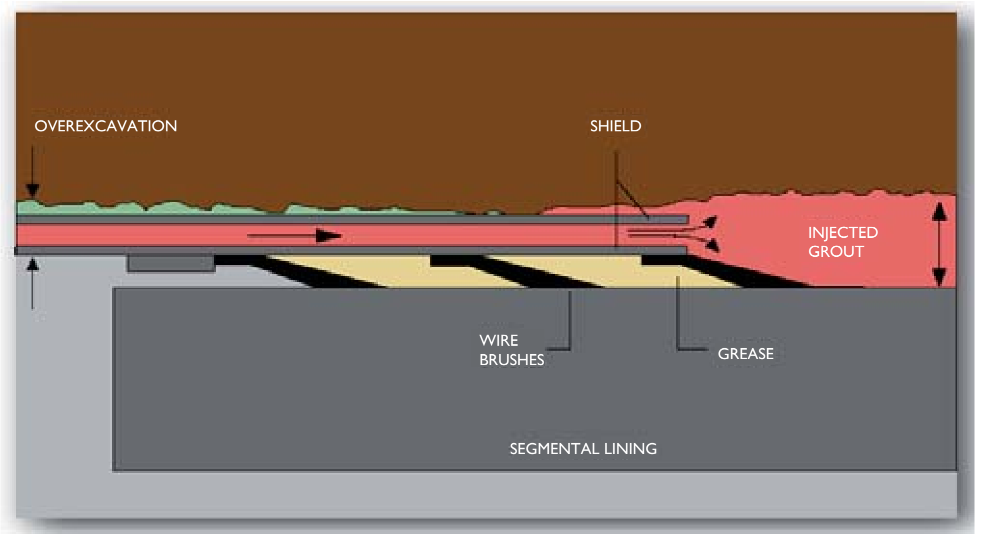


Figure 4.13 Backfilling of the tail-skin void.

water could fill that void, but without providing any support effect. In some projects, a special system has been implemented to fill the annular gap by injecting bentonite slurry at a pressure whose value depends on the face-support pressure, but the practicality of the method has yet to be proven. For sure, this is an important field for future trials and improvement, focused on searching for means to support the ground around the shield. This part of the annular gap represents the last source of potential settlements without any effective countermeasure.

4.5.4 Control system

The computerized-control system (data logger) is used to manage the enormous flow of data continuously acquired from the TBM, furnishing the operator with the needed information for the correct driving of the machine (Fig. 4.14). Generally, the modern machines are equipped with a direct connection to the workstations located at the surface or far away from the job site, where trained users can have continuous access to all registered data provided by the machine. This makes it possible to continuously monitor the machine's performance and to identify any needs for maintenance. It is also useful to acquire all these data for on-line back-analysis that allows the adjustment of the excavation parameters in real time, in order to manage the potentially dangerous situations and avoid their occurrence. Furthermore, in case of an accident, this data logging system makes it possible to understand the causes of the incident.

The following are the fundamental parameters that are necessary for monitoring the process and controlling the excavation (in the form of graphs and tables), while the specific differences between the control methods of the SS and the EPBS techniques are illustrated in Section 6 where the construction control is discussed in more detail.

Figure 4.14 An example of a control panel (left) and a guidance screen (right) in an EPBS machine.

- Pressure in the excavating chamber and along the screw conveyor.
- Excavated and extracted quantities of material.
- Volume and pressure of injected grout in the annular gap.
- Torque, stroke, rotating speed, and advancement velocity of the cutterhead.

4.5.5 Muck treatment

The main difference between mucking system of HS and EPBS is that in the HS, the muck transport is made by pumping a liquid mixture of water, bentonite, polymers, and muck through a pipe circuit connecting the TBM plenum to the separation plant on the surface. In the EPBS, the muck transport is usually done by train or by a continuous belt conveyor. In a few, special EPBS cases, muck has been transported via pipeline and pumping system by adding water to the extracted material.

At the surface, the liquid mixture coming from the HS has to be separated into its *solid* and *liquid* components, in a special "separation plant" (see Subsection 4.4.1.6). The solid part can be transported to the waste pile, and the liquid part can be re-used, by adding fresh bentonite, if necessary, in the mucking circuit.

The material coming from the EPBS can normally be transported directly to the waste pile, without any treatment. In some cases, when the muck has a high content of water, it needs to be dried in temporary disposal areas, by revolving it with special devices.

It is evident that the pumping system, the pipeline, and the separation plant represent an additional element for the excavating cost, from both the points of view of investment and exploitation.

4.5.6 The choice between HS and EPBS

The criteria for making a choice between HS and EPBS in urban areas are affected by many elements, including economical and environmental factors. However, from the point of view of face stability and settlement control, the two techniques have to be considered as equivalent.

Until 2005, the most critical point for the use of an EPBS was related to the mechanical limit of the cutterhead-torque value which is a function of the cube of the excavation diameter. Initially, it seemed impossible to reach the limit of 12-m diameter. Today an EPB Machine with 15.20-m diameter, manufactured by Herrenknecht, has excavated a 3.6-km long tunnel in Madrid (the so-called M30 Project) in about 8 months! A second machine (of 15.10-m diameter, see Fig. 4.15) manufactured by Mitsubishi, has also finished to excavate the second bore of the same project, more or less repeating the Herrenknecht record.

Nowadays, limits on face-support pressure have to be considered. EPB Machines have been used in the Channel Tunnel Project and in the Store-Belt Project, under water tables, with pressure up to 8 bars, while the SS have been used in Elba Tunnel, Westerschelde Tunnel, and St.Petersburg Metro under pressure of up to 6 bars. But these upper limits are not soo significant when working in urban areas, where an overburden of 70 m, like in St.Petersburg, has to be considered quite a rarity.

However, in emergency situations, for example, a collapse of the face, the behaviour of the two types of machines is completely different:

a. In the case of an SS, the collapsing ground can enter into the plenum substituting the bentonite slurry and allowing the creation of a chimney; nothing can be done to avoid it, even if the process is under control, apart from injecting some expandable material, hoping to find the moving "slurry bubble" in time.
b. In the case of an EPBS, collapsing ground cannot enter into the plenum, which is already full of stiff material. If the process is under control, the TBM operator

Figure 4.15 The Mitsubishi 15.1 m diameter EPB machine used in Calle 30 Project in Madrid.

can easily close everything and ask for the intervention of the tunnel manager to adopt the foreseen countermeasures, for example, ground treatment all around the critical zone, which, if successful, could avoid serious consequences on the surface.

Definitely, the choice is the responsibility of the Contractor (unless already specified by the Owner) who will take into account all the relevant aspects of the project, his experience, the possibility of installing a sep aration plant, the necessity to have a cutterhead strong enough to excavate also in rock, the maximum face-support pressure, and the permeability of the ground and its fine-particle content along with all the other design elements.

The really important thing is that, once the choice is made, a skilled TBM driving crew supported by an experienced, control team should implement a robust and integrated control system to accompany the excavation process because:

"The correct choice of machine operated without the correct management and operating controls is as bad as choosing the wrong type of machine for the project" (BTS, 2005).

5

Tunnel design

5.1 PREDICTION AND CONTROL OF TUNNELLING-INDUCED SETTLEMENTS AND ASSESSMENT OF THEIR IMPACTS

5.1.1 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 (1) 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).

The focus of this section is on short-term, tunnelling-induced settlements. 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. m^3 per meter advance). In other words, volume loss/area of cross section × 100 is the expression that is normally used for VL.

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. 5.1a).
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 radially (Fig. 5.1b). 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. 5.1c).
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 radially 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. 5.1d).

The sum of the two radial displacements (Fig. 5.1b, 1c) is termed "radial loss". The sum of the "face loss" and the "radial loss" gives the overall volume loss, V_L, 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 pressurising 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.

Table 5.1 gives a summary of the failure mechanisms in reference to the example of the Porto Metro Project, Portugal (see details in Section 8.3). The construction of

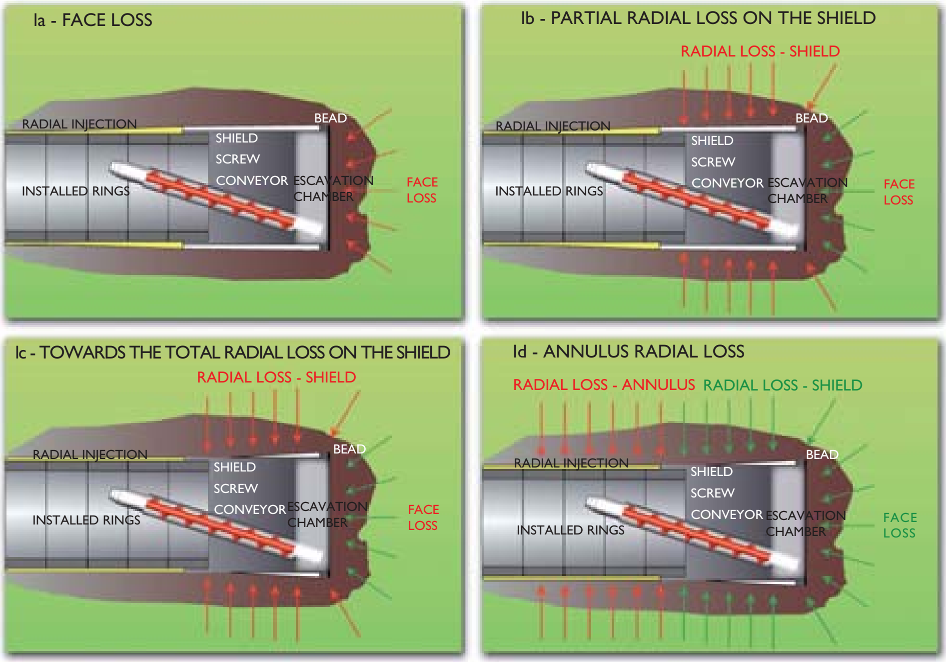


Figure 5.1 The factors contributing to the volume loss.

the underground section of the metro took place in a densely populated area, under about 1700 buildings. The geologic horizon (Porto Granite Formation) was complex, ranging from residual soil to sound, fairly fractured rock, characterized by the irregular presence of corestones, faults, pegmatitic dykes, and loosened horizons. The heterogeneity of the materials resulted in rapidly changing properties within short distances. The locally metastable and collapsible structure exhibited by the residual soil could generate a high potential for collapse of the tunnel face, depending on the high porosity and reduced cohesion of the residual soil. In addition, the ground followed an elastic-brittle-plastic behaviour, leading to sudden, unforeseeable failures at the surface, if the ground was not adequately supported or was over-excavated.

Therefore maintaining face stability is possible, provided it is achieved by establishing a comprehensive procedure consisting of investigations, research, desk studies, calculation methods, and innovations (see Section 5.2).

However, there is a minimum V_L that cannot be totally avoided: that is the radial loss on the shield due to the geometry of the shield itself. This volume loss component is easy to calculate once the geometrical details of the TBM are known. If the ground is weak, and a complete relaxation of the excavated profile is expected on the shield before the tail void injection can intervene and react against ground movements, then the overall radial volume loss on the shield can be transmitted to the surface as settlement. Occasionally, the magnitude of the corresponding surface settlement can potentially cause unacceptable damages to particularly sensitive buildings. If this is the case, then remedial actions have to be taken (see Section 5.1.7).

As mentioned before, the volume loss at the tunnel depth can result in ground loss propagating up to the ground surface by producing settlements. The net volume of the surface settlement trough (V_S) will be approximately equal to the volume loss, V_L, at the tunnel depth in most ground conditions. If the ground response is at a constant volume (i.e. undrained), the relationship will be exact. When tunnelling under drained conditions, e.g. in dense sands, V_S will generally be less than V_L because of dilation (Cording *et al.*, 1975). So it can be roughly estimated that $V_S \cong 0.7\ V_L$. Finally, in loose

Table 5.1 The instability mechanisms during operation of a non-conventional EPB-TBM in Porto Granite Formation (Grasso *et al.*, 2003)

Mechanism	*Description geological context*	*Typical*	*Triggering factors*
Global failure	• Major wedge detachments • Face instability • Instability of the excavation profile	• Fractured rock mass • Very weathered granite • Residual soil • Leached granite	• Inappropriate face support during advance • Over-excavation • Slow reaction to sudden changes of face conditions
Local failure	• Minor wedge detachments • Instability of weathered granite along discontinuities • Instability of pockets of more loose and/or leached granite	• Fractured to very fractured rock mass • Mixed face conditions • Very weathered granite • Residual soil • Leached granite	• Wrong muck conditioning • Incomplete filling of the chamber with excavated and conditioned material • Insufficient compressed air pressure during manned intervention • Difficulties in predicting distribution of very loose soil-like material with routine site investigations
Piping	• Liquefaction of metastable, loose, leached weathered granite or residual soil along discontinuities or prefer-hydraulic channels within the rock mass	• Very fractured rock mass. Persistent fractures filled with residual soil and/or leached granite • Very weathered granite with pockets and/or leached granite	• Internal erosion • Hydraulic and/or mechanical shocks • Difficulties in limiting pressure oscillations in the working chamber • Excessive hydraulic gradient between the excavation chamber and the surrounding environment
Progressive failure	• Progressive propagation of voids above the tunnel resulting in sudden, brittle collapses after TBM passage		• Initial cavities • Over-excavation • Incomplete filling of the tail void with longitudinal grouting

granular soil, or collapsible materials, V_S could be greater then V_L in case of negative dilation.

The magnitude of the volume loss, V_L, depends mainly on the type of ground and tunnelling method. Recent experiences with closed-face mechanized tunnelling (EPB and Slurry Shields) have generally shown that in sands and gravels, a high degree of settlement control can be achieved and small volume losses are recorded (i.e. often $V_L < 0.5\%$), while in soft clays, V_L ranges between 1 and 2%, excluding the long-term settlements.

Leblais and Bochon (1991) reported volume losses in the range 0.2–0.9% for 9.25-m diameter tunnels driven through dense, fine Fontainebleau sands at depths ranging from 22 m to 52 m; values of 0.8–1.3% were observed when tunnel was very shallow with the tunnel crown being only 4.1–7.2 m below the ground surface. Volume losses reported by Ata (1996) for a 9.48 m diameter slurry shield in Cairo at a depth of about 16 m in medium-to-dense sands below the water table were in the range of 0.2–1% with a mean of about 0.5%.

5.1.2 Theory for prediction of the greenfield settlements

Despite the fact that the presence of existing buildings at the ground surface will modify the development of ground movements depending on the proximity of the structures to the tunnel, it is important to understand the development of tunnelling-induced settlements on a greenfield site before considering the added complexity of an existing building.

The problem of tunnel-induced settlements has interested many researchers in the last 40 years and many notable review papers have been published (*among others, by* Peck, 1969; Cording *et al*., 1975; Mair *et al*., 1996, 1997; Attewell *et al*., 1982, 1984; Rankin, 1988; New *et al*., 1991; Leblais *et al*., 1995).

Therefore, the discussions of the greenfield settlement through theories are contained in a number of relevant papers and books to which reference should be made. The theory is not re-discussed here. However, a summary of the main approaches for the case of a single-tube tunnel in a homogeneous medium is given in Tables 5.2 to 5.5.

In the case of twin tunnels, the superimposition of the effects is generally accepted, that is, the settlements induced by the two separate tubes of a twin tunnel are calculated according to the formulas in Tables 5.2 to 5.5; and are summed up to obtain the total settlement (Fig. 5.2a).

However, when tunnelling in soils with a reduced distance between the two tunnels (typically, a separation of two diameters or less between the two axes), the construction of the first tunnel may notably affect the soil conditions: reduced confinement, stress release, and reduction of the strength parameters of the soil. Consequently, the second tunnel will be excavated through a "different material" and the induced settlements related to the second tunnel will be generally greater.

In this case the design should take into account the occurrence of potential soil softening after the excavation of the first tunnel. Therefore, the assessment of settlements induced by the second tunnel could be undertaken, for example, with residual soil parameters.

Table 5.2 Summary of the most common available methods for the assessment of greenfield movements due to tunnelling for the case of a single-tube tunnel in a homogeneous medium: *generalised expression for surface settlements*, Attewell and Woodman (1982)

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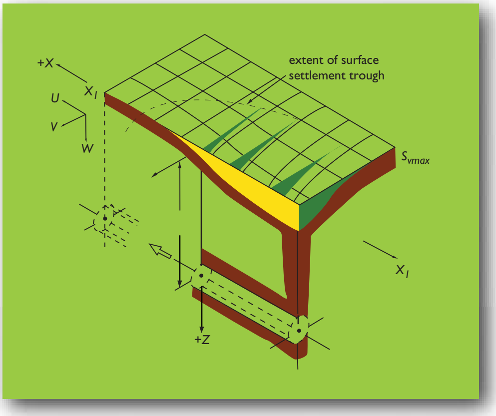
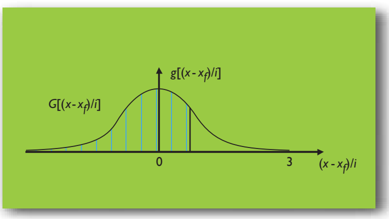
STATEMENT OF THE PROBLEM

Settlements are a 3D problem.
We can be interested in evaluating

- The transversal settlements trough in a certain section while it is developing
- The longitudinal settlement trough along or in a position parallel to the tunnel axis





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GENERALISED EXPRESSION FOR SURFACE SETTLEMENTS:

$$S = \frac{V_s}{\sqrt{2\pi}\cdot i}\cdot e^{-\frac{y^2}{2i^2}}\cdot\left\{G\left[\frac{x-x_i}{i}\right]-G\left[\frac{x-x_F}{i}\right]\right\}$$

where:
S: surface vertical settlement at an (x, y) location [m];
y: distance of the considered point from the tunnel axis [m];
x: longitudinal position of the considered surface point [m];
V_s: volume of the settlement trough per meter of tunnel advance [m³/m], defined as a percentage V_L of the unit volume *V* of the tunnel;
x_i: initial position or starting section of the tunnel [m];
x_f: position of the tunnel face [m];
i: trough width parameter, expressed as: $i = k\ z_0$, where "*k*" is a dimensionless constant, depending on soil type, and "z_0" is the depth of the tunnel axis below surface
G: function defined as:

$$G(\alpha) = \frac{1}{\sqrt{2\pi}}\cdot\int_{-\infty}^{\alpha} e^{-\frac{\alpha^2}{2}}\,d\alpha$$

where:
$\alpha = (x - x_i)/i$
$G(0) = 0.5$ when $x = x_f$ (point above the tunnel face)
$G(1) = 1.0$ when $(x - x_i) \to \infty$
Values of *G* have been already calculated for different values of $(x - x_i)/i$ and they are available in table format.

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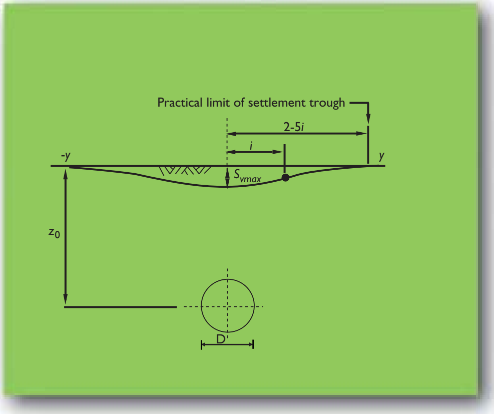
It is generally suggested that numerical modelling be performed to study the twin-tunnel interaction and properly define the rate of stress relief occurring and the corresponding change in the soil parameters (Fig. 5.3). Once the problems are numerically solved, the V_L and *k* parameters that better fit the numerically estimated settlement trough can be obtained and used for extensive settlement predictions by using the semi-empirical method and simply adopting the superimposition of the effects induced by the two tunnels, where V_L and *k* are different for the two tubes (Fig. 5.2b).

Table 5.3 Summary of the most common available methods for the assessment of greenfield movements due to tunnelling for the case of a single-tube tunnel in a homogeneous medium: *derived expression for subsidence trough in a transverse section,* Attewell and Woodman (1982)

STATEMENT OF THE PROBLEM

Transverse settlement trough



SUBSIDENCE TROUGH IN A TRANSVERSE SECTION

If the position 'x' of the considered cross-section has the following characteristics: $(x - x_i)/i > 3$ and $(x - x_f)/i < -3$, then $G[(x - x_i)/i] = 1$ and $G[(x - x_f)/i] = 0$ (i.e. the cross-section is well behind the tunnel face), then the gener-alised expression of Table 5.2 becomes as follows:

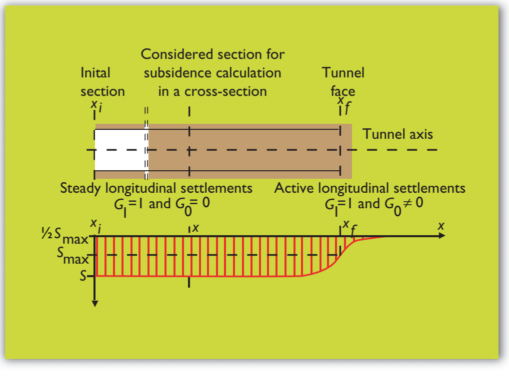
$$S = S_{max} \exp\left(\frac{-y^2}{2i^2}\right) = \frac{V_L}{i\sqrt{2\pi}} \exp\left(\frac{-y^2}{2i^2}\right)$$

Hence, the semi-empirical Gaussian curve expressing the long-term 'greenfield' settlements is obtained, as derived by Peck (1969) from over 20 case histories. It represents the subsidence trough in a transverse section well behind the tunnel face, where max. displacement due to tunnelling are already achieved.

Table 5.4 Summary of the most common available methods for the assessment of greenfield movements due to tunnelling for the case of a single-tube tunnel in a homogeneous medium: generalised expression for *the subsidence trough along the tunnel axis*, Attewell and Woodman (1982)

STATEMENT OF THE PROBLEM

Short-term longitudinal settlement trough



SUBSIDENCE TROUGH ALONG THE TUNNEL AXIS

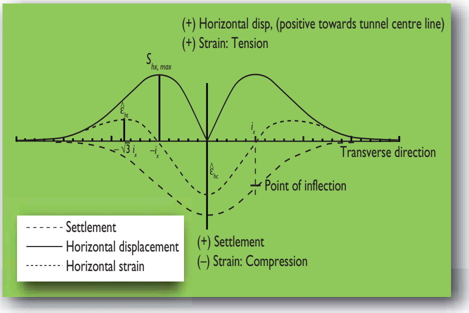
It is obtained from the general equation assuming $y = 0$, that is:

$$S = \frac{V_s}{\sqrt{2\pi} \cdot i} \cdot \left\{ G\left[\frac{x - x_i}{i}\right] - G\left[\frac{x - x_F}{i}\right] \right\}$$
$$= S_{max} \cdot (G_I - G_2)$$

If the starting tunnel position x_i and the position of the tunnel face x_F are known, then it is possible to calculate the vertical displacement for different points located ahead ($x > x_F$) or behind ($x < x_F$) the tunnel face.

When $G_I = 1$ and $G_0 \neq 0$ the longitudinal displacement is a percentage of S_{max}, being the difference $G_I - G_2 < 1$.

Table 5.5 Summary of the most common available methods for the assessment of greenfield movements due to tunnelling for the case of a single-tube tunnel in a homogeneous medium: generalized expression for *surface settlements*, Attewell and Woodman (1982)

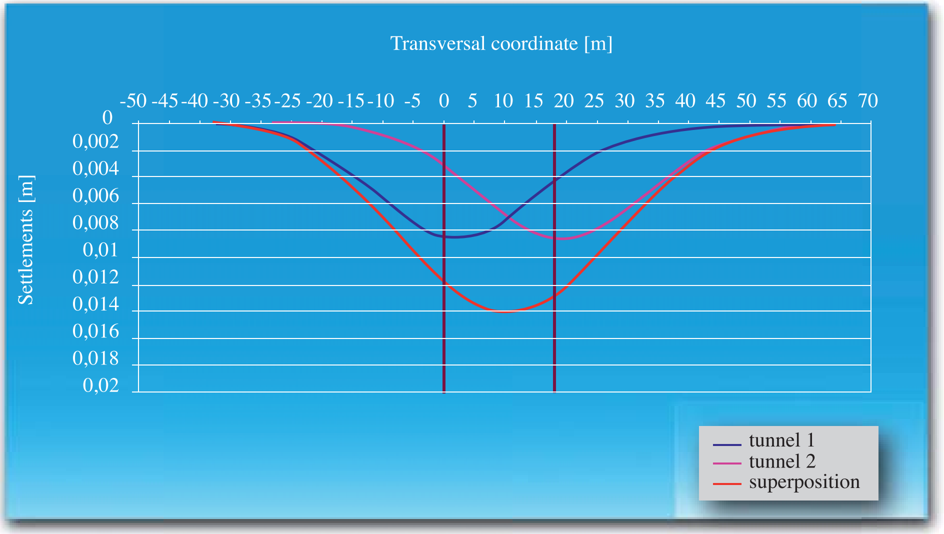
STATEMENT OF THE PROBLEM	HORIZONTAL DISPLACEMENTS AND STRAINS
Horizontal displacements and horizontal strains  	A method for predicting horizontal surface displacements induced by tunnelling was proposed by O'Reilly and New (1982) assuming that vectors of movement near the ground surface were directed towards the tunnel axis. $$S_h = \frac{y}{z_o} S_v$$ or $S_h = S_v \frac{y}{z_0 - z}$ for the subsurface region Horizontal strain at the surface is obtained by deriving S_h: $$\varepsilon_h = \frac{dS_H}{dy} = \frac{S_{max}}{z_0} \cdot \left(1 - \frac{y^2}{i^2}\right) \cdot e^{-\left(\frac{y^2}{2i^2}\right)}$$ The maximum horizontal displacement occurs at the point of inflection, while the maximum horizontal strains develop at $y = 0$ (compression) and $y = \sqrt{3}\, i$ (tension).

The semi-empirical settlement theory, briefly summarised in Tables 5.2 to 5.5, offers the advantages of easy implementation and a good degree of reliability, provided that the basic hypotheses of the theory are respected.

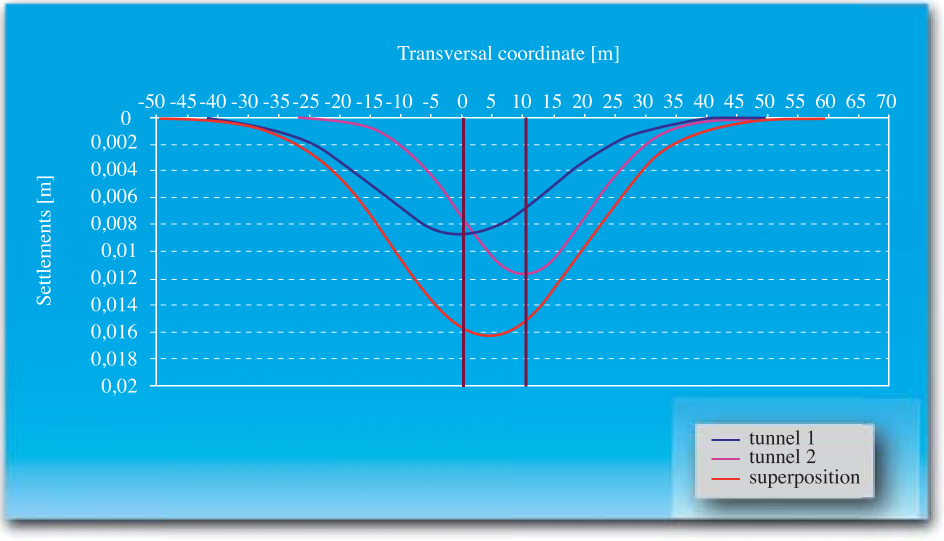
Very often tunnelling in urban areas has one to face more complex geology such as cohesive strata and cohesionless materials and heterogeneous grounds. Consequently semi-empirical theory becomes less reliable. However, the limits of the semi-empirical method can be overcome to a certain extent (Selby, 1988).

In a homogeneous medium the volume of ground moving into the tunnel may be transmitted to the surface, but because of the tendency to spread over a much greater horizontal area during its upwards passage, the magnitude of the resulting movement in any given direction attenuates. At horizons between the tunnel crown and the ground surface, the shape of the settlement trough is similar but the values of "S_{max}" and "i" will vary with depth "z" (Fig. 5.4a). Reference can by made to Table 5.2 for definition of the symbols used in various equations.

In case of strata of both cohesive and cohesionless materials, the profile of the ground movements follows the sequence of the strata (Fig. 5.4b). Based on the calibration through numerical modelling, Selby (1988) suggested using the simple formula (Eq. 5.1):



(a)



(b)

Figure 5.2 Settlement prediction for twin tunnels: (a) no interaction of the tube and superimposition of the effects and (b) interaction of the tubes with reassessment of the ground parameters after the excavation of the first tube.

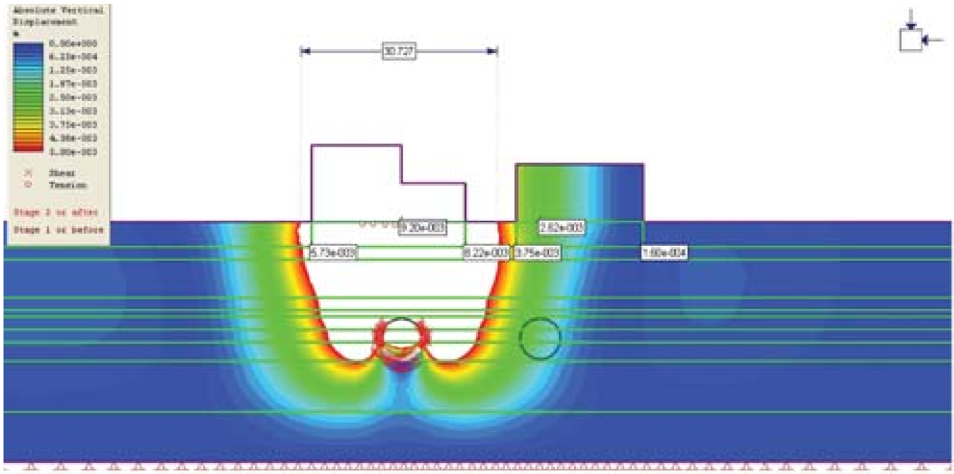


Figure 5.3 Example of numerical modelling to study a twin tunnel interaction.

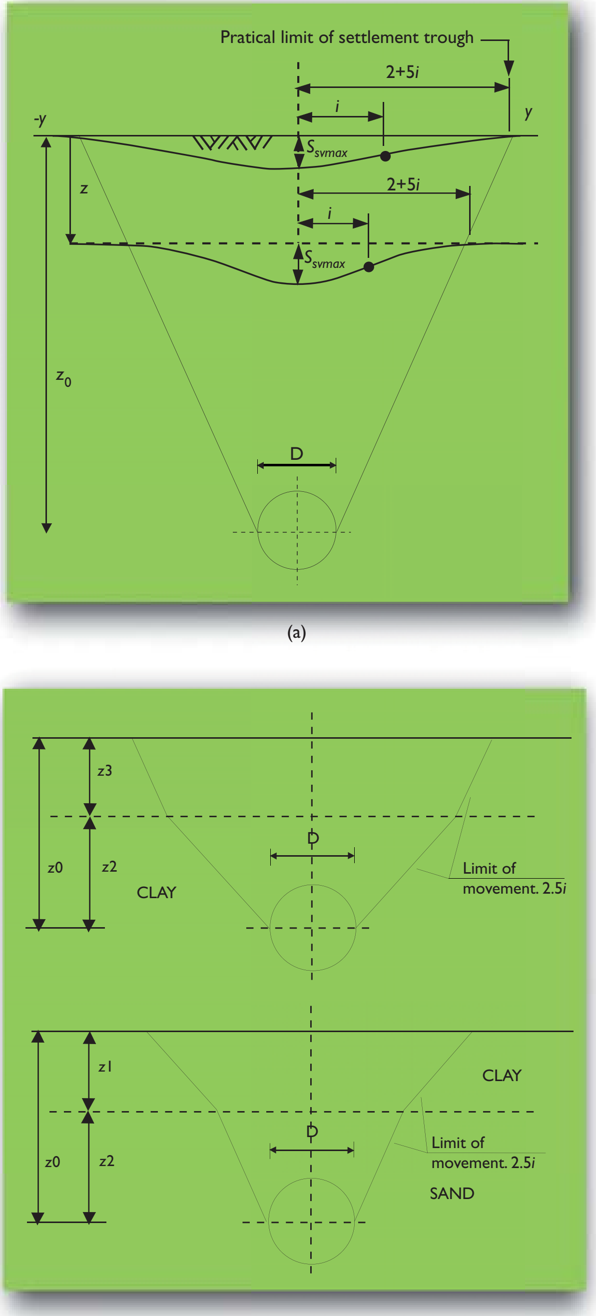
$$i = \sum_{I}^{n} k_n z_n \quad \text{and,} \quad \text{therefore:} \; k_{eq} = \frac{\Sigma_1^n k_n z_n}{z_0} \tag{5.1}$$

On the other hand, it can be readily envisaged that the shallower the tunnel, the bigger the importance of the role played by the strata immediately around the tunnel in controlling the generation of the ground loss and the transmission of settlements at the surface. Thus, for tunnel depth >1.5D, it is suggested to use a weighted formula, to assess k_{eq}, which takes into account the major influence of the ground immediately above the tunnel (e.g. the first 1.5-diameter thickness, see Fig. 5.5). The strata within 1.5D are given a weight, $\lambda > 50\%$, where λ can be initially determined through numerical analyses and then extensively used in the semi-empirical formulations (Eq. 5.2) to provide quick and simple predictions of the greenfiled settlements (Chiriotti *et al.*, 2000).

$$k_{eq} = \frac{(1-\lambda)\cdot(z_1k_1 + z_2k_2 + \cdots\cdots + z_mk_m) + \lambda\cdot(z_{m+1}k_{m+1} + \cdots\cdots + z_nk_n)}{(1-\lambda)\cdot(z_1 + z_2 + \cdots\cdots + z_m) + \lambda\cdot(z_{m+1} + \cdots\cdots + z_n)} \tag{5.2}$$

When dealing with heterogeneous ground, the application of semi-empirical methods becomes more difficult since the intrinsic limitations of the methods become more and more evident. This is true, for example, in the case of tunnelling through a rock mass subject to chemical and physical weathering processes, which may have produced a mixture of sound rock, weak and heavily fractured rock, and residual soils with or without cohesion. The volume loss around the tunnel still occurs, with different magnitudes. However, there are cases where:

- the generated volume loss will not be completely transmitted at the surface because of a partial overburden in good rock;



(a)

(b)

Figure 5.4 Transmission of settlements upwards to the surface: (a) in a homogeneous medium and (b) in a layered medium with different consistency of the strata.

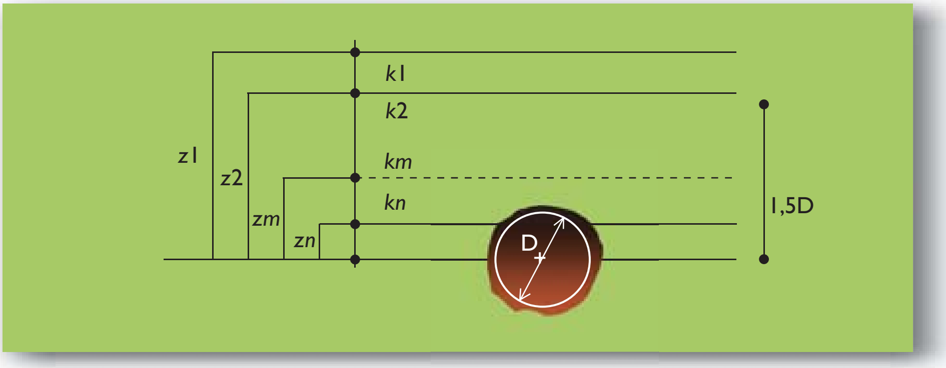


Figure 5.5 Proposed method for calculation of k_{eq} in a layered ground with depth greater that 1.5D.

- the volume loss will be transferred to the weakest material, leading to an asymmetric settlement trough, where the maximum settlement is deviated with respect to the tunnel axis location;
- when tunnelling, even in a rock-like material, the occasional presence of weathered bands or faults will cause potential settlement transmission;
- situation of mixed face condition will potentially lead to bigger volume loss than the case of homogeneous face in soil-like material, due to the extremely different strength of the rocks and to the thrust force being mainly applied on the harder component that can potentially cause over-excavation of the weakest component, and
- the elastic-brittle behaviour exhibited by the rock mass can cause a delay in the settlement response at the surface.

All of the above cases can be properly simulated through numerical analysis and the relevant settlement at the surface can be assessed accordingly. However, the application of numerical methods to assess all the potentially critical scenarios along the tunnel alignment, when tunnelling in a urban environment, can be very time-consuming (and not flexible enough) to be used systematically during construction to give quick settlement analysis feedbacks. For this reason an alternative methodology needs to be used when tunnelling with TBMs in non conventional media. Such a methodology, "Matrix Approach" (Chiriotti *et al.*, 2001), is presented below.

The Matrix Approach consists in identifying, along the tunnel alignment, a set of "control volumes", which are geologically and structurally homogenous, and are meaningful with respect to the geological reference model, including all the adverse conditions for the different failure mechanisms previously identified (see Table 5.1), and exhibit the same response to the excavation in terms of settlement. The geological conditions at the tunnel face, with respect to the overburden conditions, and the various combinations of the face and overburden conditions, are used to develop the Matrixes shown in Tables 5.6. & 5.7. Each identified combination is associated with (1) the most probable set of expected values for the two input parameters used for settlement prediction (i.e. the volume loss V_L and the k parameter, Table 5.6) and (2) the likelihood of such settlements to be transmitted to the surface (Table 5.7).

The values of V_L and k are selected using two simple criteria: (1) the face stability is guaranteed by adequately pressurising the material inside the TBM's plenum and (2) the longitudinal injections of the tail void are executed regularly. In this situation the main component of ground loss is the ground relaxation around the shield, which can be geometrically calculated. Then, assuming, for example, that the corresponding theoretical value of V_L is 1%, V_L = 1% will be associated with the soil-soil face/overburden combination, with cohesion around the tunnel being almost irrelevant. For the mixed-soil face/overburden combination, V_L is increased (e.g. to 1.2%) to take into account a certain face loss due to a possible limited over-extraction. All the other combinations are derived as a consequence.

Table 5.6 Example of Matrix Approach for assessing the volume loss and the k parameters in non conventional media for TBM tunnelling, considering the scenarios resulting from the combinations of face and overburden conditions

Table of V_L and k values		**GEOLOGICAL CONDITION OF THE TUNNEL FACE**			
		1) Soil-like material	*2) Mixed condition (soil and rock mass)*	*3) Faults and/or weathered bands*	*4) Discontinuous rock mass and weak rock*
OVERBURDEN CONDITION	A) Soil-like material	Cohesion around the tunnel: $c = 0 \rightarrow V_L =$ 1.0%, $k = 0.3$ $c > 0 \rightarrow V_L =$ 0.8%, $k = 0.5$	Cohesion around the tunnel: $c = 0 \rightarrow V_L =$ 1.2%, $k = 0.3$ $c > 0 \rightarrow V_L =$ 1.0%, $k = 0.5$	Cohesion around the tunnel: $c = 0 \rightarrow V_L =$ 1.0%, $k = 0.3$ $c > 0 \rightarrow V_L =$ 0.8%, $k = 0.5$	Cohesion around the tunnel: $c = 0 \rightarrow V_L =$ 0.8%, $k = 0.3$ $c > 0 \rightarrow V_L =$ 0.5%, $k = 0.5$
	B) Mixed condition (soil and rock mass)	V_L = 0.5–0.7% (*) Cohesion around the tunnel: $c = 0 \rightarrow k = 0.3$ $c > 0 \rightarrow k = 0.5$	V_L = 0.6–0.8% (*) Cohesion around the tunnel: $c = 0 \rightarrow k = 0.3$ $c > 0 \rightarrow k = 0.5$	V_L = 0.5–0.8% (*) k = 0.5–0.7	V_L < 0.5% k = 0.5–0.7
	C) Faults and/or weathered bands	V_L = 0.4–0.8% (*) (**) Cohesion (***): $c = 0 \rightarrow k = 0.3$ $c > 0 \rightarrow k = 0.5$	V_L = 0.5–0.9% (*) (**) Cohesion (***): $c = 0 \rightarrow k = 0.3$ $c > 0 \rightarrow k = 0.5$	V_L = 0.6–1.2% (*) (**) Cohesion (***): $c = 0 \rightarrow k = 0.3$ $c > 0 \rightarrow k = 0.5$	V_L = 0.4–0.9% (**) k = 0.5–0.7
	D) Discontinuous rock mass and weak rock	V_L = 0.3–0.5% k = 0.5–0.7	V_L = 0.4–0.6% k = 0.5–0.7	V_L < 0.4% k > 0.7	V_L < 0.2% k > 0.7

(*) Potential for progressive failure (see Table 5.1).
(**) Due to the inclined weak zones the tunnelling-induced effects can be transmitted in directions different from the vertical one.
(***) The cohesion is referred to the material within the singularities.

Table 5.7 Example of Matrix Approach for assessing the likelihood of settlement transmission to the surface in non conventional media for TBM tunnelling, considering the scenarios resulting from the combinations of face and overburden conditions

Likelihood of settlement transmission		GEOLOGICAL CONDITION OF THE TUNNEL FACE			
		1) Soil-like material	*2) Mixed condition (soil and rock mass)*	*3) Faults and/ or weathered bands*	*4) Discontinuous rock mass and weak rock*
OVERBURDEN CONDITION	A) Soil-like material	HIGH	HIGH	HIGH	MEDIUM–HIGH
	B) Mixed condition (soil and rock mass)	MEDIUM	MEDIUM	MEDIUM-LOW	LOW
	C) Faults and/or weathered bands	MEDIUM–HIGH (*)	MEDIUM–HIGH (*)	MEDIUM–HIGH (*)	MEDIUM (*)
	D) Discontinuous rock mass and weak rock	MEDIUM–LOW	LOW	IRRE–EVANT	IRRELEVANT

(*) With reference to zones that are not necessarily above the tunnel crown.

The values of V_L and k, assigned to the different face-overburden combinations can be initially checked and adjusted by performing a set of numerical analysis to calibrate the initial picture depicted in the matrix.

Then, the Matrix Approach is also used to indicate the likelihood of transmission of the theoretical settlements to the surface, since in a heterogeneous material the normal volume loss associated with tunnelling does not always have a direct effect on the surface or pre-existing structures, as it is expected in a homogeneous medium. The likelihood matrix can initially contain a qualitative definition of the likelihood of occurrence of the reference settlements at the surface (Table 5.7).

At this point the matrixes can be used to obtain the prediction of greenfield settlements along the tunnel alignment, together with their likelihood of occurrence. During construction from time to time the PAT, Plan for Advance of Tunnel (see also Section 2.6), will include the back-analysis of the monitoring data and the actual recorded settlements to adjust the matrix approach and better fit the encountered scenarios.

When monitoring data are made available during construction, the qualitative definitions in the likelihood matrix could be eventually replaced by F_V and F_K factors. These are multipliers that, starting from the V_L and k values estimated at the tunnel depth, can empirically express how the settlement trough is actually transferred at the surface. F_V and F_K factors will be clearly equal to 1 both for soil-soil and weak rock-weak rock face/overburden conditions, but will be different from 1 in other cases. Where progressive failure can be triggered F_V will be >1 and F_K will be <1; where overburden mix condition can prevent the majority of settlements to be transmitted at the surface or can ease the settlement transmission, then F_V will be <1 and F_K will be >1.

The advantages of the method are mainly related to the reduction of the numerical analyses required and to the number of quick settlement assessments that can be done by still using the semi-empirical method in an empirical, calibrated and project-focused manner.

The method was successfully implemented by the authors in the Porto Metro (see Section 8.3) excavated through the Porto Granite Formation, and recently adapted to the ground in Rome to make the preliminary settlement forecasts and building risk assessment of the new Line D of the Rome Metro (duration: 2006–2007).

Another limitation of the semi-empirical settlement theory, briefly summarised in Tables 5.2 to 5.5, is the way the longitudinal settlement is calculated. According to the simplified theory, about 50% of the maximum vertical settlement S_{max} occurs ahead of the tunnel face. While this assumption is quite appropriate for conventional tunnelling, where the total surface settlement is mainly due to deformations ahead of the tunnel face, it is not adequate to represent the development of longitudinal settlements in mechanized tunnelling (HS or EPBS).

Settlement observations on the centre line above a 6.05-m diameter EPB shield in loose, silty sands and soft clay in Taipei (Mohr *et al.*, 1996) show that most of the construction settlements are associated with the tail void. Very little settlement has occurred ahead of the tunnel face. Similar observations for EPB and Slurry Shield tunnelling, predominantly in sands and silts, are reported by Nomoto *et al.* (1995) in their survey of Japanese shield tunnelling. Pre-settlement directly above the tunnel face of a 9.48-m diameter slurry shield in Cairo, at a depth of about 16 m in medium-to-dense sand overlain by clay, was found to be in the range of 0.25–0.3 S_{max} (Fig. 5.6).

This range is in accordance with the recent case histories: the Porto Metro, the Turin Metro, the Bologna High Speed railway link (Minguez *et al.*, 2005), described in Sections 8.3, 8.4, and 8.6, respectively. The pre-settlement ahead of the tunnel face reached an average 25% of the final settlement in these 3 cases.

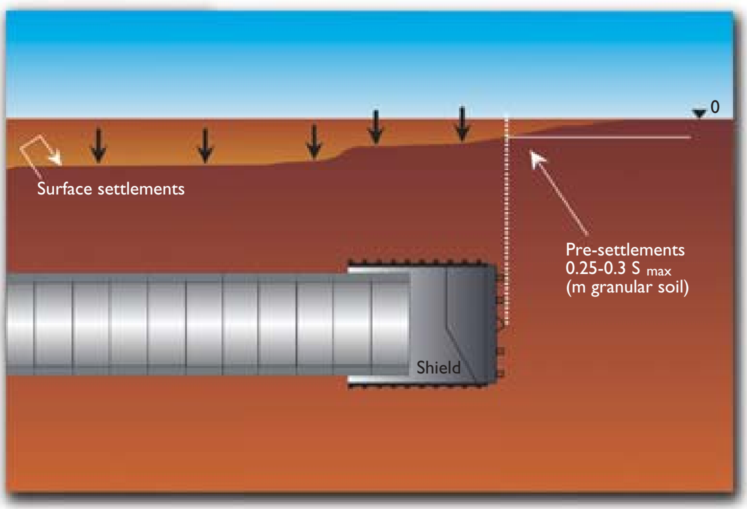


Figure 5.6 Development of longitudinal settlements in mechanized tunneling.

Case history data always have to be verified for any new project, taking into account both the geology and the characteristics of the tunnel boring machine. Once the pre-tunnelling settlement rate ahead of the tunnel face is validated, this information is of paramount importance to establish the settlement trends during construction. Design settlement predictions can be timely validated or adjusted; and potential critical scenarios can be detected in advance and measures taken to control the tunnelling-induced deformations and damage at the surface.

5.1.3 General approach for predicting the risk of tunnelling-induced damages

The process of assessing the risk of damage for buildings, potentially affected by tunnelling-induced settlements, consists mainly of two groups of activities: (a) the Building Condition Survey – BCS – to check the real conditions of buildings prior, during, and after tunnel construction, and (b) the Building Risk Assessment – BRA – to estimate the potentially expected damages on the basis of settlement predictions and the intrinsic vulnerability of the structures.

The general approach is shown in Figure 5.7, and the following steps are foreseen:

- Identify the "control parameters", i.e. those parameters that govern the response of a building to settlements (Fig. 5.8).
- Determine the general criteria for setting limits for the settlement and heave as functions of the specific damage classification system adopted for the project, based on the values assumed by the "control parameters".
- Perform the general ground movement prediction (greenfield movements) to determine the "construction zone of influence" (or "control zone") where buildings have to be analysed to determine their risk of damage (e.g. all buildings within the 5 mm settlement and 1/750 angular-distortion contours, or all buildings at a certain distance on each side of the tunnel alignment).
- Perform settlement-sensitivity analysis (i.e. make an assessment of the condition of each building with respect to how much ground movement it can tolerate before any visible damage starts to appear) for each identified building within the "control zone" and define its tolerance levels in terms of maximum settlement, angular distortion, or deformations.
- Compare the settlement predictions with the results of the settlement-sensitivity analysis and then classify all identified buildings into different risk categories.
- Prepare a Ground Movement Analysis Report to document the expected ground movements and response of adjacent structures and services with consideration of: the ground conditions, the arrangement of the structures, the type of the adjacent structures and uxtilities, and the method of construction.
- Single out the buildings at risk and requiring protection.
- Identify the buildings requiring survey and special monitoring during construction.
- Define the settlement-risk management strategy.
- Archive and maintain all relevant building data in a dynamic and relational GIS data base for use by all parties involved.

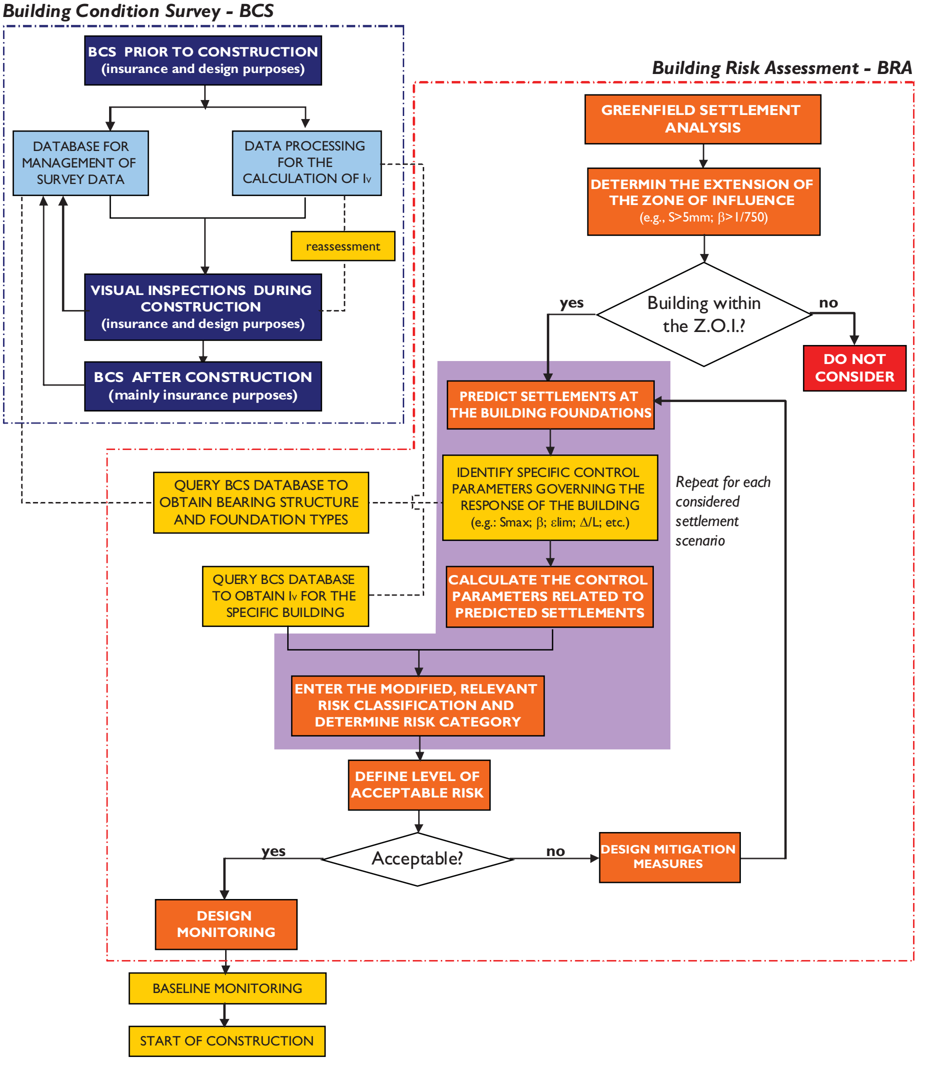


Figure 5.7 Schematic flowchart of the BCS and BRA activities.

5.1.4 Building Condition Survey (BCS)

The condition surveying of all buildings and certain inspectable utilities within the zone of influence of the underground construction works shall consist of distinct stages of surveys to map defects, namely: prior to construction, during construction, and post-construction surveys.

To protect all the parties involved, it is good practice to record the condition of all structures within the control zone, independent of whether damage is foreseen/ has occurred or not. Maybe there are instances where particularly sensitive or important structures which, although apparently not at risk by such a classification, may lie

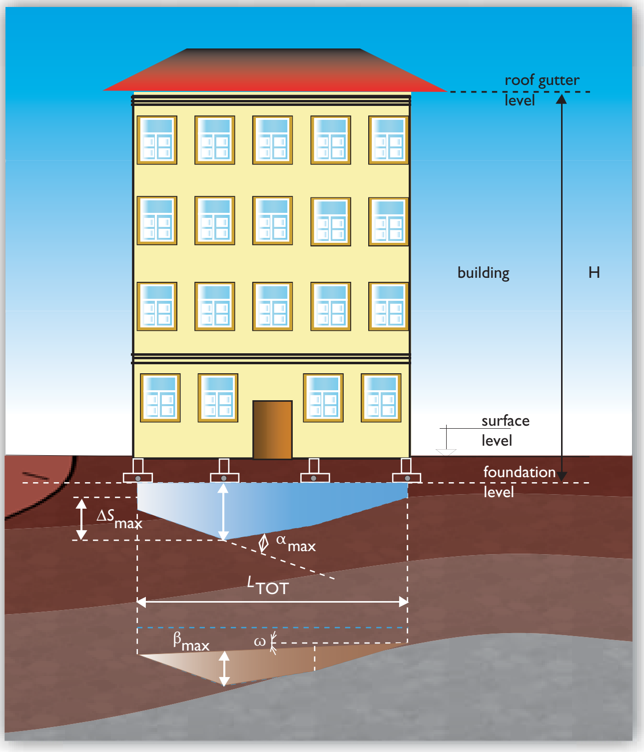


Figure 5.8 "Control parameters" that regulate the behaviour of a building towards settlements. (Burland *et al.*, 1977). Key: S_{max} : maximum vertical settlement, ΔS_{max} : Maximum differential or relative settlement, α_{max} : Maximum angular strain (sagging when positive; hogging when negative), β_{max}: maximum angular distortion, ω:Tilt (rigid body rotation of the whole superstructure or a well-defined part of it), Δ_{max}: maximum relative deflection (max. displacement relative to the straight line connecting two reference points with a distance *L*).

within the overall zone of influence of tunnelling activity and require condition surveys. In addition, an accurate BCS – Building Condition Survey – management is also extremely useful to handle all sorts of potential claims raised by property owners.

The BCS consists in collecting information on the building history and in preparing a map of the building defects that (prior to construction) will be used to assess the vulnerability of the building.

Specifications and procedures, together with forms for the structured, consistent, and coherent collection of data, should be prepared for the BCS surveys and used by a group of competent structural engineers who will perform the work on site.

Each building will be allocated a unique reference number. This is central to the control and communication of information pertaining to each property. All information related to property-schedule pages, sketches, and photographs will carry the unique reference number. This procedure is also facilitating the management of data through databases.

For the prior-to-construction stage of survey, the activities for every single building will focus on the following aspects:

- A cadastral research will be carried out to collect information about: the age of the building, the project drawings, the type and depth of foundations, the number of storeys, type of bearing structure, history of previous repair works, and addition of extra storeys. The research will also determine if the building is inscribed in lists of historic or architectural heritage that will make it a particularly sensitive building.
- A visual inspection of the building conditions: all visible parts of a building will be inspected and reported regarding utilization, observation of cracks and verticality, list of defects, and photographic records. In assessing any building, it is usual to include an inspection of the basement and the roof, and to include a selection of intermediate floors for a high-risk structure. In some instances, it will be desirable to inspect the nature and conditions of the foundations.

The position of a photograph will be recorded on a standard sketch form that will also locate the defect positions. Photographs will at least include: facade identification shot(s); photograph of each elevation; general photographs of interior to establish overall condition; detailed photographs of relevant defects where the use of sketches and descriptive text are not adequate (e.g. areas of dampness penetration, significant signs of structural distress and cracking, complex cracking).

Defects noted during inspection of the structure, fixtures and fitting, will be those referred to as relevant defects, for example: cracks or signs of movement/vibration to the building structures and fabric; signs of dampness within the building associated with the loss of integrity of the structure and fabric; and evident remedial and repair works associated with cracking, movement or dampness. Since the condition survey should be directed at an assessment of the building response, the location, orientation, spacing, and persistence of cracks as well as identification of potential planes of weakness or existing strengthening measures that may modify the structural behaviour will all be taken into account.

In pre-tunnelling surveys, cracks will be broadly classified as follows: hairline cracks (just visible to naked eye), up to 2 mm (minor cracks), over 2 mm (major cracks).

During construction it is often necessary to record the interim condition of a property, produce an updated list of defects, both internal and external, and prepare a Comparative Report for each of those buildings where visible damages have been reported. Photographs are also often used to supplement such an inspection. It is important that construction records are maintained and visual inspections are carried out regularly and systematically. Intermediate condition surveys during tunnelling at appropriate intervals may be required to ensure that building damage is correctly attributed.

The result of the prior-to-construction BCS will be used to assess the vulnerability of the inspected buildings. Vulnerability is an intrinsic characteristic of a building depending on its own history and expressing how far the building condition is from the optimum and perfect one. The higher the vulnerability the lower is its tolerance towards additional, induced deformation before exhibiting a specific type of damage.

It is possible to express the vulnerability through a so-called Vulnerability Index I_V (Chiriotti *et al.*, 2000, 2001), which is derived from an analysis of the information collected during the BCS through the use of engineering judgment. An example of calculation of the vulnerability index is given in the following two pages.

Example of Calculation of Vulnerability Index, Part A

PROJECT NAME	Building code	PAGE
CALCULATION OF THE VULNERABILITY INDEX Chiriotti *et al.*, 2001)	**0001**	1/2

Maxium value: 25

A. STRUCTURAL BEHAVIOUR OF THE BUILDING				
Characteristic	***Index***		***Assumed value***	
A.1. Horizontal structural elements				
A.1.1. Wood structure	6		6	x
A.1.2. Reinfor ced concrete	0			
A.1.3. Mixed structure	3			
A.2. Vertical structural elements				
A.2.1. Masonry elements	6		6	x
A.2.2. Steel elements	0			
A.2.3. Reinforced concrete elements	3			
A.2.4. Mixed elements	4			
A.3. Foundations - source of information				
A.3.1. Direct (drawings, contractor)	0			
A.3.2. Indirect (property owner, inhabitants, for similarity with known structures, assessed)	4		4	x
A.4. Type of refurbishment, if any				
A.4.1. Unknown	2			
A.4.2. Increasing opening in the façade (or bearing walls)	6			
A.4.3. Modifications maintaining the construction method	0			
A.4.4. Modifications improving the construction method	3			
A.4.5. Consolidation (bearing structure or foundations)		5		
A.4.6. Adding floors	4		4	x
A.4.7. Small interior works	0			
State of the refurbishment works (*)				
A.4.a. Done or in progress	1	0	x	
A.4.b. Designed	0	1		
A.5. Presence of basement levels				
A.5.1. No	0			
A.5.2. Yes	3		3	x
			PARTIAL TOTAL A.	**23**

(*) correction factor

Maximun value: 25

B. ORIENTATION AND POSITION OF THE BUILDING					
Characteristic	***Index***		***Assumed value***		
B.1. Orientation	S.T.	L.T.	S.T.	L.T.	
B.1.1. L1 / L2 < 0.5	5	10	5	10	x
B.1.2. 0.5 < L1 / L2 < 2	6	6			
B.1.3. L1 / L2 > 2	10	5			
B.2. Group effect of buildings					
B.2.1. Isolated building Type A (L1, L2 < 2D)	15				
B.2.2. Isolated building Type B (L1, L2 > 2D)	5				
B.2.3. Isolated building Type C (L1 < 2D; L2 > 2D)	10				
B.2.4. Isolated building Type D (L1 > 2D; L2 < 2D)	10				
B.2.5. Grouped buildings parallel to the tunnel axis	0	7	0	7	x
B.2.6. Grouped buildings perpendicular to the tunnel axis	7	0			
B.3. Position (relative to tunnel) factor multiplying B1 and B2					
B.3.1. x/D < 1	1		1		x
B.3.2. 1 < x/D < 3	0,5				
B.3.1. x/D > 3	0				
		PARTIAL TOTAL B.	**5**	**17**	

Example of Calculation of Vulnerability Index, Part B

PROJECT NAME	Building code	PAGE
CALCULATION OF THE VULNERABILITY INDEX Chiriotti *et al.*, 2001)	**0001**	2/2

Max. value: 10

C. FUNCTIONAL BEHAVIOUR OF THE BUILDING			
Characteristic	*Index*	*Assumed value*	
C.1 Use of the building			
C.1.1. Highly sensitive building (hospital, building with sensitive instrumentations, monument, historical building)	10	10	x
etc.)	5		
C.1.3. Low sensitive building (parkings, abandoned buildings)	0		
		PARTIAL TOTAL C.	**10**

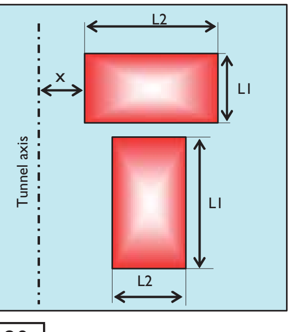
Maximum value: 20

D. AESTHETIC FEATURES OF THE BUILDING			
Characteristic	*Index*	*Assumed value*	
D.1. Historic / artistic heritage			
D.1.1. No	0		
D.1.2. Yes	12	12	x
D.2. Internal not bearing walls			
D.2.1. Wood	1	1	x
D.2.2. Bricks	4		
D.2.3. Cartongesso	3		
D.2.4. Alluminium and glass	2		
D.3. External finishes			
D.3.1. Artistic tailing	4	4	x
D.3.2. Ordinary tailing	3		
D.3.3. Plaster	2		
D.3.4. Other	1		
		PARTIAL TOTAL D.	**17**

Max.v.: 20

E. STATE OF THE BUILDING			
Characteristic	*Index*	*Assumed value*	
E.1. General visual conditions			
E.1.1. Good	0		
E.1.2. Medium	4	4	x
E.1.3. Bad	8		
E.2. Signals of settlements in the surrounding area			
E.2.1. Yes	4	4	x
E.2.2. No	0		
E.3. Cracks			
E.3.1. Major cracks and extensive patterns	8		
E.3.2. Cracks and some patterns	5	5	x
E.3.3. Isolated minor cracks	3		
		PARTIAL TOTAL E.	**13**

LEGEND
L1: average dimension in the direction parallel to the tunnel alignment
L2: average dimensionin the direction perpendicular to the tunnel alignment
S.T. = short term
L.T. = long term
x = distance of the building from the tunnel axis
D = tunnel diameter



Long-term vulnerability index:	**80**
Shaort-term vulnerability index:	**68**

The relevant building information is systematically input into forms and grouped into "macro-families" (functionality, serviceability, aesthetic quality, bearing structure, and defect characteristics) that summarize the main aspects influencing the sensitivity of the building towards differential settlements and deformations.

Brainstorming sessions and engineering judgments are used to identify the macro-families according to the local quality of architecture and real estate industry.

The index of vulnerability, I_V, can be classified into 5 categories with different degrees of severity, using the following divisions (as used in Tables 5.8 & 5.9) of the normalized scale of 1 to 100:

0–20, negligible; 20–40, low; 40–60, slight; 60–80, moderate; 80–100, high.

Additional criteria are used to adjust the I_V; details of these criteria are a short-term (I_{Vst}) and long-term (I_{Vlt}) index (in reference to the alignment of the tunnel with respect to the buildings), the former being used mainly for buildings lying above the tunnel centreline.

The I_V scale is then used to adjust the available damage classifications by reducing the allowed range of control parameter(s) per damage category: given a certain induced settlement or deformation, a vulnerable building will suffer more damages (= higher risk category) than a building in good condition with low vulnerability.

5.1.5 Building Risk Assessment (BRA)

In order to establish a specific damage classification for buildings, it is worthwhile to define the types of damage a building can undergo. The three generally accepted damage classifications are as follows:

Aesthetic damages are related to slight cracking of the structures, affecting mainly the internal walls and their finishes. Aesthetic damages are easy to repair and generally, redecoration is sufficient to cover the light cracks.

Functional damages are related to the loss of functionality or serviceability of parts of the building (e.g. doors and windows may be stuck and pipelines can be damaged) or of sensitive devices located inside the building (such as precision instruments that are sensitive to differential movements); the structural integrity of the building is not affected, however, the lack of serviceability can have commercial and economic impacts on the building and the activities it hosts.

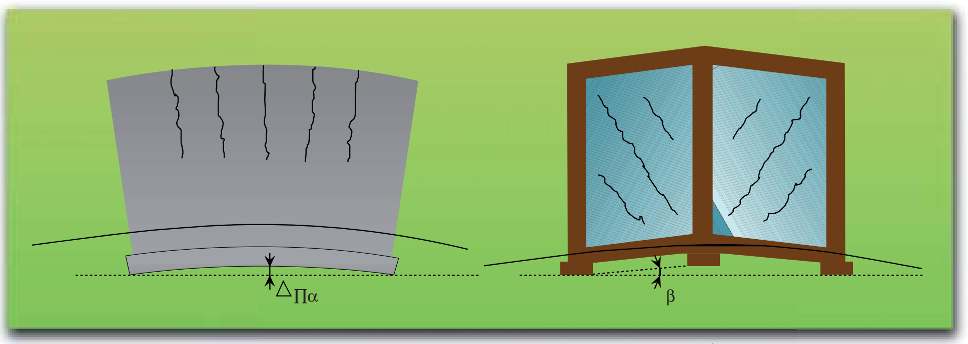
Structural damages are related to cracking or excessive deformations of the bearing structures and can lead to the partial or total collapse of the building. Structural damages can sometimes remain partially hidden beneath the finishes. However, whitewash and plaster are good indicators of the cracking propagation.

The damage classifications available in the technical literature are based both on the type of damage and on the range of values that particular control parameters assume as a consequence of movements induced on buildings by external factors (such as tunnelling). Damage classifications use different control parameters according to the specific type of structures they refer to.

The damage classification proposed by Burland *et al.*, in 1977 (Table 5.8) is mainly applicable to masonry structures and shallow spread foundations and is based on the deflection ratio Δ_{max}/L (Fig. 5.8a) that is related to the maximum tensile strain ε_{max}. It is important to note that, in spite of ε_{max} being the main control parameter, a parallel control has to be performed on the maximum settlement at the level of the building foundations, that should be kept below 250–350 mm (according to building quality) in order to prevent damages related to the serviceability of the building.

Table 5.8 Damage classification established by Burland *et al.* (1977)

Category of risk of damage	*Degree of severity*	*Description of typical damage*	*Crack width [mm]*	*Control parameter (tensile strain) ε_{lim} [%]*
0 aesthetic	Negligible	Hairline cracks	<0.1	0–0.05
1 aesthetic	Very slight	Fine cracks which are easily treated during normal decoration. Damage generally restricted to internal wall finishes. Close inspection may reveal some cracks in external brickwork or masonry.	<1.0	0.05–0.075
2 aesthetic	Slight	Cracks easily filled. Redecoration probably required. Recurrent cracks can be masked by suitable linings. Cracks can be visible externally and some repointing may be required to ensure watertightness. Doors and windows may stick slightly.	<5.0	0.075–0.15
3 aesthetic/ functional	Moderate	The cracks require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Watertightness often impaired.	5–15 (many crack with width >3 mm)	0.15– 0.3
4 functional/ serviceability	Severe	Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some lose of bearing in beams. Service pipes disrupted.	15–25 (but depend on the number of cracks)	>0.3
5 structural	Very severe	Major repair job involving partial or complete rebuilding. Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability.	>25 (but depend on the number of cracks)	



(a) (b)

Figure 5.9 Probable behaviour of different kind of bearing structures undergoing a "hogging mode" type of induced deformations: (a) masonry and (b) framed buildings.

The damage classification proposed by Rankin in 1988 (Table 5.9) applies to framed building with isolated foundations or with pile foundations where the distance among piles is such that the "bearing group-effect" is not triggered. Damage is related to the differential settlements among the isolated foundations and the angular distortion β becomes the most relevant control parameter, that is also accompanied by the maximum settlement S_{max} (Fig. 5.9b).

In general, the threshold between category 2, aesthetic damages, and category 3, functional damages, identifies two distinct families of causes. Damages related to category 2 or lower are the consequence of a combination of causes related both to the intrinsic behaviour of the building (plaster or concrete shrinkage, thermal variations, intrinsic elastic deformations, etc) and to the differential movement of the ground. Hence, this type of damage can be completely independent on tunnelling induced movements. On the other hand, damages related to a category higher than 2 are certainly related to external causes.

Finally, it should be pointed out that the above damage classifications are valid for buildings in good conditions, i.e. without initial defects. As a consequence, the classification should be corrected to fit the condition of the buildings that show relevant defects. Corrections for the damage classification should be established on the basis of building defect (or condition) surveys (see Section 5.1.3).

In case of the Porto Metro Project (described in Section 8.3), it was decided to lower the range of the control parameters in the two damage classification schemes (Tables 5.8 & 5.9) by dividing the value of the control parameters by a reduction factor F_R which ranges from 1.0 to 2.0 (Chiriotti *et al.*, 2000), in relation to the Vulnerability Index, I_V (Tables 5.10 & 5.11). However, F_R should be decided case by case for each Project. The rectified classifications for this project are shown in Tables 5.10 and 5.11. It should be noted that for $0 < I_V < 20$ the reduction factor is equal to 1.0 and, therefore, the original classification (of Tables 5.8 & 5.9) is retained.

It was observed that the majority of the inspected buildings were characterised by a negligible or low I_V. Consequently, the approach was not over-conservative, but

Table 5.9 Damage classification established by Rankin (1988)

Category of risk of damage	*Degree of severity*	*Description of typical damage*	*Control parameters*	
			β_{max}	S_{max} *[mm]*
1 Aesthetic	Negligible	Superficial damage unlikely.	<1/500	<10
2 Aesthetic	Slight	Possible superficial damage which is unlikely to have structural significance.	1/500–1/200	10–50
3 Functional	Moderate	Expected superficial damage to buildings and expected damage to rigid pipelines.	1/200–1/50	50–75
4 Serviceability and structural	High	Expected structural damage to buildings and damage to rigid pipelines; possible damage to other pipelines.	>1/50	>75

identified and treated all buildings that were extremely sensitive and in a relatively poor condition.

The methodology established for the building condition survey (BCS) and building risk assessment (BRA- see Section 5.1.4) in the Porto Metro tunnels was used in 2000–2003 by the Client to establish the state-of-the-art standard to be adopted by all of the station designers. The BCS-BRA methodology was recognized also by the LNEC (Laboratório Nacional de Engenheria Civil, Lisbon). The approach has since been used as a reference in the Technical Specifications of the Athens Metro Extensions, the Tel Aviv Red Line Metro, and the new Line D of the Rome Metro (at the study level).

The assessment of the control parameters is performed for each building within the control zone, based on the prediction of settlements at the level of the building foundations. The semi-empirical approach described in 5.1.2 for the calculation of the greenfield settlement is used, where z_0 is now the depth of the tunnel axis with respect to the foundations of the building. This simplified approach is based on the following assumptions:

- the problem of tunnelling beneath a building is considered 2-dimensional;
- the building deforms according to the greenfield settlement trough;
- the possibility of the damaged structure altering its stiffness and ground interaction is not allowed for, and
- for calculation of the tensile strain, the building is assimilated to an ideal beam having a length L, a height H (distance between the foundations depth and the roof-gutters elevation) and a Poisson's ratio of 0.3.

Since the interaction problem between the building and the ground is considered 2-dimensional, a set of calculation sections for each building within the control zone has to be selected, mainly parallel and perpendicular to the tunnel axis (Fig. 5.10).

The analytical methods for assessing ε_{max}, S_{max} and β_{max} are summarized in Table 5.12. These methods are especially useful for a preliminary assessment of the behaviour of a building under a certain settlement scenario.

Once the control parameters have been calculated for the specific building as a result of the expected settlement trough (also called "reference settlement scenario")

Table 5.10 Burland damage classification (of Table 5.7) adjusted by the use of the vulnerability index I_V for the Porto Metro Project (Chiriotti *et al.*, 2000)

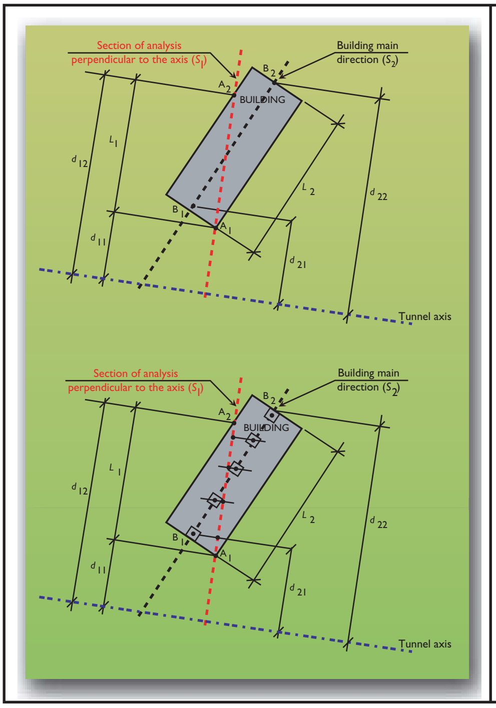
Category of damage	*Vulnerability index I_V of the building*									
	Negligible		*Low*		*Slight*		*Medium*		*High*	
	$0 < I_V < 20$		$20 < I_V < 40$		$40 < I_V < 60$		$60 < I_V < 80$		$80 < I_V < 100$	
	Reduction factor F_R									
	$F_R = 1.0$		$F_R = 1.25$		$F_R = 1.50$		$F_R = 1.75$		$F_R = 2.0$	
	Control parameter									
	ε_{lim} [%]		ε_{lim} [%]		ε_{lim} [%]		ε_{lim} [%]		ε_{lim} [%]	
	min.	max.	min.	max.	min.	max.	min.	max.	min.	max.
0	0,000	0,050	0,000	0,040	0,000	0,033	0,000	0,029	0,000	0,025
1	0,050	0,075	0,040	0,060	0,033	0,050	0,029	0,043	0,025	0,038
2	0,075	0,150	0,060	0,120	0,050	0,100	0,043	0,860	0,038	0,075
3	0,150	0,300	0,120	0,240	0,100	0,200	0,860	0,171	0,075	0,150
4 to 5	>0,300		>0,240		>0,200		>0,171		>0,150	

Table 5.11 Rankin damage classification (of Table 5.8) adjusted by the use of the vulnerability index I_V for the Porto Metro Project (Chiriotti *et al.*, 2000)

Category of damage	*Vulnerability index I_V of the building*									
	Negligible		*Low*		*Slight*		*Medium*		*High*	
	$0 < I_V < 20$		$20 < I_V < 40$		$40 < I_V < 60$		$60 < I_V < 80$		$80 < I_V < 100$	
	Reduction factor F_R									
	$F_R = 1.0$		$F_R = 1.25$		$F_R = 1.50$		$F_R = 1.75$		$F_R = 2.0$	
	Control parameter									
	S_{max} *[mm]*	β_{max}	S_{max} *[mm]*	β_{max}	S_{max} *[mm]*	β_{max}	S_{max} *[mm]*	β_{max}	S_{max} *[mm]*	β_{max}
1	<10	<1/500	<8	<1/625	<6,7	<1/750	<5,7	<1/875	<5	<1/1000
2	10–50	1/500–1/200	8–40	1/625–1/250	6,7–33	1/750–1/300	5,7–28,5	1/875–1/350	5–25	1/1000–1/400
3	50–75	1/200–1/50	40–60	1/250–1/63	33–50	1/300–1/75	28,5–43	1/350–1/88	25–37,5	1/400–1/100
4	>75	>1/50	>60	>1/63	>50	>1/75	>43	>1/88	>37,5	>1/100

at the structure foundation depth, then the expected category of damage can be determined.

However, for those buildings estimated to be "at risk", more detailed calculations should be required and the use of numerical modelling should be enhanced.



❑ SHALLOW SPREAD FOUNDATIONS

The control parameters are calculated at least along two directions:

- direction S_1, perpendicular to the tunnel centreline, intercepting the longest segment (L_1) on the planimetry of the building or passing through the corner of the building perimeter that is closest to the tunnel axis;
- the direction S_2 that coincides with the main axis of the building, being the analysis done along the L_2 segment.

Hence, calculations are performed along segments A_1–A_2 and B_1–B_2, respectively.

❑ ISOLATED FOUNDATIONS

The control parameters are calculated along the same directions S_1 and S_2, but at the location of the isolated foundations (S_1) or at the location of their projection on the segment along which the calculation is performed (S_2).

Figure 5.10 Example of calculation sections for buildings within the control zone.

Once the building risk scenarios have been assessed, a small percentage of the analysed buildings will, generally, fall into the highest categories of damage (>3), so that mitigation measures will be theoretically required prior to tunnelling.

However, the BRA methods used for the preliminary assessment of the risk scenarios are conservative, since they neglect the building-soil interaction and the effects of the building stiffness on the modification of the greenfield settlement trough.

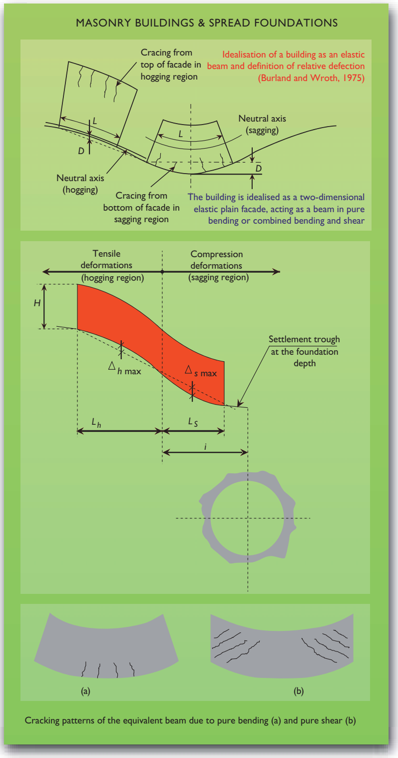
Consequently, it is advisable that, for buildings falling into the highest categories of damage, additional detailed analyses should be performed in order to confirm the preliminary results of the BRA. This will include detailed 2D and/or 3D numerical modelling, allowing for the proper simulation of the soil-structure interaction.

5.1.6 Utility risk assessment

There are practical difficulties in assessing the pre-construction state of utilities and it is likely that assessment of age, type, risk of possible damage, and assessment of safety will determine the need for diversion or isolation from the effects of movement, or acceptance of such movements.

O'Rourke and Trautmann (1982) identified two boundary modes of deformation for pipes (Fig. 5.11): *perfectly flexible* (bending and flexural strain following

Table 5.12 Summary of the available analytical methods for calculating the main control parameters used in the BRA



see Fig. 5.7 for meaning of symbols

Burland and Wroth (1974-a) proposed that building damage could be related to the TENSILE STRAIN, as the maximum between the tensile strain due to bending (ε_b) and the one due to shear (ε_d, diagonal strain). They considered that normally the stiffness and tensile strength of the foundation was enough to prevent the ground horizontal tensile strains from being transmitted to the superstructure.

Boscardin and Cording (1989) highlighted case his-tories where this was not the case. They proposed a modification to take into account the horizontal tensile strain ε_h by simply adding it to the strains due to bending (ε_b) and shear (ε_d).

$$\varepsilon_{max} = \max(\varepsilon_{bt}; \varepsilon_{dt})$$
$$\varepsilon_{bt} = \varepsilon_h + \varepsilon_{bmax}$$
$$\varepsilon_{dt} = 0.35\varepsilon_h + \sqrt{(0.65\varepsilon_h)^2 + \varepsilon_{dmax}}$$

ε_h is obtained by deriving S_H (see Table 2).

$$-i < y < i \rightarrow \text{compression } \varepsilon_h$$
$$y < -i; y > i \rightarrow \text{tensile } \varepsilon_h$$

Expressions for ε_b and ε_d are derived as func-tions of the deflection ratio Δ/L.

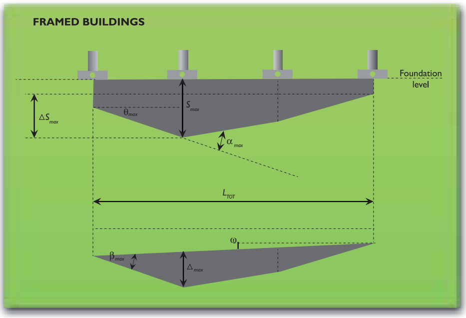
$$\frac{\Delta}{L} = \left\{\frac{L}{12t} + \frac{3I}{2tLH} \cdot \frac{E}{G}\right\}\varepsilon_{b\,max}$$
$$\frac{\Delta}{L} = \left\{1 + \frac{HL^2}{18I} \cdot \frac{G}{E}\right\}\varepsilon_{d\,max}$$

where:

L: length of building in hogging or sagging region on the settlement trough;
H: height of building;
t: distance of extreme fibre in hogging or sagging region ($t = H/2$ in sagging; $t = H$ in hogging);
E,*G*: elastic and shear modulus of the building ($E/G = 2.6$ for masonry *b*.; $E/G = 12.6$ for framed *b*.);
I: moment of inertia of building acting as a beam ($I = H3/12$ in sagging; $I = H3/3$ in hogging)

(*continued*)

Table 5.12 (*continued*)

	The calculation of both S_{max} and β_{max} is related to pure geometrical considerations. The first step is to calculate the settlement trough by using the semi-empirical method. Then the following steps are done: – at the location of each foundation along the calculation section the vertical settlement S is calculated; – the rigid body rotation of the whole superstructure (tilt) is determined; – each couple of subsequent foundation points is considered and the relevant β value is determined; – S_{max} and β_{max} are determined as the maximum among the calculated values.

the settlement trough, which may lead to rupture or intolerable deformations) and *perfectly rigid* with flexible joints (individual rigid pipe sections with rotation and axial slips at joints leading to leakage or disengagement). The performance of pipelines will depend on: history and existing conditions, relative stiffness of pipe and soil, movement capacity of any joints, location of any joints relative to shape of displacement profile, and resistance to shear between the soil/backfill and the pipe.

The majority of old water mains and gas distribution pipes are made of grey cast iron and, in an urban area, these may constitute up to 90% of the existing system. Grey cast iron is a brittle material with a failure strain significantly lower than what could be tolerated by the modern ductile iron, steel or plastic pipes. Since cast iron mains are more susceptible to movements and the limiting criteria for failure are more stringent, they have been subject to considerable study in order to provide design criteria for movements in relation to the effects of tunnelling and other adjacent excavations.

The following guidance on the behaviour of cast iron mains has been suggested (O'Rourke *et al.*, 1982):

- allowable slip at joints = 25 mm;
- allowable rotation = 0.5–1.0%;
- less than 200-mm diameter pipes perform as relatively flexible, and
- over 200-mm diameter pipes perform as relatively rigid.

Many studies have highlighted the importance of strain history on existing cast iron mains. Most of the cast iron mains are old, their installation was often poorly controlled, and the backfill and bedding were poorly specified. It is likely that post-installation pipeline deformations have already occurred. Consequently, old cast iron pipelines can already be strained to a critical level and incapable of sustaining further tunnel-induced deformations.

O'Rourke and Trautmann (1982) approached this subject empirically and derived a tentative relationship between cast-iron-pipe diameter and a limiting value of the slope of the settlement trough, above which damage may occur:

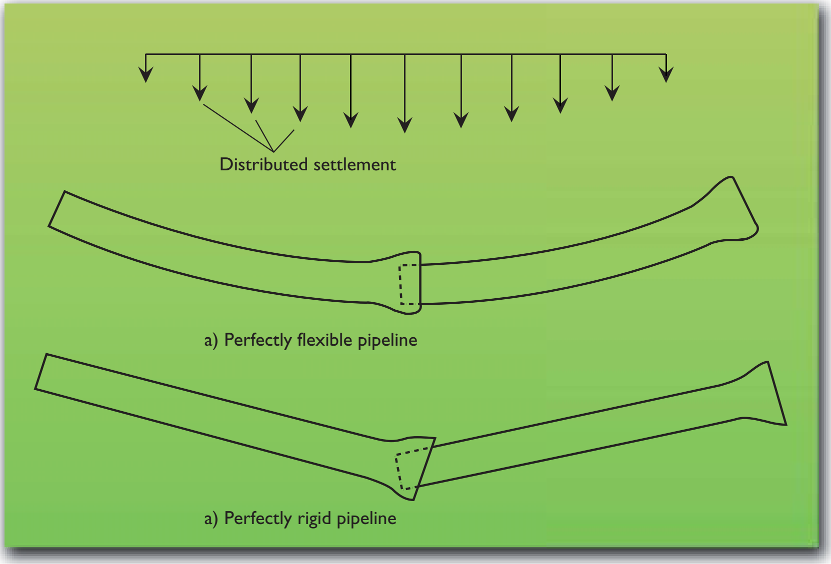


Figure 5.11 Boundary models for deformation of pipes (O'Rourke *et al*., 1982).

S_{max}/ i_y = 0.012 for relatively rigid pipes, D > 200 mm (i.e. β_{max} of 1/140) or,
S_{max}/ iy = 0.02 to 0.04 for relatively flexible pipes, D < 200 mm (i.e. a max value of 1/80–1/40)

However, the database for this relationship is extremely limited and additional information is required before it can be applied with confidence.

A comprehensive methodology for the assessment of the tunnelling-induced movement for rigidly-jointed buried pipelines, in sections parallel and transversal to the tunnel, is given by Attewell and Taylor (1984), while a method for evaluating potential damage to cast iron pipes induced by tunnelling is given by Bracegilder *et al*. (1996).

Finally, it is unusual to require routine instrumented monitoring of pipelines. If there were any residual doubts concerning the safety and performance of the pipelines during tunnel construction, it would have been eliminated normally by adopting positive measures and a conservative approach during the design process.

5.1.7 Presentation of risk-assessment results

Using the results of the BCS and BRA analyses, all the buildings within the control zone will be ranked according to their expected category of damage.

When a GIS (dynamic and relational) database is used to store and manage all the data related to the different phases of the BCS and BRA processes, thematic maps can be easily obtained with reference to the expected settlements, type of buildings and their bearing structures and foundations, distribution of the vulnerability index, and the categories of damage (Fig. 5.12).

The principle of designing through risk scenarios (indicated in Section 3.3.1) can also apply to the BRA. Usually, different scenarios of potential risk of damage could

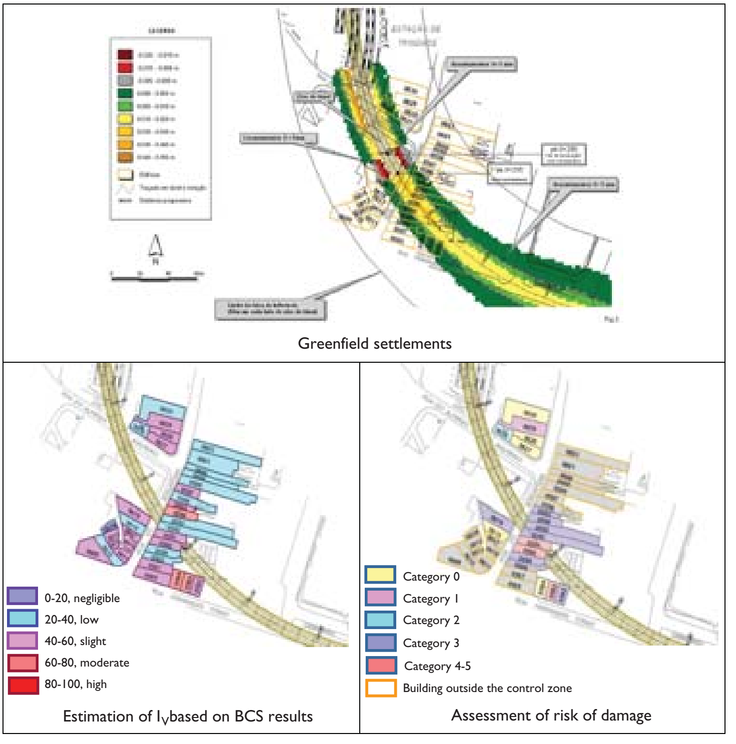


Figure 5.12 Example of the schematic maps obtained with a GSI system that manage the BCS and BRA data.

be defined, by taking into account a range of potential volume losses, obtained by adding an expected variance (typically ±0.2%) to the V_L reference values. The variance is meant to account for minor residual risks (e.g. particularly challenging mixed tunnel-face conditions, temporary plugging of one of the tail-void injection lines, local drop of the face-support pressure for limited periods of time, etc.). It does not include any effect of over-excavation, since one of the BRA assumptions is that tunnelling is performed in a controlled way and risks of over-excavation are avoided.

5.1.8 Use of risk-assessment results and reduction of the predicted initial risk

At the early project stage the results of the preliminary BRA can be used to evaluate if special studies are needed for reducing the volume loss due to the overcut of the shield machine (the so called "physiological volume loss"). Depending on the boring machine and the ground quality, the magnitude of the settlements due to this relaxation can potentially cause unacceptable damages.

Generally speaking, if such a condition extendes to more that 40–50% of the buildings, it is important to intervene directly on the design of the shield itself before it is manufactured. One improvement could be the shortening of its length in order to reduce the corresponding, unavoidable volume loss. Such a mitigation measure was studied, for example, for the Amsterdam North/South Metroline (Van Hasselt *et al.*, 1999). Otherwise, if settlements resulting from the physiological volume loss are critical for only a limited number of buildings (say, up to 20%), then mitigation measures involving both the TBM-operation procedures and the ground and/or building improvement need to be designed and a Building Protection Policy has to be established.

Implementing a Building Protection Policy means associating the actions with the different expected categories of damage. A Building Protection Policy should include making all the appropriate mitigation measures foreseen to be implemented before the tunnel excavation, aiming to reduce the initial damage within acceptable risk categories by acting on the likelihood and/or on the impact related to the initial risk.

Hence, actions will be designed to reduce the risk of damage for those buildings that are considered at risk (mitigation measures) and to control the residual risk for all the buildings within the zone of influence (primary countermeasure).

The Building Protection Policy will vary according to the type of building and will follow a decision making process based on key factors such as: the estimated risk category; the commercial value of the building; the historical value of the building; the use of the building; its state of occupancy; the cost of implementing mitigation measures vs. the cost of doing nothing (and evacuating); etc. An example of a Building Protection Policy is given in Table 5.13 (after Chiriotti *et al.*, 2000); it should be noted that this is a pure example, and the policy has to be studied in detail project by project.

Among the currently available mitigation measures for reducing the likelihood of occurrence of unacceptable damages, four broad groups are described below and illustrated in Figure 5.13:

1. Increase the capacity of the building to sustain additional stresses and the induced settlements by strengthening the structure and thus modifying its response to the effects of tunnelling. This can include propping the building inside and/or outside or underpinning the building foundations. The main forms of underpinning include traditional-mass-concrete, underpinning by grouting, large diameter piles and micropiles, creating reinforced concrete beams connecting the plinth foundations, and using jacking systems.
   Traditional mass concrete could be installed by excavating below the foundation in short sections and casting a new mass concrete footing at a 2–3 m maximum depth beneath the original foundation level. Underpinning by grouting can be used to improve spread foundations and may include: (1) compaction grouting (i.e. injection of stiff mortar into granular soils to fill in the porosity and compact the material around the injection hole) to improve the quality of the ground immediately beneath the original foundations and (2) the execution of jet grouting columns below the improved ground to further stiffen the foundations. Large diameter piles are likely required in order to achieve a total underpinning where the building loads are taken down to below the tunnelling-affected zone for buildings close to the works. They can only be installed externally and then connected to the building foundations via cantilevered pile caps to transfer the loads.
   Hence, a complete access around the perimeter of the building is required. Micropiles can be installed directly from the basement area, using compact drilling

Table 5.13 Summary of the Building Protection Policy, together with the key factors considered for a hypothetical case

Considered factors to outline the Building Protection Policy

Damage (category and type)		*Commercial value*		*Inhabited*		*Historical or sensitive*		*Likelihood of damage occurrence (**)*				*Action Classes (*)*			
		M/H	*M/L*	*yes*	*no*	*yes*	*no*	*H*	*M*	*L*	*I*	*A*	*B*	*C*	*D*
0/1	Aesthetic (***)	n.c.	n.c.	n.c.	n.c.	×				×	×	■			
		(****)				×		×	×				■		
							×	×	×	×	×	–	–	–	–
2	Aesthetic (*****)	n.c.	n.c.	n.c.	n.c.	×		×	×	×	×		■		
							×	×	×	×	×	■			
3	Incipient functional damage	n.c.	n.c.	n.c.	n.c.	×				×	×		■		
		n.c.	n.c.	n.c.	n.c.	×		×	×					■	
		n.c.	n.c.	×			×			×	×		■		
		n.c.	n.c.	×			×	×	×					■	
		n.c.	n.c.		×		×			×	×	■			
		n.c.	n.c.		×		×	×	×				■		
	Functionality and serviceability heavily affected	n.c.	n.c.	n.c.	n.c.	×				×	×			■	
		n.c.	n.c.	n.c.	n.c.	×		×	×						■
		×		×			×			×	×			■	
		×		×			×	×	×						■
		×			×		×			×	×		■		
		×			×		×	×	×					■	
			×	n.c.	n.c.		×			×	×		■		
			×	n.c.	n.c.		×	×	×					■	
4/5	Structural	n.c.	n.c.	n.c.	n.c.	×		×	×	×	×				■
		×		×			×	×	×	×	×				■
					×		×			×	×			■	
					×		×	×	×						■
			×	×			×	×	×	×	×				■
					×		×			×	×		■		
					×		×	×	×					■	

M/H = medium to high; M/L = medium to low; H = high; M = medium; L = low; I = irrelevant.

(*) Action Classes

- A Basic monitoring scheme.
- B Additional instruments to be added to the basic monitoring scheme in order to obtain a detailed measurement of the movements, if required.
- C Detailed monitoring scheme, including real-time monitoring, if required. Visual inspections and BCS records during the excavation of the tunnel (e.g. tunnel face between 20 m behind and 50 m ahead of the building position). Protective measures prior to construction are not foreseen, but countermeasures have to be defined for timely activation when adverse trends are shown. In some cases the material for erecting an emergency propping system has to be made available in the proximity of the building. When dealing with crumbling buildings, potentially subject to functional or structural damage, external protection measures (such as scaffoldings) have to be put in place and the foot-path diverted on the opposite sidewalk.
- D Mitigation measures prior to construction have to be implemented. Detailed monitoring scheme, including real-time monitoring, if required. Visual inspections and BCS records during the excavation of the tunnel (e.g. tunnel face between 20 m behind and 50 m ahead of the building position).

(**) If using the Matrix Approach

(***) Damages due to the normal lifecycle of the building, not related to tunnelling

(****) n.c. = not considered for decision making

(*****) Aesthetic damages potentially related to external factors, including TBM tunnelling

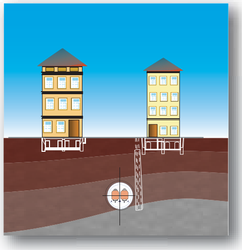
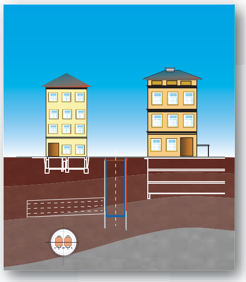
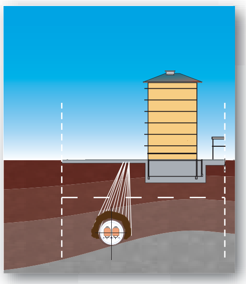
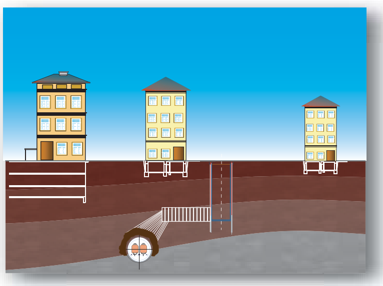
	Sub-vertical shielding structure (injection or jet grouting)	Treatment executed from the surface. A shielding structure is created between the building foundations and the tunnel. Injections and jet grouting columns can be both reinforced with micropiles.
	Sub-horizontal shielding structure (injections, jet grouting columns or compensation grouting)	Treatment executed from a service shaft. A shielding structure is created between the building foundations and the tunnel. Injections and jet grouting columns can be both reinforced with micropiles. Compensation grouting can be pre-scribed for more sensitive cases.
	Subvertical injection (or jet grouting columns) to consolidate around the tunnel	Protection arch on the tunnel perimeter, executed from the surface
	Inclined injection (or jet grouting columns) to consolidate around the tunnel	Protection arch on the tunnel perimeter, realised from pilot tunnel (+ service shaft)

Figure 5.13 Turin Metro Mitigation measures: layout of four different types of interventions.

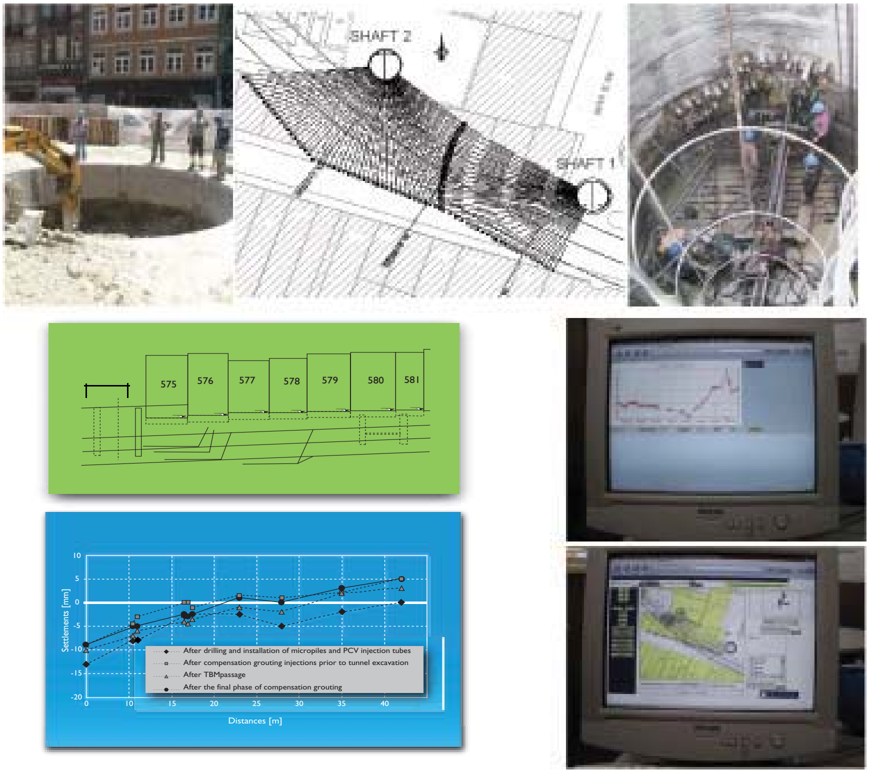


Figure 5.14 Porto Metro Project. Compensation grouting in low cover area and locally collapsible residual soil executed at the final section of tunnel line C; a GIS system was used to manage real-time monitoring and guide the compensation injections prior, during and after tunnelling (Chiriotti *et al.*, 2005).

rigs. The load transfer mechanism is achieved by drilling sub-vertical micropiles through the existing foundation and then connecting them to the foundation. Furthermore, a closely-spaced micropile scheme within the soil mass will also provide soil reinforcement. Jacking systems can be used in conjunction with piled underpinning to permit adjustments to be made in order to compensate for the behaviour of the support system during and after the transfer of loads, or when the existing footings are utilised and adjustments to building movements are achieved by the partial or complete decoupling of the superstructure.

2. Drastically reduce the possibility of tunnelling-induced settlements by shielding to prevent the migration of settlements towards the building foundations. This involves the installation of a physical barrier between the building foundation and the tunnel. The barrier is not structurally connected to the building foundations and, therefore, it does not provide direct load transfer. The intention is to smooth the shape of the settlement trough, thus reducing the effects on the area adjacent to the protected building. The shielding barrier typically consists of consolidated and/or reinforced

ground such as curtain walls of injections, jet grouting columns, micropiles, or a combination of these systems. The barrier can be either sub-vertical, if executed from the surface, or sub-horizontal, if executed from service shafts dug on purpose or from the deepest basements of adjacent buildings.

3. Drastically reduce the likelihood of volume losses that could result in settlement. This is obtained by consolidating the ground around the tunnel crown or the entire tunnel section before tunnel excavation (see Fig. 5.13) by using the most adequate consolidation technique. However, special prescriptions for the TBM drive can also be useful to reduce the occurrence of excessive volume losses. Besides the procedures for face-support pressure and tail-void injection controls, bentonite injections along the shield body can be prescribed, for application at certain locations along the tunnel alignment, to prevent the complete relaxation of the excavated profile on the tail of the shield, thus minimizing ground loss. TBM stoppages, over week-ends or for maintenance purposes, should be properly managed in order to avoid long stoppages at critical locations.
4. Compensate for the settlements. This can be achieved by jacking techniques (see above) or by compensation grouting (Fig. 5.14). The aim of compensation grouting is to create a stiff "pillow" of consolidated soil in a zone of ground between the building foundations and the tunnel crown so that not only the de-confinement of the ground around the tunnel is not totally transferred to the buildings (shielding effect), but also a controlled heave of the foundations can be induced by injecting grout into the "pillow" to balance settlements due to tunnelling, in terms of both the location and the magnitude. The successful application of the compensation grouting technique depends on the accurate real-time monitoring of ground and building movements and on precise timing and exact control of quantity and location of grout injections. The monitoring system and the injection tubes have to be installed well in advance, with respect to tunnel excavation, in order to allow time for the preparation works. Usually three injection phases are foreseen: before the TBM passage, to stiffen the ground; during the TBM passage, to compensate for excessive settlements; and after the TBM passage, to recompress the ground, if needed.

   Stiffening the ground "pillow" may require a combination of compaction (or permeation) grouting and fracture grouting (or *claquage*). The aim of permeation grouting is to improve the strength and permeability characteristics of the ground and prepare it for the subsequent grout injection phase by injecting a fluid grout-mix able to penetrate the pores of the soil according to its permeability characteristics without causing major mass movements. Fracture grouting consists of forcing into the ground (to be compacted) small volumes of a viscous grout with sufficient pressure to create a network of fractures along which the recompression of the ground is achieved.

   When the adequate level of recompression of the "pillow" is achieved, the building starts to react at any further *claquage* such that well controlled and localised movements can be induced, if desired. Usually, before the TBM passage, it is common not only to stiffen the ground up to the point where the building starts to react at any additional *claquage* injection, but to cause a controlled heave corresponding to the expected settlements, so that (during tunnel excavation) settlements will be compensated by this heave and the amount of compensation injections will be reduced.

The design of any mitigation measure will generally include the following aspects:

- development of the typical protection measures, including the necessary calculations and drawings showing their typical layout;
- preparation of the complete set of technical specifications for executing the related works;
- preparation of a short report for every building at risk for which a protection measure is foreseen, proving the effectiveness of the proposed mitigation measures in terms of increased safety and reduced risks, also through numerical modelling, if appropriate, and
- definition of the typical schemes for the monitoring plan.

Finally, the specifications for systematic visual inspections of buildings during tunnelling should be prepared. The check-list used to detect abnormal situations during visual inspections should include at least the following records: presence of new cracks, opening up of pre-existing cracks (evolution of the cracks), pattern of cracks, movements of blocks in masonry structures, movement of coating elements (e.g. tiles), recent plaster fall, condition of windows, doors and pipelines, buckling of floors, buckling of walls, and conditions of basement and roof.

The mitigation measures to reduce the negative consequences of potential damage can also include evacuating or purchasing the buildings. The advantages and disadvantages in terms of costs and social impacts have to be carefully estimated. In case of previously damaged buildings, demolition can also be a valid alternative.

The cost of applying direct mitigation measures will always have to be weighted against the cost and implications of the more radical solutions, which include doing nothing, adopting an observational approach, and dealing with damages and claims as they arise.

5.1.9 Monitoring-design for controlling the residual risks

As indicated in Section 3.3, one of the key elements of the Risk Management Plan, RMP, in urban tunnelling is to make sure that a clear Plan of Controls is in place to manage residual risks during construction, installation, and final testing.

The design of the monitoring system is one of the central elements of the Plan of Controls, together with visual inspections and active recording and interpretation of TBM parameters.

An adequate monitoring system implies that: (1) key parameters and/or indicators relevant to the validation of the design hypothesis, the identified residual risks, the safety conditions, and the quality of the works have to be singled out; (2) relevant monitoring procedures have to be put in place; and (3) threshold values have to be set for all the relevant indicators in order to activate countermeasures, should anomalous trends be detected.

Monitoring is the basis of a flexible design approach by which the design hypotheses are systematically checked, through controls on site, and countermeasures are pre-defined to react when the encountered conditions are different from the reference scenario. Furthermore, the reference scenario of the section already excavated is systematically back-analysed to match the reality which is then used to update the predictions of the reference scenario for the next tunnel section to be excavated (Fig. 5.15).

A rational approach to instrumentation was established by Dunnicliffe (1988, 1992), together with a check-list of fundamental monitoring programme components. Starting from this check-list, and adding the most recent and state-of-the-art experiences gained from mechanized tunnelling in urban areas, the following components can be identified for a comprehensive and effective design of a monitoring system (after Chiriotti *et al.*, 2000):

- Define the project conditions: geology, geomorphology and geotechnical properties; groundwater conditions; and the status of nearby structures and services.
- Define the purpose of instrumentation.
- Choose variables to be monitored.
- Select instruments.
- Identify additional observations required.
- Select instrument locations.
- Select type of readings.
- Predict the likely behaviour to obtain a range of likely responses and to identify threshold values for construction and/or safety control.
- List the specific purpose for each instrument, in accordance with the scope of monitoring.
- Prepare instrument specifications.
- Plan the installation of the instrumentation.
- Define the frequency of readings.
- Assign tasks and responsibilities.
- Manage effectively the monitoring results.

Finally, the rational approach to instrumentation must be accompanied by a calibration phase of the designed monitoring system and the relevant location of instruments and threshold values. Having estimated a range of ground losses and the associated pattern of the likely displacements for a particular tunnel, it is good practice to verify these at

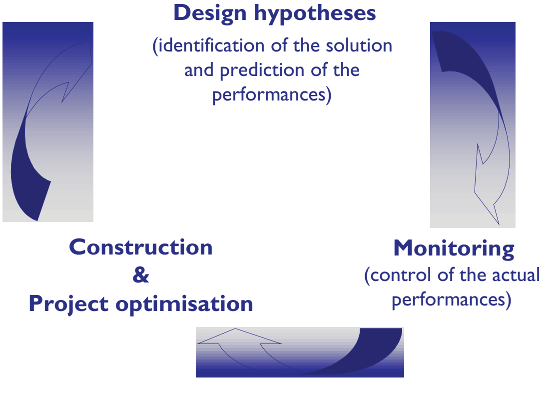


Figure 5.15 Observational method: basic concept of the flexible design approach and the role of monitoring.

the early stage of a project by carefully and heavily monitoring an area representative of the ground conditions. Usually this is accomplished in the so-called "learning curve" section at the beginning of the tunnel excavation, since it is unlikely to incur significant expenses to perform the calibration in a convenient open-space location.

5.1.10 Pre-construction design of countermeasures and mechanism for their activation

In geotechnical engineering, particularly tunnelling in urban areas, to respond effectively to the complex interaction variables, safety, time and cost constraints, a fully integrated design and construction procedure is required. The observational method can basically fulfil this requirement (see also Fig. 5.15), being based on the following strategy (Grasso *et al.*, 1999):

- assessing the most probable conditions and the most unfavourable conceivable deviations;
- establishing a working hypothesis of behaviour, anticipated under the most probable conditions for design;
- identifying and selecting the key-parameters to be observed during construction, understanding the most critical aspects of the project;
- defining the most appropriate instruments and the optimum way for forecasting pointwise the values of each key-parameter;
- at the design stage, simulating the possible crisis scenarios and defining a course of potentuially applicable actions or modification of design (i.e. countermeasures) for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis, and
- predefining the procedures to activate the countermeasures, taking in due consideration the timing factor.

The activation procedures are usually related to the predefined attention or alarm thresholds defined for each controlled key-parameter. Countermeasures have to be activated in case thresholds are exceeded.

When attention thresholds are exceeded in mechanized urban tunnelling, a common countermeasure is to increase the frequency of the readings in order to detect if the alarm value is approached following an expected path or through a potential critical trend leading to soon overcome the alarm threshold. In parallel, a detailed review of the TBM parameters is generally required (see Section 6).

When alarm values are reached and have potentially exceeded, a full review of the design assumptions based on monitoring, visual inspections and TBM data must be performed and, in parallel, countermeasures aiming to prevent critical scenarios will be implemented. Countermeasures will include:

- modify the operational ranges of the TBM driving parameters;
- modify the conditioning agent for better pressurising the excavated material in the excavation chamber for EPB boring machines;
- execute additional probing ahead of the tunnel face and boreholes from the surface to check the geological and hydrogeological conditions;

- propping and even evacuating buildings;
- timely activate the compensation injections or the jacking systems where foreseen, and
- TBM stoppage and execution of consolidation works to allow the safe restart of the excavation.

The countermeasure will have to be selected according to the degree of severity of the critical scenario.

5.2 THE DESIGN OF FACE-SUPPORT PRESSURE

5.2.1 Review of methods for stability analysis of the tunnel face

The stability of the tunnel face is one of the fundamental factors in selecting the method for excavating a tunnel in soft ground and in urban areas.

When using TBMs, evaluation of the face-support pressure is a critical component in both the design and the construction phases. However, specific recommendations or technical norms are not available as common guidance for the design. In current practice, different approaches are often employed, both to evaluate the stability condition of the face and to assess the required face-support pressure.

Estimation of the design value of the face pressure is required for attainment of the conditions of necessary stability for advance of the shield and for meeting other priority needs during the excavation in urban environment, including:

- control of surface subsidence and, in general, preservation of the pre-existing structures, and
- conservation of the hydro-geologic equilibrium.

In addition to the prediction and control of tunnelling-induced subsidence discussed in Section 5.1, the current section focuses on the subsidence component associated with the extrusion of, and deformation at, the tunnel face.

The face-support pressure design must employ a sequential analysis, first, to verify the equilibrium conditions of the excavation face and, second, to identify the consequent stabilizing measures for a complete control of the development of deformations and water inflows.

The available methods for analysis of face stability include:

- Analytical methods: (a) based on the Limit Equilibrium Method (LEM) and (b) based on the earth pressure theory (see details in Sections 5.2.2.1).
- Numerical methods, two-dimensional and three-dimensional (discussed in detail in Section 5.2.2.2).

A comparison of the characteristics of the numerical and analytical analysis methods is provided in Table 5.14.

The term "Global equilibrium pressure" refers to the possibility of verifying the effects of the estimated stabilisation pressure on the face support and surface settlements.

The following observations can be made on Table 5.14:

- Considering the remarkable complexity of the interaction between excavation and soil, only a 3D numerical analysis is, theoretically, in a position to supply a reliable and complete result of the excavation effects and of the consequently needed stabilisation pressure.
- The 2D numerical analyses offer a different perspective for their use in transversal and longitudinal sections. Only in the case of longitudinal sections, it is possible to simulate (with reasonable approximation) the strain behaviour at the face and to evaluate the applied pressure effects.
- The limit equilibrium methods are useful in situations of geotechnical uncertainty, for their relative simplicity of application, and the possibility to readily perform the sensitivity and/or probabilistic analyses. However, they do not provide quantification of the surface settlements.
- The method using the equilibrium-limit states, or the earth pressure theory, is only useful for defining the horizontal theoretical pressure to be applied for maintaining the soil within the limit of deformations (that characterize the on set of active or passive limit states).

The three-dimensional numerical analysis appears to provide the highest potential for the required simulations. However, the employment of relatively simple simulations is of practical use, especially in the initial design phase and during the construction phase, for a rapid simulation of the conditions to be excavated. In general, the optimum approach would be to make a combined application of various methods, with the weight assigned to each method as a function of the design phase and its complexity.

5.2.2 Methodology for calculating the face-support pressure

The following subsections provide details of the methodology used for analyzing the stability of the tunnel face. The analytical methods, based on the equilibrium limit

Table 5.14 Comparison of the different analysis methods

Analysis	*Construction process simulation*	*Face stability*	*Yielding band development*	*Settlement analysis*	*Face-stabilisation pressure*	*"Global" equilibrium pressure*
Numerical 3D	Yes	Yes**	Yes	Yes	Yes	Yes
Numerical 2D T*	No	No	Yes	No***	No	No
L*	(Yes)	Yes**	Yes	(Yes)	Yes	(Yes)
Limit equilibrium methods	No	Yes	No	No	Yes	No
Earth pressure theory	No	No	No	No	Yes	(Yes)

* T, L= transversal section, longitudinal section; ** Face stability is confirmed by measurement of settlements; *** "No" because it is not possible to simulate in this case the effect of the applied face-support pressure on settlement; "Yes" or "No" express the capability of each method to provide results in the categories indicated in top row; "(Yes)" means approximate evaluation only.

theory are discussed in Sections 5.2.2.1, and some applications of the numerical methods are outlined in Section 5.2.2.2.

5.2.2.1 Analytical methods

The methods using the analytical approach can be subdivided into two categories:

- Global limit equilibrium methods, LEM.
- Limit analyses stress methods, LASM.

The global limit equilibrium methods generally factor in:

- The iterative definition of the critical failure surface.
- Assumption of the stress distribution along the failure surface.
- Resolution of the problem through global equilibrium equations of the soil, considered as a rigid body.

The limit analyses stress methods (LASM) perform the stress analysis mainly to provide upper bound and/or lower bound solutions, respectively, from the static and the dynamic point of view.

It is important to emphasise that such methods concur with the definition of the equilibrium limit pressure of the system and, therefore, for achieving the design objectives; in addition, the assumption of an adequate safety factor during the choice of the geotechnical parameters assumes particular importance.

The factor of safety aspect will be discussed later, after a short review of the well-known analytical methods, whose application will depend on the geotechnical characteristics of the ground. For example, in cohesive, saturated soil it will be appropriate to employ the methods where the analysis can be made in terms of undrained shear strength (Broms *et al.*, 1967).

The main characteristics of the selected analytical methods for estimation of the face support-pressure are listed in Table 5.15. References to all these methods are provided in the Reference list in this book. Additional details are provided in Appendix 4.
The analytical solutions provide a useful design instrument, but they are not sufficient for a complete assessment of the long-term stress-strain behaviour of the ground, around the tunnel and on the ground surface. The analytical solutions also serve as complementary solutions for validating the results of the numerical analysis.

5.2.2.2 Numerical methods

2D numerical analysis

The 2D numerical analyses give different results depending on whether they are obtained using the cross section or longitudinal section of the tunnel. When using sections perpendicular to the tunnel axis (cross sections), it is possible to analyse for obtaining the yield zone and the strain development, but not for the face-stability conditions.

Table 5.15 Selected analytical methods for estimation of the face-support pressure

Model/method	*Analysis type**		*Failure surface*	*Failure criterion*
1. Horn model (Horn, 1961)	GE	3D	Linear (Wedge + silo)	–
2. Murayama method (Murayama, 1966)	GE	2D	Spiral logarithmic	MC
3. Broms & Bennemark method (Broms *et al.*, 1967)	GE	2D	Not defined	TR
4. Atkinson & Potts method (Atkinson *et al.*, 1977)	St	2D	Not defined	MC
5. Davis *et al.* method (Davis *et al.*, 1980)	St	2D	Not defined	TR
6. Krause method (Krause *et al.*, 1987)	GE	2D–3D	Circular	MC
7. Mohkam method (Mohkam *et al.*, 1984, 1985, 1989)	GE	2D–3D	Spiral logarithmic + Cylindrical	MC
8. Leca & Dormieux method (Leca *et al.*, 1990)	St	3D	Not defined	MC
9. Jancsecz & Steiner method (Jancsecz *et al.*, 1994)	GE	3D	Linear (Wedge + silo)	MC
10. Anognostou & Kovari method (Anognostou *et al.*, 1994, 1996)	GE	3D	Linear (Wedge + silo)	MC
11. W. Broere method (Broere, 2001)	GE	3D	Linear (Wedge + silo)	MC
12. Caquot method (Caquot, 1956) implemented by C.Carranza-Torres (Carranza-Torres, 2004)	St	3D	Not defined	MC–HB

* GE = Global equilibrium, St = Stress method; 2D, 3D = analytical formulation derived from 2-dimensional, 3-dimensional numerical analyses. MC = Mohr-Coulomb; TR = Tresca; HB = Hoek-Brown.

An example of the use of 2-D numerical analyses in a cross section is shown in Figure 5.16. The surface settlements derived from this analysis can be easily compared with the actual values obtained from monitoring. If necessary, they can also be transformed into the volume loss at the tunnel level and compared with the corresponding values assumed for the design.

However, as previously mentioned, this kind of 2-D simulation does not give information about the extrusion and stability of the face. A reasonable simulation of the stress-strain development at the tunnel face, through the construction process, could be performed by using longitudinal sections of the tunnel. In some cases, it is possible to use the 2D numerical analysis (see Fig. 5.17) to quantify the progressive effect of the stress release on displacements around the face and on the ground surface.

3-D Numerical analysis

The 3-D numerical analyses represent the more sophisticated instruments for construction simulation and verification of the face-stability conditions and settlements.

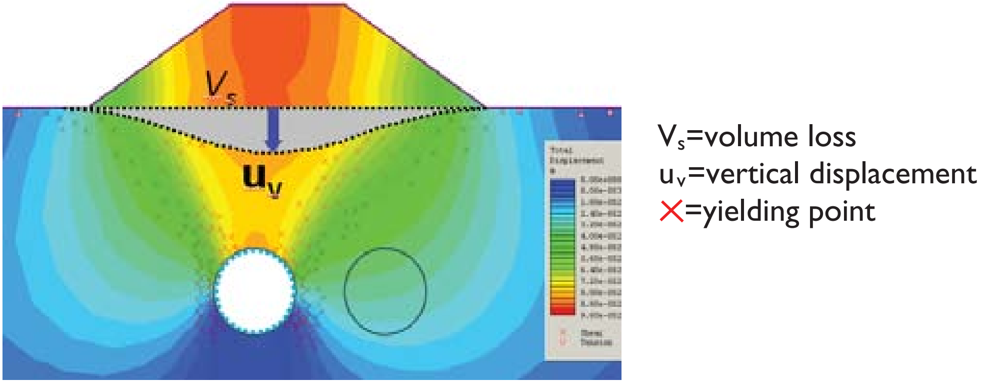


Figure 5.16 Example of 2-D numerical simulation (using a cross section) indicating the yield zones and the subsidence curve.

However, there are some uncertainties that concern the adequacy of the software and the choices in the model configuration (i.e. design of the mesh, dimension of the model, physical parameters input, boundary constraints, simulation of the correct construction phases, etc.).

It is evident that the 3-D simulations require a lot of effort and time, both for the model configuration and the calculation runs. Furthermore, the results reflect the uncertainty and variability of the input parameters. In order to make an optimum use of 3-D analysis, it is necessary to perform a sensitivity analysis to verify the effects of the variation in the input parameters on the relevant output.

Beyond such "basic" limitations, 3D numerical analyses remain as indispensable instruments to model complex situations, such as the excavation of two adjacent tunnels, the interference with structures of particular importance, and the comparison between different alternatives of the design hypotheses.

An example of 3-D model, prepared with the FLAC3D code of ITASCA, is given in Figure 5.18. The example refers to the construction plans of two adjacent tunnels (diameter: approximately 9.4 m) realised with EPBS, of the new urban railway hub of Bologna, a part of the high-speed railway connection Milan-Naples (see Section 8.6).

5.2.2.3 *Equilibrium conditions and Optimum Regime of Advancement*

Obviously, if the collapse of the face is possible or the relative deformation implies unacceptable settlements or interference, it is necessary to apply an adequate face-support or stabilisation pressure. Furthermore, mainly in urban environment, the support pressure should always guarantee a safe advancement, i.e. without risks, covering all the uncertainties and the possible parameter variability. Considering the potentially high impact of a collapse, even a very low probability of occurrence of failure may result in an unacceptable risk. For this reason, the support pressure should not go below a pre-defined safety level.

In regard to the potential of occurrence of the face instability, there are differences between the operations by Earth Balance Shield, EPBS, and Slurry Shield, SS.

In the case of EPBS, as pointed out also by Nishitake (1990), if the muck chamber is properly filled (i.e. the maximum density is achieved) with excavated earth, it is impossible that this compressed earth will be replaced by anything else. Therefore, no material can enter the TBM "plenum" (or "working chamber") and, also, in presence of potential face instability, no collapse will occur. However, when these conditions are not completely fulfilled, a false sense of security may develop. If these signs are

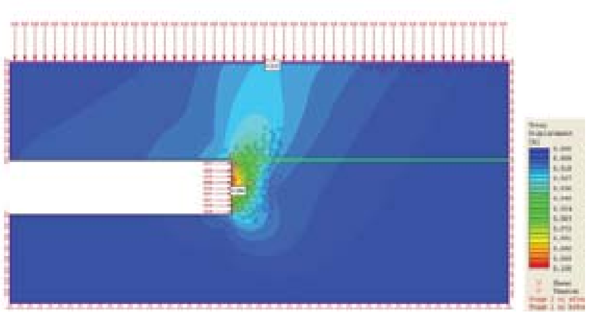


Figure 5.17 Example of 2-D numerical model using a longitudinal section.

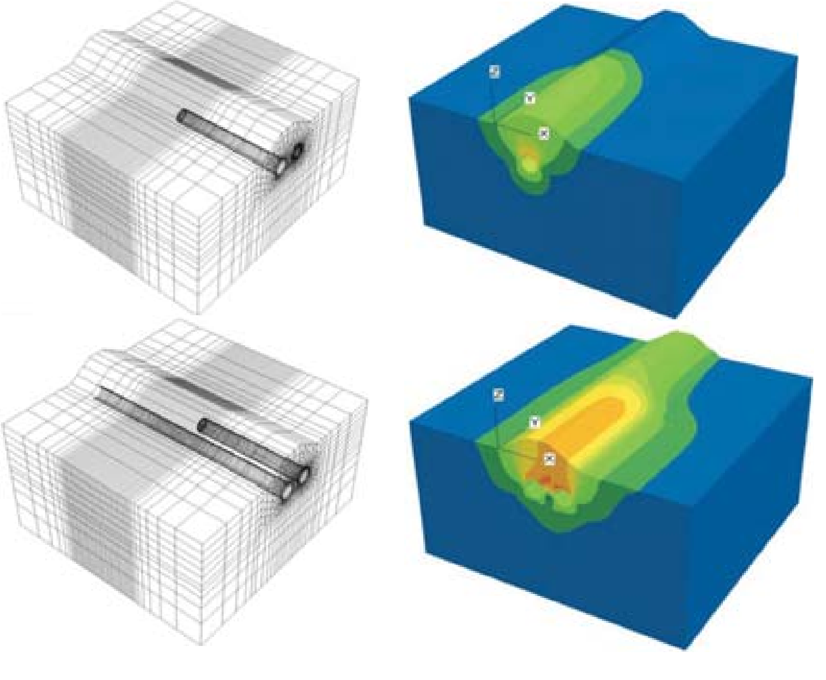


Figure 5.18 Example of 3D numerical model.

ignored, substantially adverse loss of ground may occur and, in the worst cases, has been known to result in chimneys to the surface (BTS, 2005).

When advancing by an SS, the equivalent safe condition can be achieved only if the slurry screen and the slurry cake are formed, and an appropriate face support pressure is applied. Otherwise, if heavy and/or large grains appear at the tunnel face and the pressure is not appropriate, a secure and stable state cannot be guaranteed. The ground can collapse into the TBM plenum, replacing the bentonite slurry, which will migrate up, till the ground surface (Kovari *et al.*, 2003).

The different operational modes of SS & EPBS, explained in detail in subsections 6.2 and 6.3, respectively, also involve different requirements for evaluating the necessary stabilising pressure. For example, as pointed out by Anagnostou & Kovari (1994), while using an SS, the stabilizing pressure can be the total-pressure that is transmitted by the slurry to the formed cake. In the case of an EPBS, a distinction should be drawn between the fluid pressure and the effective pressure in the working chamber (see Appendix 4).

In any case, the assessment of face-support pressure is not a simple matter and it may be useful to better focus on the following factors:

- Equilibrium condition for the advancement of the shield.
- Considerations about the choice of stabilisation pressure.
- Practical construction constraints.
- Importance of adjustment of the design during excavation.

Equilibrium condition for advancement

The discussion in this section refers specifically to EPBS, but the main conclusions may be extrapolated for application to SS. As described in Sections 4 and 6, the EPBS uses a cutterhead operating in front of a chamber entirely filled with the excavated soil. The muck is extracted in a controlled manner from the excavation chamber using a screw conveyor, which governs the pressure of the excavated material and provides earth pressure balance to the excavation face.

The face pressure is controlled by balancing the rate of advance of the shield (proportional to the excavated quantity) and the rate of discharge of the excavated material proportional to the screw conveyor rotation speed.

The equilibrium condition occurs when the muck in the plenum reaches the maximum possible density for applying an active pressure to the face and the volume of the muck extracted by the screw conveyor equals the theoretical volume removed by the cutterhead.

Clearly, if additives are introduced into the plenum to facilitate the mucking process, the volume of the additives introduced should be considered in calculating the volume removed by the screw conveyor.

In this state, the pressure exerted by the cutterhead of the EPBS shall be equal to the static earth pressure, and the ground ahead of the cutterhead remains in elastic domain.

The above observations were also confirmed by laboratory research as briefly described in the following subsections.

Learning from laboratory research

Recently, a synthesis of the results of the French national project "Eupalinos 2000" on "Mechanized Excavation in Heterogeneous Ground" and "Earth Pressure Balance Shield" has been published by AFTES (2001). In particular, as reported also by Russo (2003), the theme B1: "Control of the confinement by earth pressure: Laboratory studies on reduced models" is of interest for the argument discussed here.

The model of EPBS used in the laboratory tests (scale 1:10) and the two different cutterheads employed are shown, respectively, in Figure 5.19 and Figure 5.20.

The tests simulated the excavation into an incoherent, dry soil (fine sands with the angle of friction, $\varphi = 33°$ and the density, $\gamma_d = 13\text{–}17$ kN/m^3), continuously monitoring the pressure in the plenum of the EPBS model and in the surrounding ground, as well as the deformation and settlement induced on the surface.

The following observations are derived from a review of the 11 reports (1998–2001) presented on the above-mentioned Theme B1 of "Eupalinos 2000" French National Project.

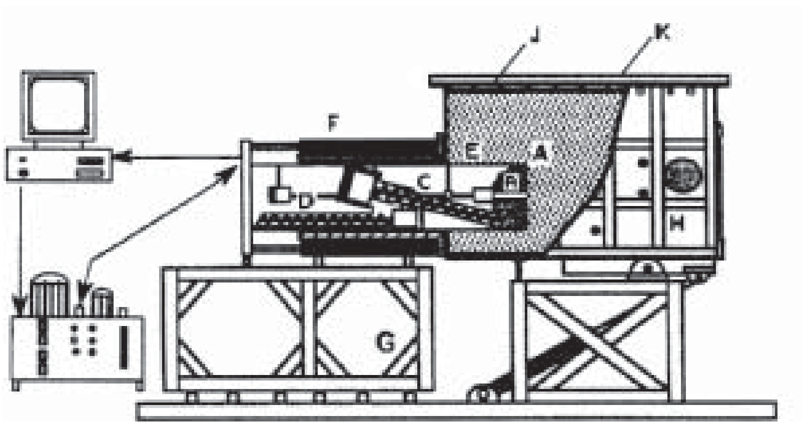


Figure 5.19 EPBS model used in laboratory tests (AFTES, 2001).

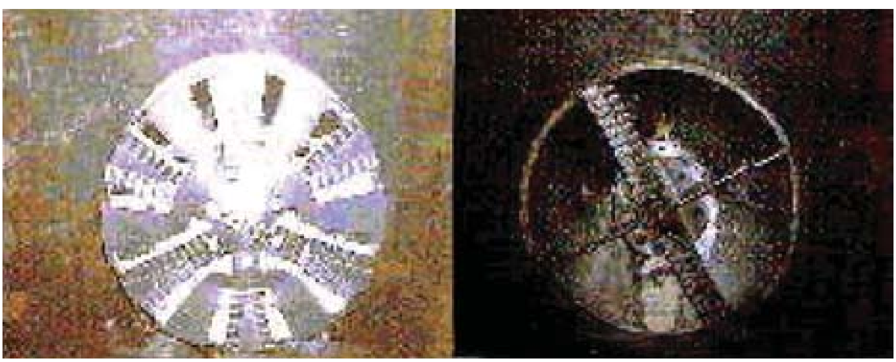


Figure 5.20 Different cutterheads employed in the laboratory tests (AFTES, 2001).

- Essentially two control parameters are able to govern the driving of the boring machine: (1) the ratio, R, between the mass of the actually extracted material from the screw conveyor and the theoretical mass, and (2) the pressure in the ground to be excavated. The first parameter, R, is the key for the advancement.
- The ideal functioning regime is reached when R = 1: this condition is the "Regime of equilibrium" and should be attained by starting the excavation with the adequate confinement and avoiding the plasticization of the ground in advance (Fig. 5.21).
- If the volume of the extracted material is less than the theoretical volume of the material (i.e. R < 1), the passive state develops in the ground, plastic zones develop a few diameters ahead of the face, and the pressure in the plenum increases (Fig. 5.22).
- In the opposite case, when the extracted material is more than the theoretical (R > 1), the ground enters an active state, large vertical deformations occur in the zone between the cutterhead and the surface, and the pressure in the plenum decreases. It is important to observe that the trend of the pressure level to a constant value (even if R > 1) could be temporarily obtained if R is kept constant (Fig. 5.23), but this condition is dangerous, because, despite the pressure value remaining constant, the over-excavation continues.
- As implied by the previous comments, the control of the face-support pressure alone is not enough to establish the actual safe regime of excavation, due to the presence of pressure fluctuations and over- or under-extraction of the excavated material.
- However, experimental data show that when a state of equilibrium is maintained and the pressure in the chamber is stable, its maximum or peak value ranges from 0.9 to 1.1 times the existing pressure of the ground at rest. Moreover, the graphs in the Figures 5.21–5.23 show that the average pressure values are approaching the active earth pressure.

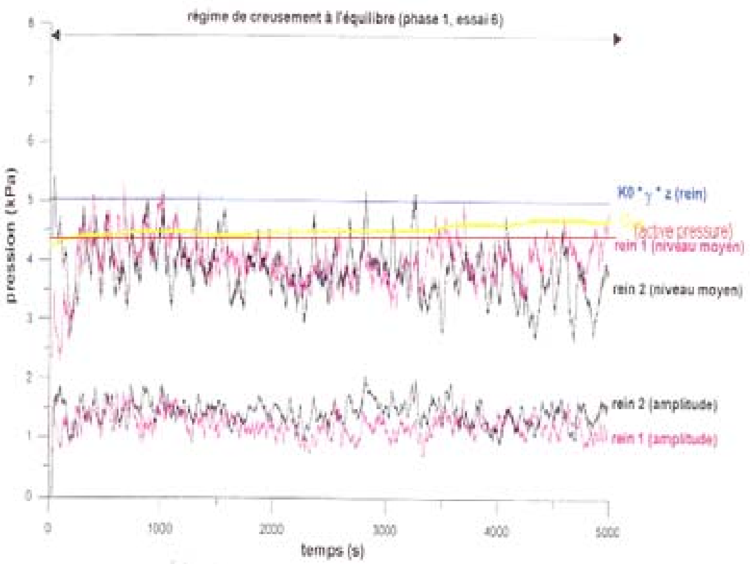


Figure 5.21 Regime of equilibrium (R = 1).

The concept: the Optimum Regime of Advancement

As indicated by the laboratory experiments, the Optimum Regime of Advancement (ORA), even in terms of control of displacements of the ground surface, would involve two conditions: (1) balance between the excavated and removed material and (2) stable pressure condition in the working chamber. When both of these conditions are attained, the pressure released by the face cut by the EPBS should equal the at-rest earth pressure, σ_T. From another point of view, at least theoretically, the choice of the face pressure should not be the primary design problem, but be the goal to be reached through the imposition of the above-mentioned conditions.

Nevertheless, in practice, the use of these conditions can be problematic mainly due to the difficulty in verifying the weight-equilibrium condition (R = 1), given that:

- The exact in-situ density of the ground is often unknown, especially in complex geotechnical environments, and only an approximate value can be assigned. As a consequence, the definition of the weight to be excavated would not respect the required precision.
- The muck is frequently conditioned with additives (foams, polymers, bentonite, etc.) for improving its granulometry and workability. Therefore, the weight and the induced effects of these additives must be considered (see, for example, Herrenknecht *et al*., 1995). However, it should be noted that this conditioning is frequently necessary to achieve homogeneous conditions of the muck in the plenum, allowing the correct transmission of the support pressure to the tunnel face (see also Section 5.2.2.4).

In any case, as a logical consequence of the laboratory results, it seems possible to derive another interesting conclusion: if a face support pressure, different from the at-rest earth pressure, is applied to the tunnel face, it will not be possible to reach and maintain the ORA condition. In other words, the driver of the EPBS cannot keep a stable condition of equilibrium for the advancement and will be forced to make continuous adjustment of the key parameters (i.e. the advance rate and the speed of the screw conveyor).

An example of achieving the ORA is provided by the construction of the new lines of the Porto Metro using EPBS. In this project, the working ranges of the main excavation parameters were fixed in the design and continuously controlled during the excavation process (Guglielmetti *et al*., 2002).

Table 5.16 Main features of the examined section, Porto Metro, EPBS

Geology	Complex conditions: prevalent, completely weathered granite (W5) and/or residual soil (W6), with local presence of boulders of relatively less weathered granite (W3/W4).
Geotechnical condition	γ' = 10–12 kN/m^3; c' = 0–20 kPa; φ' = 30–34° k, k_0 = 0.5 (assumed); $K = 10^{-5} - 10^{-7}$ m/s
Geometrical conditions	H = 18.2 m; h_0 = 14.8 m; D = 8.7 m; $h_w = h_0 - D$ = 6.1m

γ' = submerged density; c' = effective cohesion; φ' = effective friction angle; k_0 = at-rest coefficient; K = coefficient of permeability; H = overburden; h_0 = water head above the tunnel floor; D = diameter of excavation.

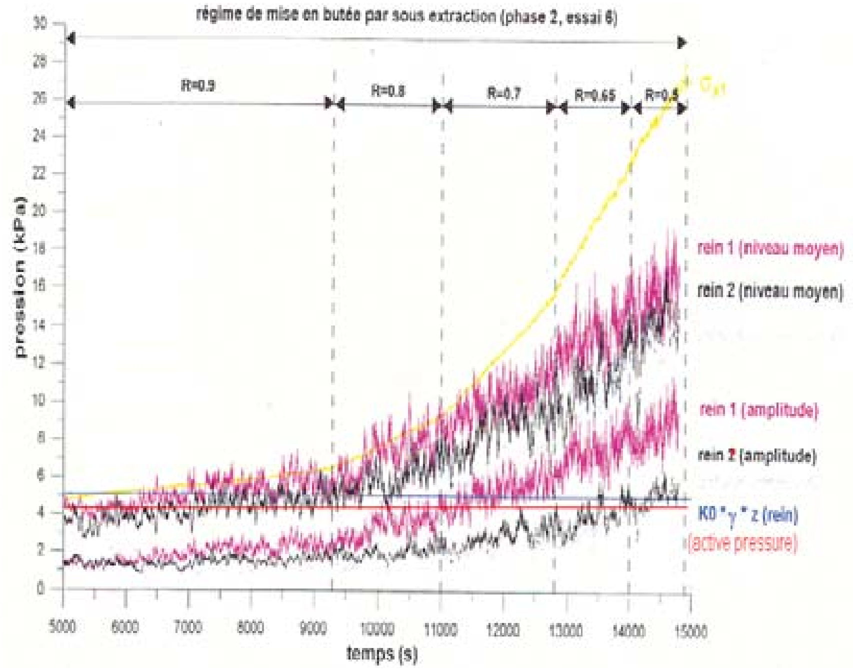


Figure 5.22 Regime of under-extraction (R < 1).

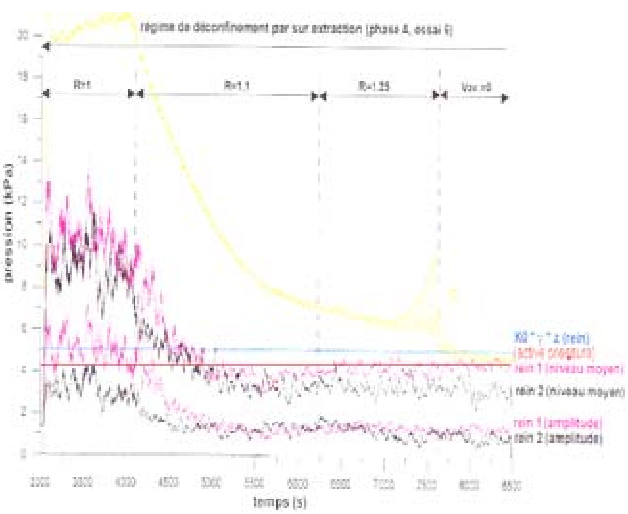


Figure 5.23 Regime of over-extraction (R > 1).

Figure 5.24 refers to one section of the tunnel associated with the geotechnical and geometrical conditions reported in Table 5.16.

The following are some explanatory notes on Figure 5.24:

- The sensor "P7" on the bulkhead, which is one of the 7 sensors (P1 → P7), is located at about 1m below the tunnel crown.
- The water pressure to balance the seepage forces is $\sigma_W = 61$ kPa if measured at tunnel crown ($h_w = h_0 - D = 6.1$ m).
- Under the condition of hydrogeological equilibrium, and considering the worst geotechnical scenario, the requirement of a stabilizing "effective pressure" $\sigma'_T = 21$ kPa was derived by the method of global wedge analysis (Anagnostou-Kovari method). It is important to note that this condition requires the density of the muck in the plenum to have a "minimum" acceptable value (in this case $y_{muck,min} = 14$ kN/m^3).
- The total pressure should be $\sigma_T = 61 + 21 = 82$ kPa.
- In consideration of safety and to account for the pressure fluctuations in the plenum, a final average value of total pressure $\sigma_{T\,(P7)} = 120$ kPa was selected as the basic reference, with a range of 100–160 kPa as the lower and upper alarm limits, respectively. It is noted that applying a total pressure of 120 kPa in this case is nearly equivalent to the assumption of a FS = 2 regarding the shear parameters. Given the density of the muck, this implies a pressure of about 230 kPa at tunnel invert.

The pressure recorded on the bulkhead (P7) and the difference between the theoretical and the actual flow of material through the screw conveyor are shown in Figure 5.24. The data were collected every 10 seconds during the excavation for the examined ring (1.4-m long) and its positioning (final part of the graphs, "standstill"). The following observations are possible:

- The face-support pressure (P7) is not constant, but follows a "sinusoidal" type of curve.
- This behaviour seems to be mainly determined by the Machine operator, by means of a continuous adjustment of the screw conveyor speed, in order to achieve

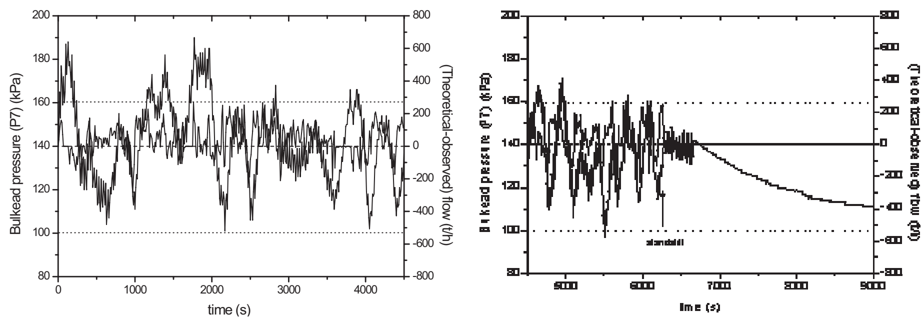


Figure 5.24 The variation with time of the bulkhead (P7) pressure and the difference between the theoretical and the actual material discharge rate through the screw conveyor.

the objective of maintaining both the pressure within the design limits and the balance of excavated and extracted volumes in equilibrium.

- However, due to safety reasons, the operator must operate the screw conveyor at the slowest possible speed, so that a general tendency of under-extraction could be observed, noting that under-extraction could force the pressure to increase.
- Nevertheless, when the operator tried to limit the excessive growth of pressure, by increasing the flow of material through the screw conveyor (and then moving towards a regime of equilibrium, $R \to 1$), a stable condition could not be reached anyway and the pressure quickly decreased.
- Finally, the reduction of pressure beyond the lower design limit is avoided by decreasing the screw conveyor speed: thereafter, a similar pressure fluctuation would begin.
- The "natural" tendency of the pressure to reduce was displayed also during the standstill of the EPBS.
- Both the minimum pressures and the tendency of the pressure in the plenum to decrease during standstill may confirm that the required equilibrium pressure at P7 tended to approach the lower design limit.

It is reasonable to state that the design value of face-support pressure is likely to be higher than the theoretically required value (or "true" value) and, as a consequence, it is difficult for the operator to attain the "ORA condition" ($R = 1$ & stable pressure in the plenum). Furthermore, if the "true" value should approximately correspond to the at-rest pressure, this could imply a very low k_0 coefficient and/or "true" geotechnical parameters higher than the design values (which include the factor of safety).

The conclusion is that only through a cross control of several parameters (the face support pressure, the balance of extracted and excavated material, the apparent density, and other parameters described in Section 6) can the correct and safe TBM operation be managed.

5.2.2.4 Choice of the face-support pressure

For tunnelling using a closed-face machine, the application of a face-support pressure $\sigma_T = \sigma_{k0}$ is often considered to be optimum, from the viewpoint of minimizing face deformation and keeping the face stable (Kanayasu *et al.*, 1995, Reda, 1994).

As explained in the previous section, this condition should be naturally achieved by controlling at least the two basic parameters: face-support pressure and rate of extracted material. However, it is generally difficult to determine *a priori* the coefficient of at-rest earth pressure.

From a theoretical point of view, it is evident that lowering σ_T, with respect to σ_{k0},increases the level of acceptable risk of surface settlements, which could become relevant only if plastic deformations are allowed (i.e. $\sigma_T \leq \sigma_{ka}$). As previously observed, this condition is in fact occurring when R >1 (over-excavation) and the risk increases in the presence of significant and sustained over-excavation.

Furthermore, it is frequently stated in the literature (for example, Reda, 1994) that the stability of the excavation can be adequately controlled if the face-support pressure lies between the active and the at-rest ground pressure (i.e. $\sigma_{ka} < \sigma_T < \sigma_{k0}$). As pointed out previously, the earth pressure becomes active or passive when the ground deforms plastically towards the cutterhead or in the opposite direction (i.e. the ground is pushed by the EPBS), respectively (Fig. 5.25).

Kanayasu *et al.* (1995), collaborators of a survey on Japanese Shield Tunnelling, pointed out that in most cases, the active earth pressure is used as the lowest permissible level of face pressure, but there is currently no clear principle for defining the design value of face-support pressure.

More or less on the same line, as reported by Broere (2001, see Eq. 5.3), the Dutch Centre Onderground Bowen (COB) recommends a value that is a little higher than the active pressure:

$$\sigma_T = k_a \cdot \sigma'_v + \sigma_W + 20 \text{ kPa}, \tag{5.3}$$

where σ'_v = effective vertical pressure.

Taking into account the risk of "blow-out" of the ground, as a practical rule of thumb, it is frequently suggested that the upper pressure limit measured at the tunnel crown should be the total vertical pressure, i.e. $\sigma_{T(\max)} < \sigma_v$.

But as observed later, other upper limits should also be considered.

Examples of face pressure adopted for the EPBS in Japan are summarized in Figure 5.25 and Table 5.17.

The available information indicates that in the European practice, the hydrostatic pressure (σ_W) is generally assured as a minimum value of σ_T for tunnels (at least at shallow depth) in an urbanized environment and a supplementary component for the ground thrust is added (see, for example, Leblais *et al.*, 1996, Guglielmetti *et al.*, 2002). Some relevant cases in which the calculation of σ_T was based on LEM analysis are summarized below. The tunnels were excavated using either SS or EPBS.

- *St. Petersburg Metro (SS)*: A special case of deep tunnels in urban environment is the construction of two single-track tunnels as rehabilitation for Line 1 of the St. Petersburg

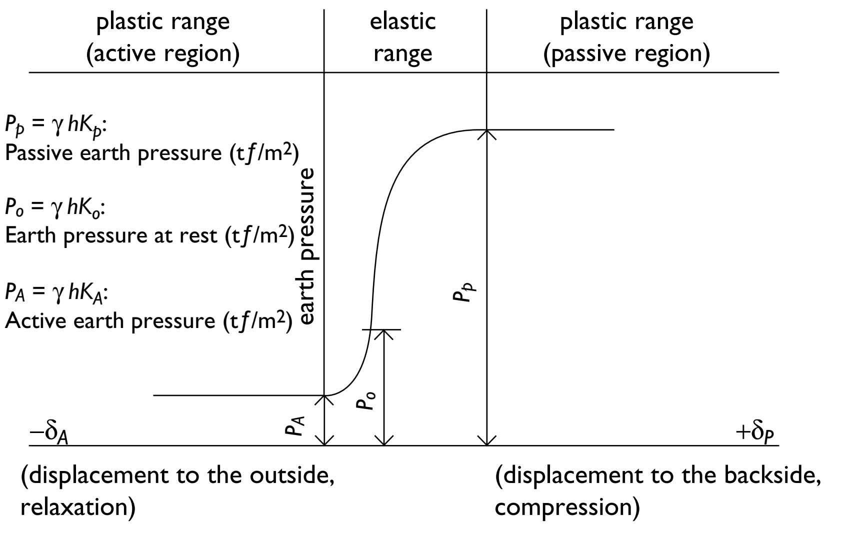


Figure 5.25 Relationship between earth pressure and displacement (Reda, 1994).

Table 5.17 Examples of face pressure used in Japan for EPB and Slurry Shield according to Kanayasu *et al.* (1995), as reported also by Broere (2001)

Machine type	*D (m)**	*Soil type*	*Support pressure used*
EPBS	7.45	soft silt	earth pressure at rest
	8.21	sandy soil, cohesive soil	earth pressure at rest + water pressure + 20 kPa
	5.54	fine sand	earth pressure at rest + water pressure + fluctuating pressure
	4.93	sandy soil, cohesive soil	earth pressure at rest + 30–50 kPa
	2.48	gravel, bedrock, cohesive soil	earth pressure at rest + water pressure
	7.78	gravel, cohesive soil	active earth pressure + water pressure
	7.35	soft silt	earth pressure at rest + 10 kPa
	5.86	soft cohesive soil	earth pressure at rest + 20 kPa
SS	6.63	Gravel	water pressure + 10–20 kPa
	7.04	cohesive soil	earth pressure at rest
	6.84	soft cohesive soil, diluvial sandy soil	active earth pressure + water pressure + fluctuating pressure (~20 kPa)
	7.45	sandy soil, cohesive soil, gravel	water pressure + 30 kPa
	10	sandy soil, cohesive soil, gravel	water pressure + 40–80 kPa
	7.45	sandy soil	loose earth pressure + water pressure + fluctuating pressure
	10.58	sandy soil, cohesive soil	active earth pressure + water pressure + fluctuating pressure (20 kPa)
	7.25	sandy soil, gravel, soft cohesive soil	water pressure + 30 kPa

D = diameter of the cutterhead.

metro, between the station of Kirovsko and Vyborgskaya (see details in Section 8.2). The tunnels were excavated by a 7.4 m Slurry Shield at a depth of about 65 m and under a maximum water pressure of 4.8 bars. The excavation was adequately controlled by a confinement of $\sigma_T = \sigma_W + 1$ bar at the face. The second component (i.e. 1 bar, in this case) covered the effective ground thrust calculated by Anagnostou-Kovari method, as well as the pressure fluctuations (measured values in the order of 0.25 bar). It may be useful to note that, in a case like this example, the effective pressure applied is likely to be in the range of 55 to 65% of σ'_{ka} (see Section 8.2).

- *Metro of Porto (EPBS)*: For the already mentioned section of the Metro of Porto, the basic pressure reference was calculated as $\sigma_T = \sigma_W + 0.6$ bar; the second term in the equation is the effective ground thrust calculated by Anagnostou-Kovari method, taking into account the worst geotechnical conditions ($\sigma'_T = 0.2$ bar plus an additional safety margin). The resulting effective pressure is practically equivalent to assuming a FS of 2 in the Anagnostou-Kovari calculations (see Section 8.3).
- *Metro of Turin (EPBS)*: For the excavation of an 8 m diameter tunnel located above the water table, the designed pressure $\sigma_T = \sigma + 0.3$ bar (where σ is the

pressure calculated following Anagnostou-Kovari assuming FS = 2) was intended for covering the possible fluctuations in the plenum (see Section 8.4).

- *New underground railway connection in Bologna (EPBS)*: A satisfactory control of the settlements in dry conditions was achieved by applying the face pressure σ_T having a value that corresponded to the value obtained by the method of Caquot/Carranza with a FS = 2 (see Section 8.6).

The above cases indicate that, when the definition of σ_T is based on the results of LEM analysis, the assumption of an adequate factor of safety, FS, becomes the main issue.

5.2.2.5 Factor of safety

Different approaches for incorporating a Factor of Safety, FS, in the design value of face-support pressure are used in the analytical methods, either based on the shear strength parameters or directly on the calculated pressures. The basic elements of these approaches are summarized below:

- The FS is applied to reduce the level of mobilization of the shear strength parameters, according to typical formulations in the form $\tau = c/FS + \sigma \cdot \tan\varphi/FS$. For instance, this approach is implemented in the Anagnostou-Kovari method (No. 10 in Table 5.15)
- With reference to the Strength-Reduction Method, integrated by Carranza-Torres (2004) (No. 12 in Table 5.15) in the Caquot (1965) solution, the FS is defined as the ratio between the natural Mohr-Coulomb parameters of the soil and critical values that would create the limit equilibrium condition, i.e. $FS = c/c_{cr} = \tan\varphi/\tan\varphi_{cr}$.
- For the "full-membrane" model, Jancsecz and Steiner (1994) (No. 9 in Table 5.15) suggest the direct application of partial factors of safety (η_E and η_W, respectively) corresponding to the calculated effective (E) and water (W) pressure values. Table 5.18 gives some suggested values of the partial factor of safety that are reported in the literature.

It is important to underline here the presence of two different "factors of safety": one "FS", which derives from the first two approaches of Table 5.18, directly referring to the critical situations; and the other called "f.s.", which has been used for defining the design value of a single geotechnical variable (generically called X).

It seems reasonable to assume that all the international references reported in the previous section would include certain factors of safety (f.s.) and, taking also

Table 5.18 Suggested partial factor of safety for full membrane model

Reference	η_E	η_W
Balthaus (1988)	1.1–1.3	1
Jancsecz (1997)*	≥1.5–1.75	1.05
II Heinenord Tunnel Design**	1.5	1.05
Botlek tunnel Design***	1.7	1.05

* A factor of safety η_B = 1.1 against blow-out is also proposed; ** Reported by Broere (2001); *** Reported by Maidle and Cordes (2003).

into account the ORA concept, it would be interesting to update them to the "real" geotechnical values, to avoid being over-conservative.

A notable procedure for defining the design value (i.e. the value including f.s.) of a certain parameter, as suggested by Cherubini and Orr (1999), in substantial agreement with Eurocode 7 (1993), involves the following steps, all of which refer to a normal distribution of the variable X:

- Calculation and/or best estimate of the statistical mean (X_μ) and Variation Coefficient (CV) of the generic parameter. $CV = \delta/\mu$ (δ = standard deviation, μ = mean);
- Definition of a reasonably safe characteristic value (X_k): for example $Xk = X_\mu \times (1 - CV/2)$;
- Definition of the design value (Xp) by applying the factor of safety to the characteristic value, i.e. $Xp = Xk/f.s.$

For example, in Eurocode 7, EC7, the following values of fs are recommended in defining Xp for different properties (Eq. 5.4):

$$c' \rightarrow 1.6;\ \tan\varphi' \rightarrow 1.25;\ c_u \rightarrow 1.4. \tag{5.4}$$

However, this general approach may be too conservative if references to σ_{ka} or σ_{k0} are made for calculating the stabilization pressure σ_T. Furthermore, depending on the type of the analysis, it seems reasonable to establish a specific reference value, using a correspondence scheme such as the one provided in Table 5.19 and in Figure 5.26.

5.2.2.6 Application of the probabilistic approach

A preferable alternative to the deterministic calculation of stability condition is the use of a probabilistic approach that allows incorporating the entire range of the geotechnical and geometrical variables in the stability calculation. A well-recognized approach is the Monte Carlo method, which is used for modelling phenomena with significant uncertainty in input, such as the factor of safety, FS.

In this way, it is possible to evaluate the probability of occurrence of a certain examined condition. For example, it is possible to analyze the distribution of the factor of safety (FS) for each face pressure and manage its probability to be lower than a target value. As an alternative, one could fix the acceptable FS and visualize, for different values of the face pressure, the corresponding probability of being lower than this target.

An example of the alternative of fixing the acceptable FS, is presented in Figure 5.27, referring to the railway tunnel of Bologna (see Section 8.6) crossing clayey sands. The Caquot's solution was applied in this example for two overburden conditions (h = 20 & 30 m), considering as input, that the shear strength parameters follow triangular distributions limited by the values reported in the figure.

Figure 5.27 shows the relationship between the face-support pressure and the probability of obtaining a FS < 1.75 for h = 30 m, and FS < 2 for h = 30 and 20 m, respectively. In all three cases, the friction angle is in the range of 32 to 38°.

The dotted lines in this figure show that, in the case of h = 30 m, the application of a support pressure of σ_T = 1.1 bar would lead to a high probability (about 59%) of having a FS < 2, while with a support pressure of σ_T = 1.35 bar the probability of having a FS < 2 would be reduced to below the level of 5%.

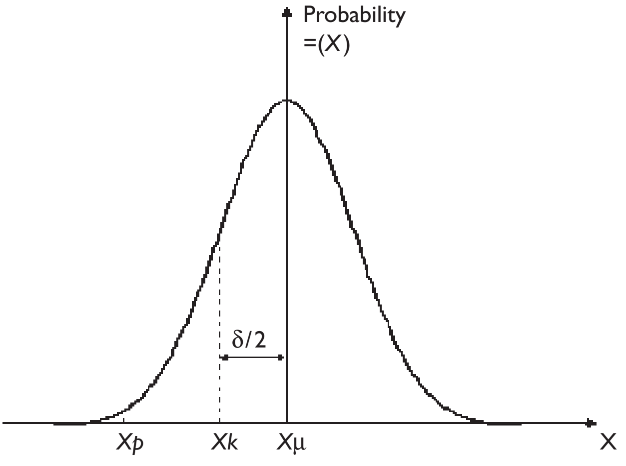


Figure 5.26 Correspondence scheme.

Table 5.19 Reference values for calculation of support-pressure

Basic reference for σT calculation	*Geotechnical value for calculation*
σ_{k0}	X_{μ}
σ_{ka}	X_{k}
$\sigma_{LEM(FS)}$	X_{p}

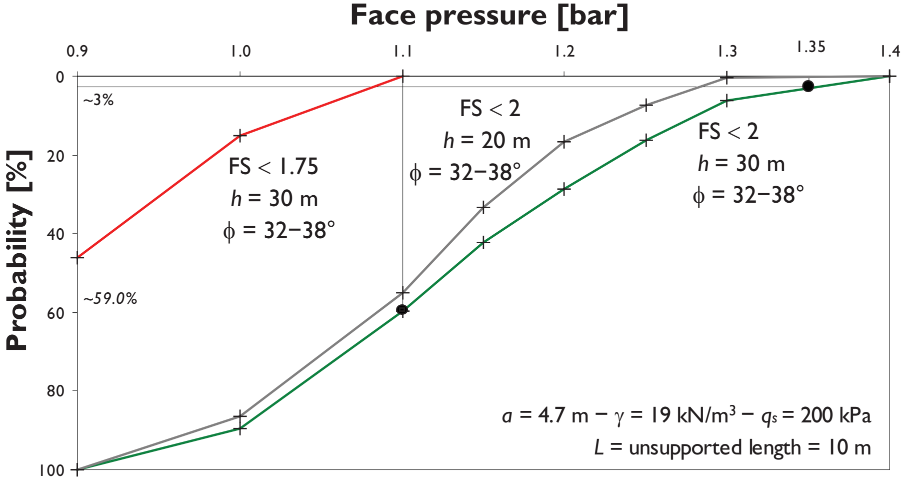


Figure 5.27 Face-support pressure and FS probability.

With a pressure σ_T = 1.10 bar, according to the red line the probability of having FS < 1.75 is practically zero; thus, it is easy to conclude that a value of σ_T = 1.35 bar would surely exclude the probability of having instability (i.e. the condition of FS < 1), which can be considered as a reasonable starting value to conduct further numerical analyses, in order to check the development of plastic zones around the tunnel and the induced settlements on the ground surface.

5.2.2.7 Comparison of the various calculation methods

For visualizing the effect of different values of face-support pressure applied to the tunnel face, the graph reported in Figure 5.28 may also be useful. In particular, the example refers to the case of the Bologna Project (Section 8.6) and presents a comparison of the results of calculation obtained by different analytical methods (in particular, referring to the Caquot's solution with FS = 2) with the relative development of plasticity in the ground, as investigated by numerical analysis with Phase 2 code from Rocscience (Rocscience, 2007). The progressive reduction of the face pressure with the increase in the friction angle (ϕ) value can be observed. Moreover, in Figure 5.28, potential stability conditions of the face are evidenced: they pass from an elastic deformation condition to the complete collapse, for progressive reduction of face pressure. From the static point of view, better conditions can result from applying a face pressure equal to the earth pressure at rest (k_0), assumed as the upper limit of the design face pressure.

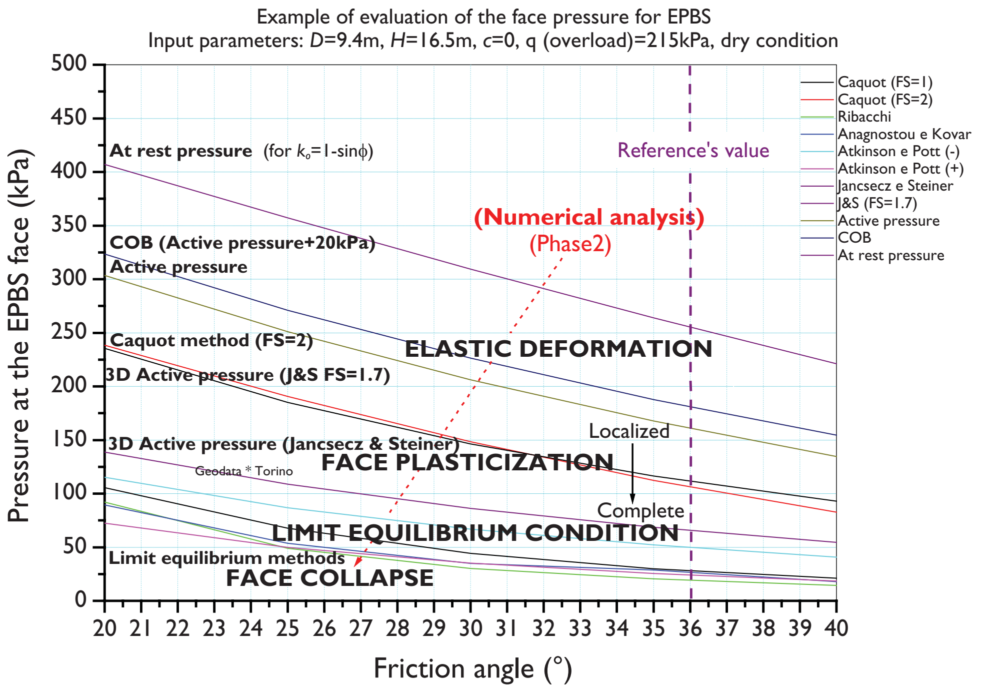


Figure 5.28 Example of evaluation (using numerical analyses) of the face pressure for EPBS and corresponding development of plastic zone (indicative).

As expected, numerical analysis confirms the occurrence of yielding zone at the face for $\sigma_T \leq \sigma_{ka}$. For the examined case, it was also observed that:

- The plastic zones seem to extend above the tunnel-crown level when σ_T is approximately equal to 0.5 σ_{ka}.
- Development of a very large plastic zone corresponds to the values derived by limit equilibrium methods (FS = 1), confirming conditions that indicate a proximity to the face collapse.
- The values calculated by the methods of Caquot (FS = 2) and Jancsecz and Steiner (FS = 1.7) provide a reasonable reference for limiting the extension of plasticization of the tunnel face and controlling the surface settlements. It is interesting to note that these values correspond to about 60–65% of σ_{ka}.

As an indicative example, if the reference values of Table 5.19 would be applied and the specific values of the friction angle would be $\varphi_\mu = 42°$ and $\varphi_\mu = 40°$, the face pressures derived from Figure 5.28 are $\sigma_{k0(\mathrm{mod})} \approx 200$ kPa and $\sigma_{ka(\mathrm{mod})} \approx 130$ kPa, which are not far from the values calculated by the Caquot and Jancsecz methods using a FS = 2.

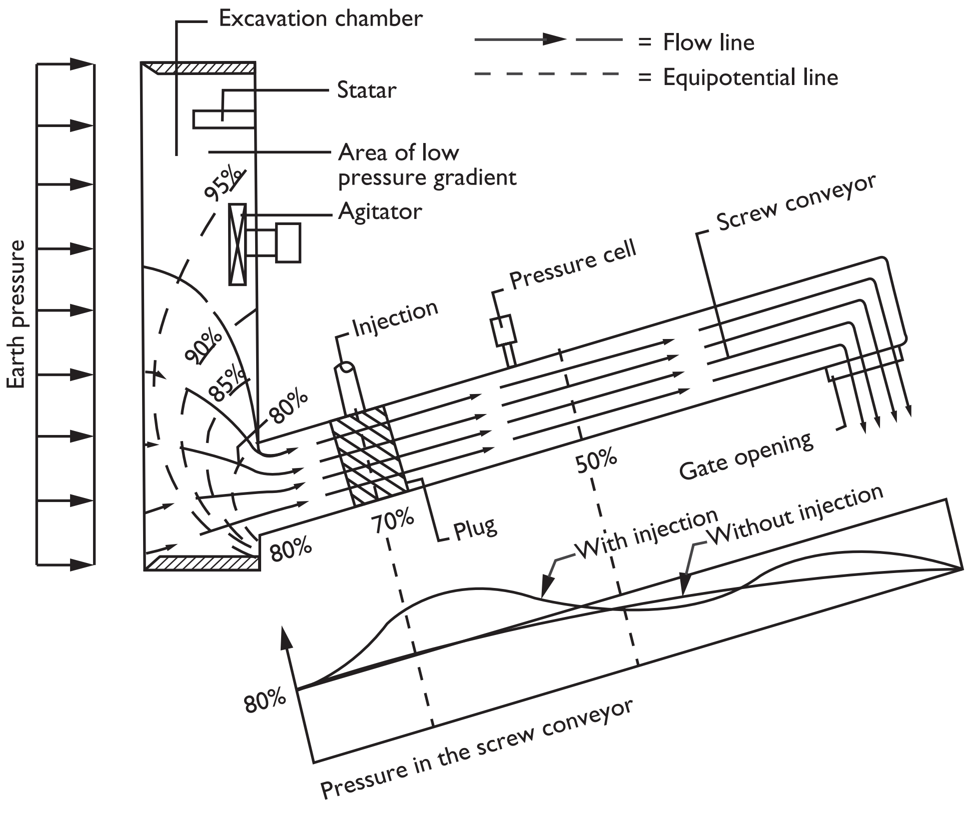


Figure 5.29 Pressure gradients in the working chamber and screw conveyor (Maidl *et al.*, 2003).

5.2.3 Construction phase

5.2.3.1 Construction constraints

The choice of the design value of the face-support pressure should ideally provide the best compromise between safety and the productivity of the tunnel-boring machine. In other words, attention should be given to the rule that the calculated pressure does not constitute practical restrictions or impediments for the advancement of the TBM.

For example, severe operational problems for the advancement of an EPBS could be caused by high values of effective support pressure applied, mainly for the following reasons:

- Uncontrollable distribution of the face support pressure on the tunnel face.
- Excessive torque on the cutterhead drive.
- High cutter-wear.
- Arching effect of the muck at the entrance to the screw conveyor.

None of these operational difficulties should occur when the muck in the plenum acts as a viscous fluid; for this reason Anagnostou-Kovari (1996) point out the necessity to maintain a sufficient water head in the plenum. This can be achieved either by maintaining a high piezometric head at the exit of the screw conveyor or by decreasing the permeability of the muck.

The best means for reducing operational difficulties, and for improving (at the same time) the correct transmission of the confinement pressure, is the proper conditioning of the excavated material, using conditioning agents such as foams and/or polymers. According to Maidl and Cordes (2003), if an EPBS is operated without conditioning, the control of the confinement pressure provides no guarantee for a stable tunnel face. This opinion is based on the fact that the pressure actually present on the tunnel face is unknown and cannot be actively managed. In fact, as indicated also by Reda (1994), and as represented in Figure 5.29, the distribution of the pressure gradients in the plenum, as well as in the screw conveyor can cause significant differences between the applied confinement at the face and the measured pressure on the bulkhead.

Borghi and Mayr (2006) refer to some experience in London, arriving at the same results. Furthermore, they state that the same problems can occur also with foam conditioning, especially in a certain kind of ground, and they suggest the use of only water and polymer in clay, for achieving a more stable pressure control. The authors have also directly experienced in Porto and in Bologna that an excess of foam (due to the presence of gaseous portion in the foam) tend to destabilize the pressure control into the plenum.

In all these cases, it should be better to try to reduce the gas content, controlling Foam Injection Rate (FIR) and Foam Expansion Rate (FER), or even using only water and polymer (see Section 6.3).

For an active control of the face support pressure in the plenum a "Secondary Face Support System" can be introduced (see Babendererde *et al.*, 2004) and Guglielmetti *et al.*, 2002), injecting bentonite slurry into the plenum. A small quantity of slurry is enough to manage the pressure value, provided that the plenum is full of dense material.

Another example of using a pressure-volume controlled foam injection, as a function of the ground conditions and the advance speed, is referred to by Maidl and Cordes (2003). Their experience in dealing with soft clay-sand layers indicates that a limit, concerning the torque and the propulsion thrust, was reached for confinement pressure of around 2.5 bar. Referring to Maidl and Hintz (2003), it is likely that such a limit-condition was determined by the sedimentation of the solid component of the muck in the lower part of the plenum, applying locally very high effective confinement pressure. However, the foams were concentrated in the upper part of the plenum during the advancement and they flowed out during "standstills", allowing the confinement pressure to drop to the level of the hydrostatic pressure, or even to a constant value, equal to the pressure of the gaseous phase of the foam.

The necessity of maintenance of the cutterhead and, in particular, of changing the cutting tools, obliges to enter into the plenum, which must be emptied and air-pressurized before the entry. This introduces another practical constraint: to actually enter a pressurized chamber for long periods in hyperbaric rooms hampering protections required for the workers, and above a certain pressure the requested pressurization and de-pressurization times (regulated by local Health and Safety rules, see forward to Section 7) are too long to allow an acceptable work-cycle time.

All these construction constraints lead to a compromise between the optimum face-support pressure (often too high) and a practical value, related to the residual, acceptable risk level of negative impacts on the surface, defined in Section 5.1.

Detailed procedures for this kind of optimization are shown in Section 6.

5.2.3.2 Monitoring and adjustment

The considerations presented in the previous subsections confirm: (1) the complexity of controlling face stability and surface settlements and (2) the high component of risk (associated with the design choice) arising from both the geotechnical and

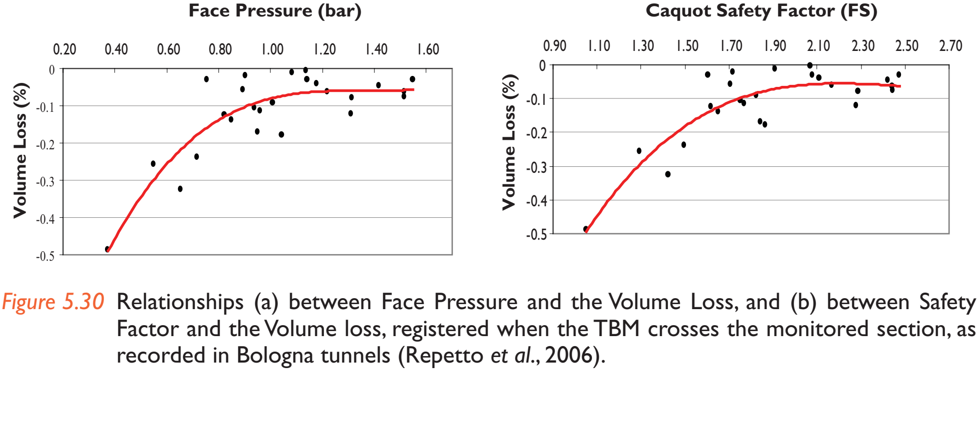


Figure 5.30 Relationships (a) between Face Pressure and the Volume Loss, and (b) between Safety Factor and the Volume loss, registered when the TBM crosses the monitored section, as recorded in Bologna tunnels (Repetto *et al.*, 2006).

construction uncertainties. In such a context, the role of control and monitoring during construction becomes fundamental, requiring a specific risk management process as discussed from theoretical and operational points of view in Sections 2 and 6, respectively.

In general terms, starting from a reasonably conservative initial value of the face-support pressure, an iterative process of controls and verifications should be activated, mainly in the learning phase of the advancement of TBM. This process should identify, in a timely manner, the effects of the face pressure applied and the sensitivity to its variations.

Concurrently, the most stable advance regime should be attempted by controlling the key parameters for the ORA condition.

The experience gained from various tunnels in urban environment underlines the need to check the direct correlation between the applied face pressure and the "pre-subsidence" that occurs up to the moment that the TBM crosses the monitored section. For example, in the case of the Bologna tunnels (see Section 8.6) in more or less clayey sands, it was observed that this subsidence component is about 20–30% of the final total of the subsidence of that section.

The pre-subsidence can eventually be transformed into the volume loss at the tunnel level and used for illustrating the relationships between volume loss and face-support pressure or safety factor (see Fig. 5.30).

Figure 5.30 demonstrates that for a face pressure higher than 1.1 bar, the "initial" volume loss is lower than 0.1%, and the corresponding factor of safety, FS, is greater than 1.9. These results appear to be in good agreement with the indications derived from probabilistic analysis, presented in Figure 5.27. The corresponding, "final" volume loss may amount to 0.3 to 0.5%.

Figure 5.30 also demonstrates that a significant reduction of the face support pressure during the advance of the machine can induce a dramatic volume loss if the characteristics of the ground are poor.

It needs to be noted that the calculation of the volume loss does not take into account the real shape of the subsidence profile. In other words, even though different subsidence profiles could lead to the same volume loss (for example, large and flat vs. narrow and convex profiles), the potential damage to the structures may actually be very different. The relationship between volume loss and settlements is specifically treated in Section 5.1.

5.2.4 Concluding remarks

The topic of calculation of the face-support pressure, σ_T, has been approached from different points of view. The definition of the design value of σ_T is a complex assessment for which different types of analysis should be developed and compared to derive a complete picture of the expected behaviour, both at the tunnel level and on the surface.

The complementary use of analytical and numerical methods should be made in order to extend the detailed results of the numerical analyses to the major number of possible parametric combinations. In this sense, the use of a probabilistic approach is highly recommended to incorporate uncertainty and variability of geotechnical and construction parameters, as well as for relating the design choices to the acceptable probability of occurrence.

The definition of the face-support pressure is not the same issue for SS as for EPBS. In the case of SS, σ_T can be simply assigned in term of total-pressure. In the case of EPBS, a distinction should be made between the fluid pressure and the effective pressure in the plenum.

Depending on the geotechnical property, hydro-geological condition and operational mode of advancement, the most adequate method of analysis should be selected for taking into account all the related phenomena (for example, filtration of the water in the plenum and/or the support medium in the ground, excess pore-pressure, etc.).

For the different types of analyses, it may be reasonable to incorporate different factors of safety while defining the relative input values of the geotechnical variables (for example, as suggested in Table 5.17, these values might, respectively, correspond to $X\mu \rightarrow Xk \rightarrow Xp$).

A reasonably safe value of σ_T should be defined mainly for the learning phase of excavation, and a stable operational condition should be attained by controlling the two key parameters for the Optimum Regime of Advancement (ORA): (1) the balance of the extracted vs. excavated material and (2) the face-support pressure in the plenum. Note that the ORA is governed by the "real" geotechnical properties of the ground (i.e. not affected by the factors of safety).

A careful control of surface settlements must be carried out and, consequently, adequate adjustments of the face-support pressure should be made, as necessary in the construction phase.

As a function of the geometrical conditions, local safety, and construction constraints, the assessment of σ_T may be correctly matched with the different design purposes, which involve different assumptions of risk: a) avoiding any stress release, b) accepting deformations within the occurrence of limit states, and c) controlling the development of yelding zones and related surface settlements up to a fixed acceptable level. Consequently, the resulting value of σ_T should be checked against: (1) the in situ, at-rest pressure, (2) the active limit state of the soil, and (3) the pressure that permits the acceptable level of yelding zones and settlements (for example, the value calculated by LEM with an adequate FS).

These three estimates can also help to determine the possible range of variation of σ_T, with definition of the upper and lower bounds for ordinary advancement. The concept of the operational range of variation of σ_T (the face-support pressure) is one of the key parameters for tunnel construction control, as further discussed in Section 6.

In addition, it has been ascertained that some construction constraints require the limitation of the upper value of face-support pressure. In any case, the absolute constraint $\sigma_T < \sigma_v$, must be respected against the risk of blow-out of the ground.

5.3 THE DESIGN OF PRECAST CONCRETE SEGMENTAL LINING

For tunnel excavation in an urban environment, the choice of a controlled excavation-face pressure shielded TBM, together with a precast lining, is the consolidated procedure that is used throughout the world.

From the point of view of lining, the use of precast segments can offer the following advantages:

- continuous support of the excavation with the shield in order to block the development of surface settlements;
- prevention of water flow into the tunnel by installing a lining which is immediately impermeable;
- ensured longitudinal thrust resistance to the TBM during excavation;
- ensured support for the TBM back-up equipment;
- shortening of the time in which the "finished" tunnel is consigned, from the point of view of civil works and, therefore, ready for the plant engineering preparation.

Finally, the advantages of an environmental nature, and those concerning safety in the work environment, cannot be neglected:

- absence of direct contact between the workers in the tunnel and the excavated ground and groundwater, and
- assembling the support in a single area of the tunnel where there is an intense use of mechanization in a basically clean, tidy, and protected work environment.

This section analyzes modern precast segmental lining types, focusing on the various components and the design process, starting from the alignments and going on to the construction details and best practices for each possible case where this technology can be used in urban environments.

5.3.1 The geometry of segmental lining

A precast lining assembled inside a shielded TBM is a sequence of elements, known as segments, with prescribed dimensions and shapes in order to ensure (see Fig. 5.31):

1. The construction of a stable lining, for both the short and long term, considering all the foreseeable loads.
2. Longitudinal continuity with respect to the tunnel alignment.
3. Rapid and safe assembling in the rear part of the TBM and under the protection of the shield.

5.3.1.1 Types of ring

From a geometrical point of view, the rings are portions of cylinders with surfaces that can be either parallel or non parallel, identified below and in Figure 5.32:

- parallel surfaces ------> straight ring
- non parallel surfaces ------> tapered trapezoidal ring
tapered universal ring

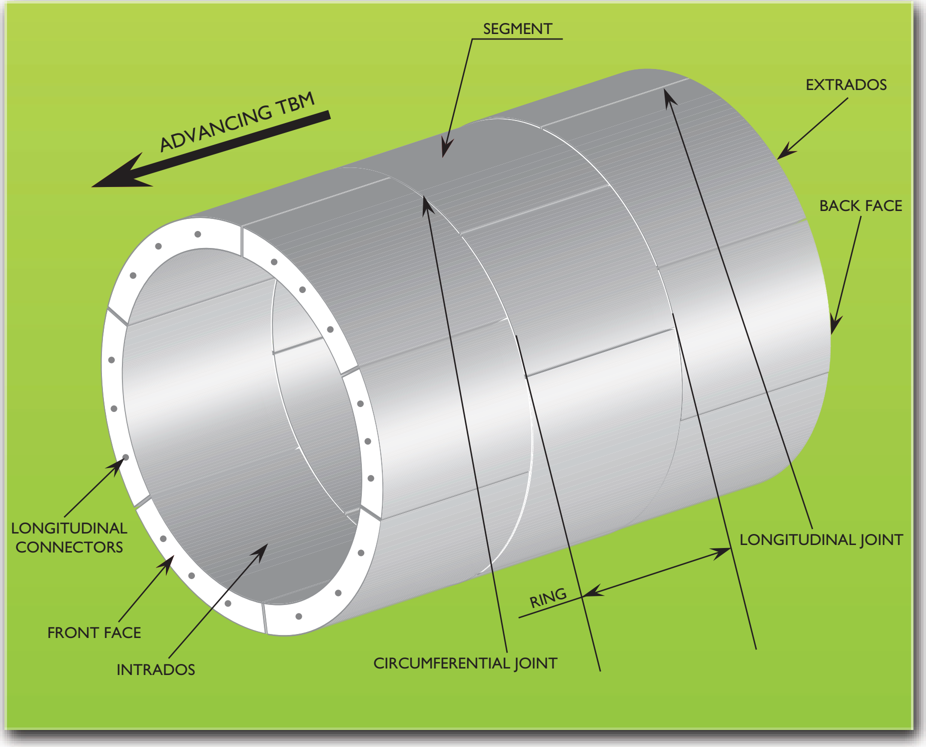


Figure 5.31 Nomenclature of the ring.

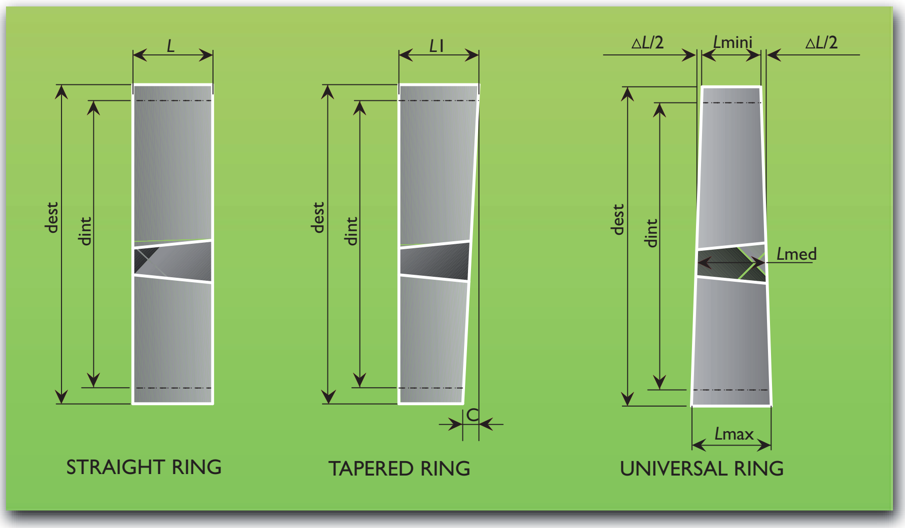


Figure 5.32 Geometry of the various types of rings.

The difference between the two types (straight and tapered) of ring simply refers to the versatility during the assembling stage, but it does not affect the function of the ring.

The straight ring can only be used in straight parts of the alignment: as a sequence, these elements can only be used to make a "tube" with a straight axis. Tapered trapezoidal rings, instead, allow the lining to follow the predefined curves of the horizontal alignment, the profile, and some accidental deviations caused by the TBM.

The use of these two types of ring at the excavation face, implies that the "right" ring must be used in relation to the specific geometric conditions of the alignment or of the TBM. Therefore, it is necessary to have the right type of rings available in the work site.

The current tendency is to use the universal ring systematically in both straight and curved parts of the tunnel. This approach allows the horizontal and vertical trend of the alignment to be followed without the use of any other special elements and to correct any deviations made by the TBM during advancement.

The geometric characteristic that makes a ring universal is its conicity, in other words, the difference between its maximum and minimum length. Figure 5.33 shows the use of a universal ring. Figure 5.34 demonstrates the real possibility to follow a horizontal curve (R plan) and a vertical curve (R alt) at the same time. The radius of curvature of vertical curves is usually one order of magnitude higher than that of the horizontal curves. Therefore, reference can only be made to the horizontal radius. In the case in which the two radii have the same order of magnitude, reference is made to the radius that derives from the composition of both curves for the definition of the geometry of the universal ring.

The universal rings of a particular geometry are known as "left-right" rings. These are truly universal rings from all points of view, but have been conceived in pairs. The geometry of the ring is equal for both, but the arrangement of the segments inside the "left" ring is diametrically opposite to that of the "right" ring, so that an alternation of left-right rings allows a straight line alignment to be followed with the key segment always being at the top.

In order to have a straight line using universal rings, it is necessary to turn each ring by 180° in reference to the previous one, alternatively having the key segment (k-segment – see Fig. 5.36) both on the top and the bottom. Using the right ring and

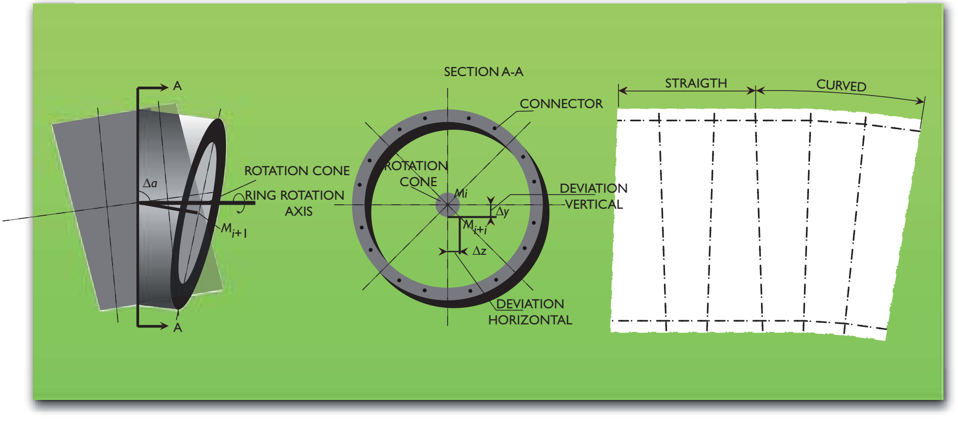


Figure 5.33 Concept of the universal ring and its possible assembly in a curved alignment.

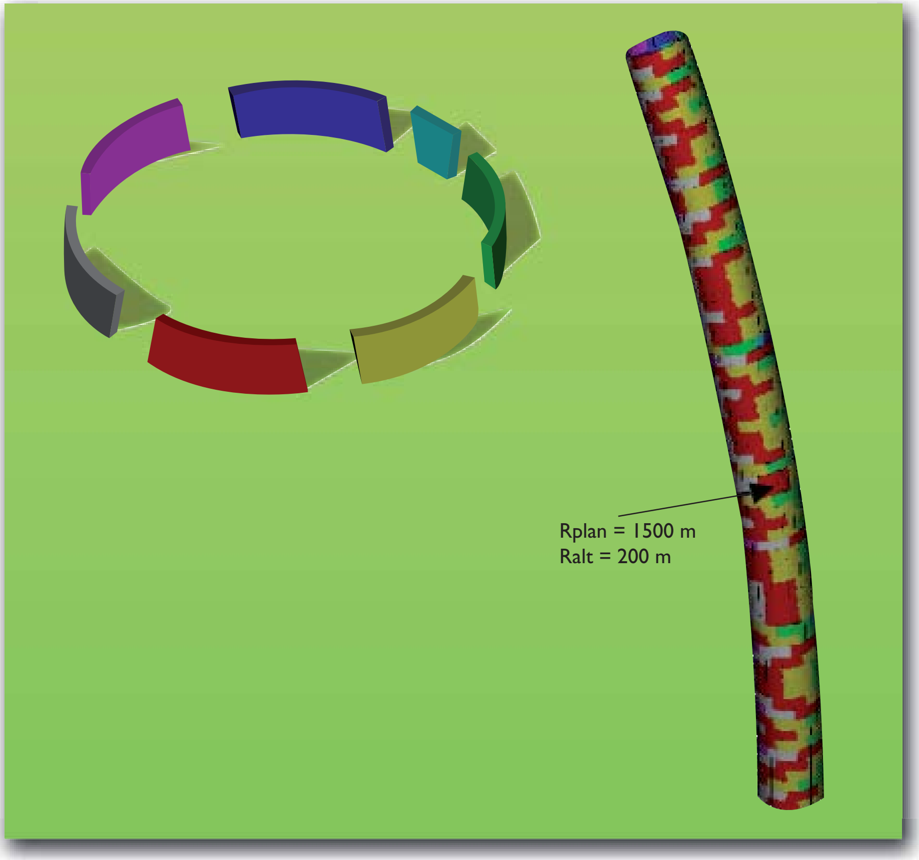


Figure 5.34 Possible assembling in a curved alignment for a the universal ring.

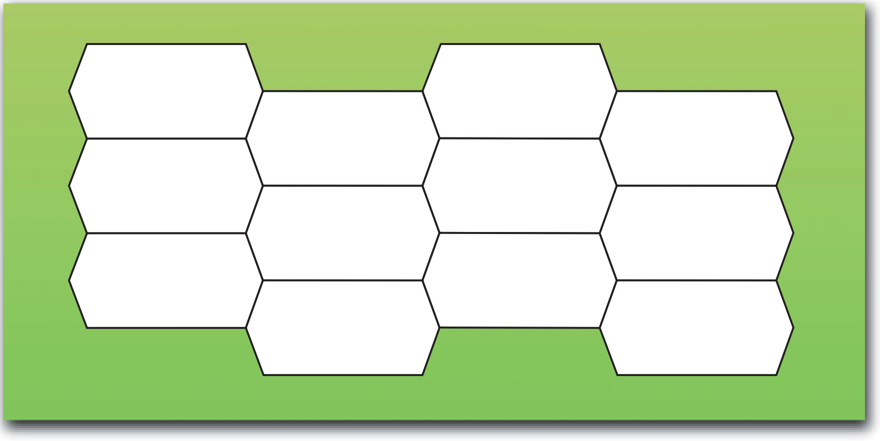


Figure 5.35 Ring with hexagonal elements.

the left ring, it is possible to always have the key segment, or k-segment, on the top and, therefore, to be able to construct the ring from the bottom upwards.

This system makes it necessary to have two types of ring, but it is particularly advantageous when there are only a few curves and the "universality" is mainly dedicated to correcting the inevitable, though small, TBM driving errors. One way of avoiding the use of two rings, without having the k-segment first at the top and then at the bottom, is by placing the k-segment on one side of the ring.

Finally, it is necessary to mention that there is a particular family of precast linings, known as hexagonal elements (or honeycomb elements), which is shown in Figure 5.35. This configuration does not allow a real ring to be made, but it simply creates a repetitive sequence of equal elements that directly form the entire lining. It is often used in particular environments, for example, in long and deep tunnels as well as in hydraulic tunnels, constructed with double shielded TBMs, without control of the pressure at the face. It is rarely used in urban areas as it does not offer sufficient guarantees of a good hydraulic seal; it also does not allow curves to be followed, except those with a very wide radius, which are not very common in these situations.

5.3.1.2 Types of segments

In order to understand how the shape of the segment is chosen, it is necessary to examine the assembly process of the ring inside the tail of the shield. Each ring is assembled inside the tail of the TBM by an erector. The assembly process generally involves the construction of the ring starting from the first segment, and finishing up with the key element, the k-segment, whose presence is always foreseen and is, of course, placed at the opposite side of the ring that has the counter k-segment (see Fig. 5.36).

The k-segment has the shape of a trapezoid with the largest side facing the front of excavation and it is usually smaller than all the other elements. In order to install the key segment, it is necessary to have two counter-key segments with inclined sides to correspond with the shape of the key segment.

For the remaining part, the other elements can have any of the specific geometrical shapes that are illustrated in Figure 5.37. Apart from the aforementioned hexagonal shape, the others are all quadrilateral (rectangle, trapezoid, or rhomboid).

All the previously illustrated rings can therefore be constructed with the shapes that are available. The Figures 5.38 and 5.39 show different ring configurations with differently shaped segments whose choice depends on the segment-connection system inside the rings and on the sequence of these rings, as further explained in the following sections.

5.3.1.3 Geometrical tolerances

Due to the needs of a very accurate coupling among the various segments to compose a lining ring and of placing the rings following a sequence compatible with the alignment to form the complete tunnel lining, the geometrical tolerances of the pre-cast segments is a very important issue and this is especially true for mechanized tunnels lined with universal segments.

The order of magnitude of the relevant tolerances ranges from 0.1 mm to 1 mm, depending on the specific part of each segment considered. Such values are very

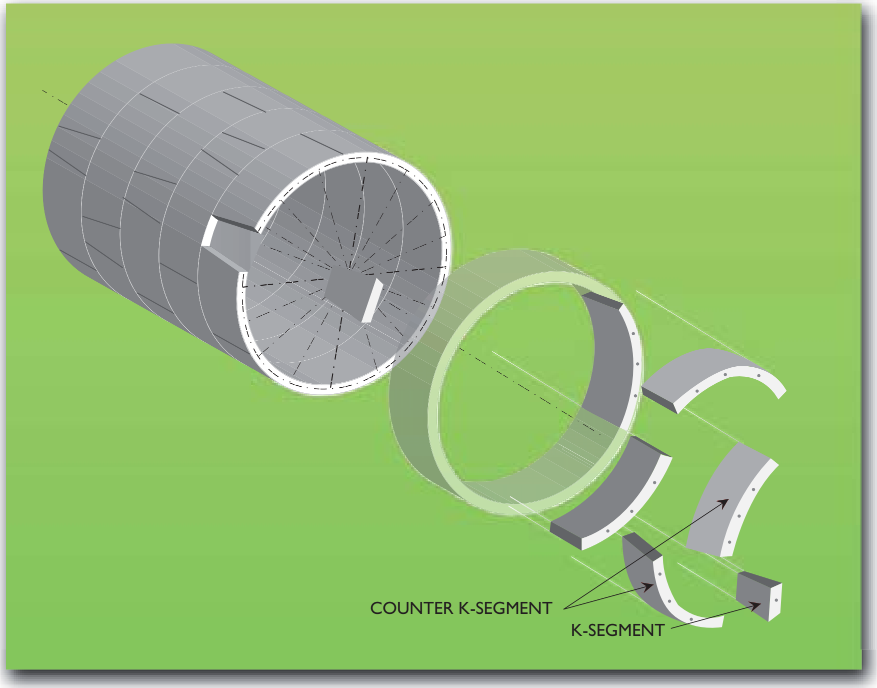


Figure 5.36 Assembling process of the ring.

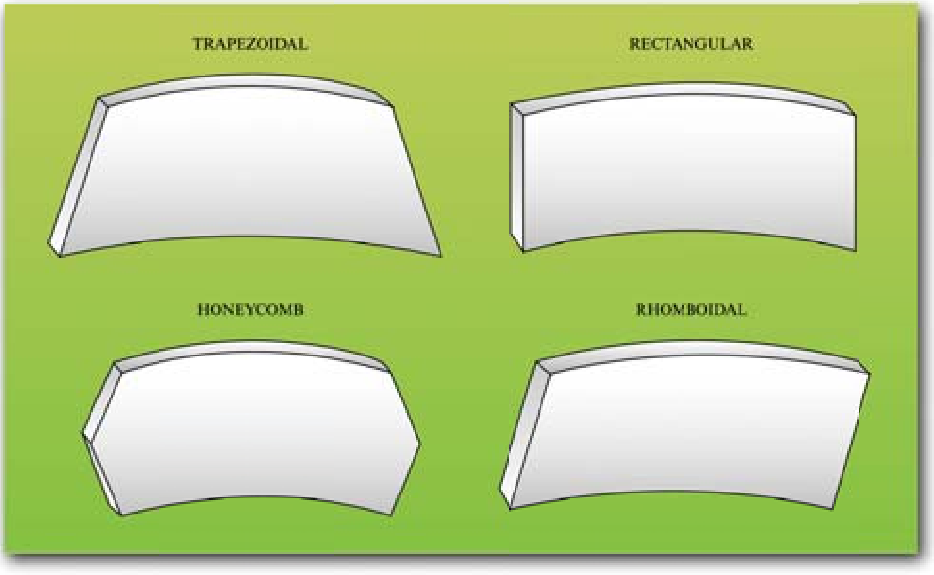


Figure 5.37 Geometry of the segments.

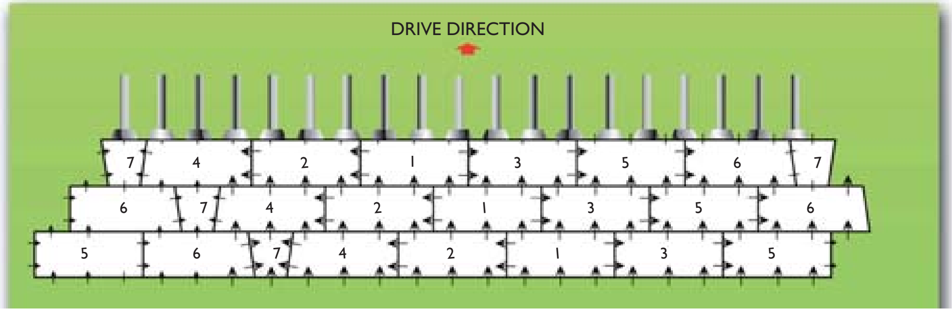


Figure 5.38 Ring configuration with rectangular segments (internal unfolded view).

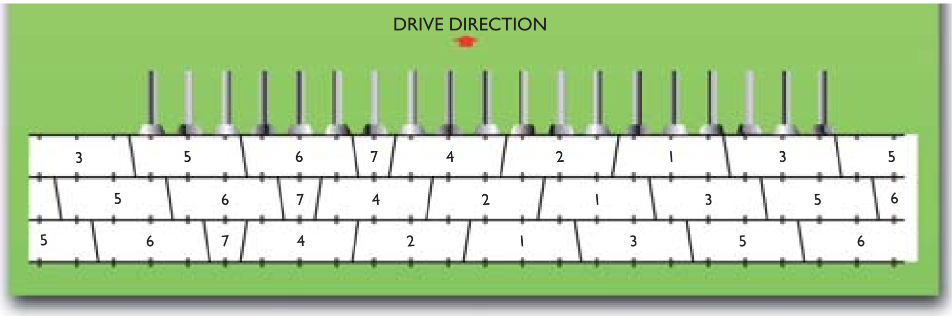


Figure 5.39 Ring configuration with trapezoidal segments (internal unfolded view).

difficult to check directly on a concrete element and thus the measurements are made indirectly on the moulds used to cast the segments. The moulds have to be designed and manufactured with the high precision normally adopted for steel structures, and the inner surfaces of the moulds should be finished by a machine tool. The dimensions of the moulds to be checked before shipping to the construction site are the following:

- Length.
- Width.
- Planarity.
- Angular distance.
- Depth of the various recesses.

The direct geometrical check of an assembled ring is made just at the beginning of the segment production process, at the prefabrication plant where the first few sets of segments from the trial production are assembled to form 2 or 3 rings, laid down on a levelled platform one above the other. In this way, the actual dimensions of the obtained segments and lining rings can be measured at full scale and compared with the respective design values. Furthermore, it is also possible to make use of the same set up to conduct loading tests to check the stress-deformation behaviour of both the segments and the assembled rings.

After the initial full-scale check, the tolerances of the segments are checked, during routine production, periodically and directly on the moulds at a frequency depending on the casting process, i.e. the number of segments cast per day. And the dimensions to be checked are the same as those checked at the site of the mould's manufacturer (see the list above).

Finally, as an example, a typical technical specification for the various tolerances of segment moulds (and thus the resulting segments) is shown in Figure 5.40.

5.3.2 The accessories

The single segment, up close, is much more complex than it first seems; and is composed of different elements, which can be referred to as accessories, and by particular geometries that protect them from failure and make their assemblage easier (Fig. 5.41).

5.3.2.1 The connections between segments and rings

The connections between segments and rings can, at present, be divided into 2 categories:

- Joints with bolts: the segment is first placed in position and then the bolts are inserted and tightened.

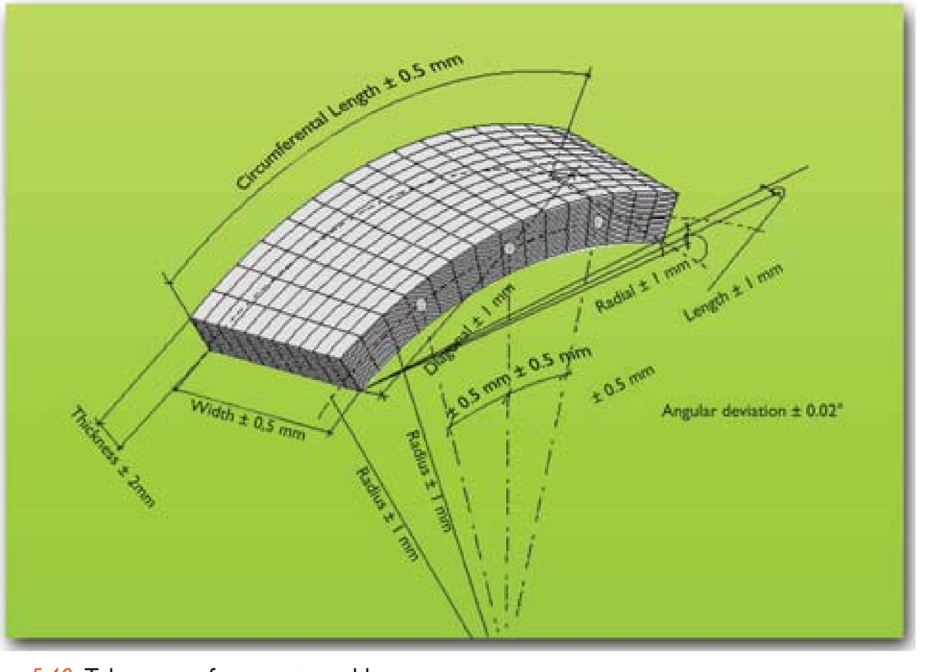


Figure 5.40 Tolerances of segment moulds.

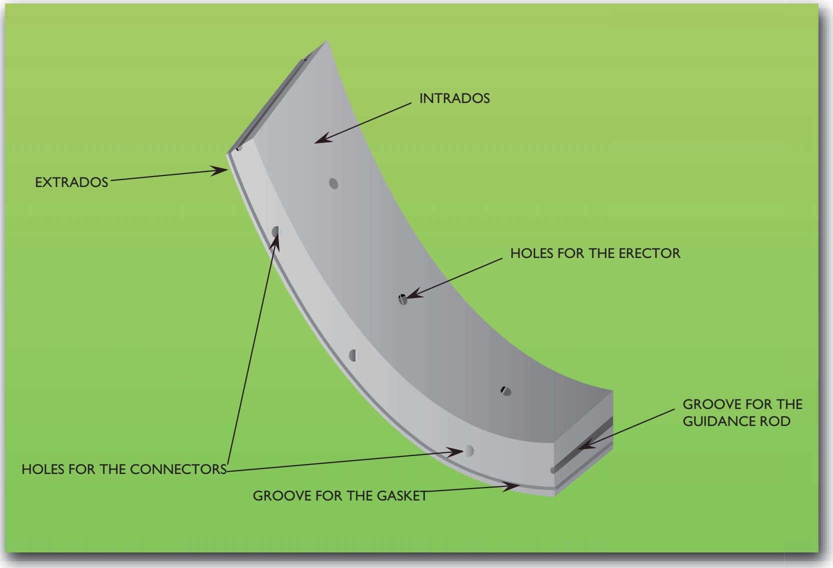


Figure 5.41 3D view of a complete single segment.

- Joints with dowels: the connectors, which are completely covered and hidden, are inserted into the segment during the assemblage and are mortise-inserted (or dove tailed) into the segment of the last assembled ring.

The first category, joints with bolts, requires more effort in the construction of the mould because it is necessary to create "pockets" and "grooves" into which the bolts are inserted. It is also necessary to have more personnel in the tunnel to insert the bolts. This type of connection is traditionally correlated to rectangular segments (see Section 5.3.3) and is generally used both between rings and between segments, within a ring.

The bolts themselves are metallic while the embedded threads, if present, are generally in plastic.

Figure 5.42 shows the typical housing of a straight bolt. Attention should be paid to the following geometrical details:

- The pockets should be large enough to allow the head of the bolt and the pneumatic wrench to be easily inserted; and the minimum distance from the bolt axis to the walls of the pocket must be at least 60 mm.
- The slot side of the pocket should have a conicity of at least 1°.
- The bolt slot in the segment that houses the nut should have a compatible conicity so that the insertion of the bolt into the tunnel will be well guided and fast.
- The bolt axis should pass through the centre of the segment.
- The distance between the end part of the nut and the extrados of the segment

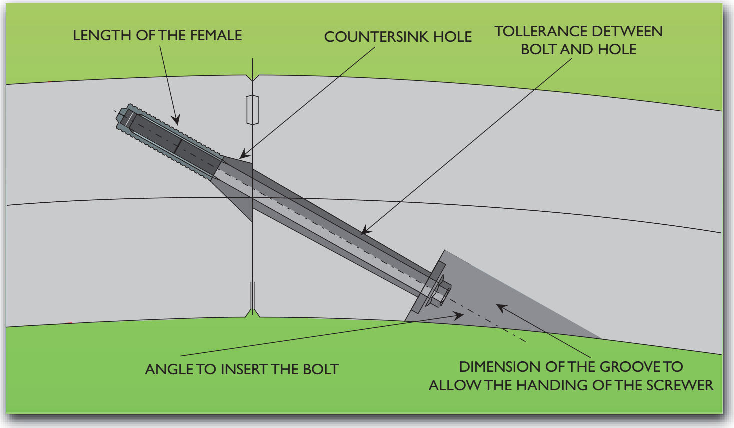


Figure 5.42 Section of a typical housing for a single bolt.

should be sufficient to not interfere with the lining bars and, therefore, as a minimum, it should be 1.5–2.5 cm more than the cover side.

Bolt joints with curved elements also exist, but these are less common. However, the details concerning the geometry of the straight elements are basically also valid in this case.

Joints with dowels require less work for the construction of the mould and less manpower in the tunnel as the insertion is automatically performed by the erector when the segment is positioned.

The dowels and nuts, when present, are made of plastic and sometimes have the core in steel. Figure 5.43 shows the typical housing of a pin, which is placed on the axis with the middle point of the segment, for the variety with a nut and without a nut, in which the pin is directly forced into a hole cut out of the concrete.

Because of the kinematics of the assemblage, this type of connection only intervenes between the rings, while a guidance rod is used between segments of the same ring (see Fig. 5.44); it allows the segment to be guided into its position during the assembly stage and it functions as a shear pin.

The connection with dowels can be used only for rhomboidal and/or trapezoidal segments to avoid early crawling of the gaskets during the segments-approach phase of the ring assembly.

5.3.2.2 *The segment-erection system*

The method of picking up segments by the erector systems can be divided into two main categories:

- “vacuum” types, picking through suction, and
- mechanical types.

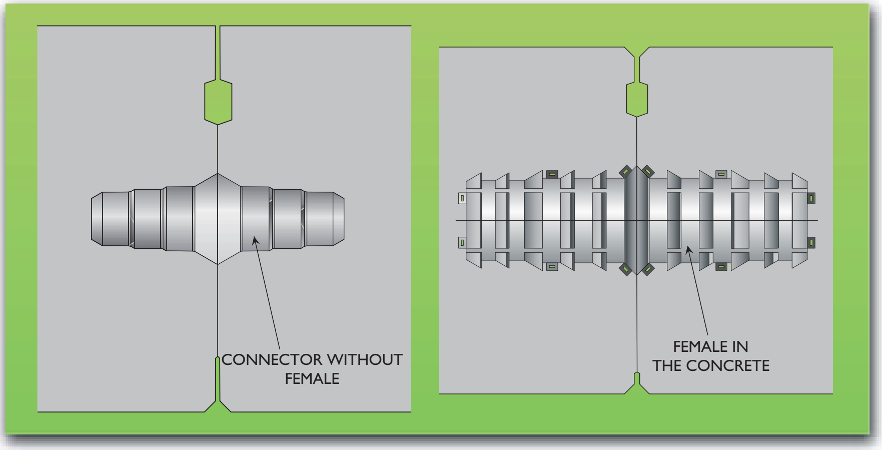


Figure 5.43 Section of a typical room for a single plastic connector (dowel).

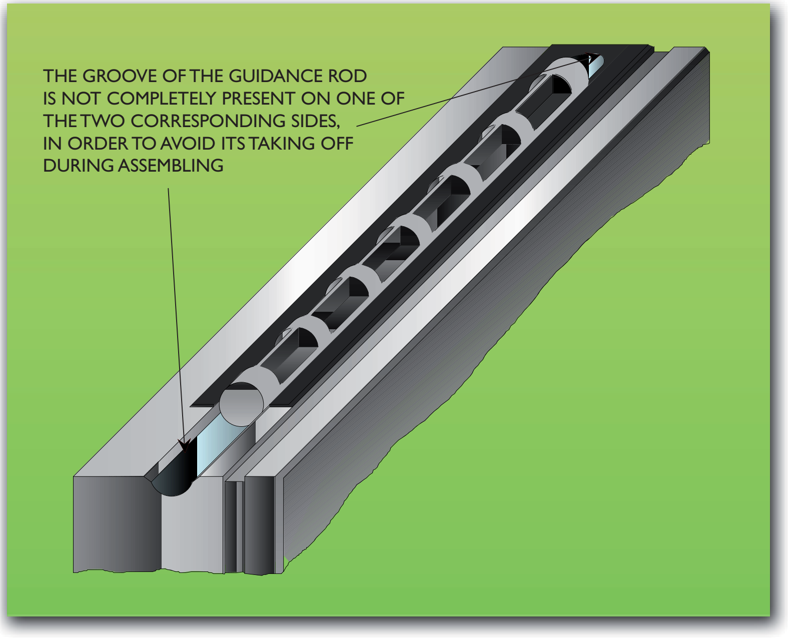


Figure 5.44 Geometry of the guidance rod.

The vacuum system functions through a large suction cup, divided into two or three sections for safety reasons, which, leaning on the intrados, grips the segment creating a void between its plate and the concrete surface. Two centering conic devices on which the system is positioned are used for greater safety (see Fig. 5.45).

The mechanical system involves a rapid screw thread in the centre of the segment into which a large screw is inserted. This screw has a spherical head that is large enough to be hitched by a tool. Four elements that press on the intrados of the segment to prevent the segment from rotating around the taking point complete the erector (see Fig 5.46).

5.3.2.3 The waterproofing system

In general terms, waterproofing of the ring is guaranteed by the following factors, all of which are equally important:

- an overall optimal quality of the concrete and of the segment, resulting from the high level strength of the concrete that is used together with an accurate prefabrication process;
- provision of care when moving the individual segments to avoid the formation of cracks, even latent ones;

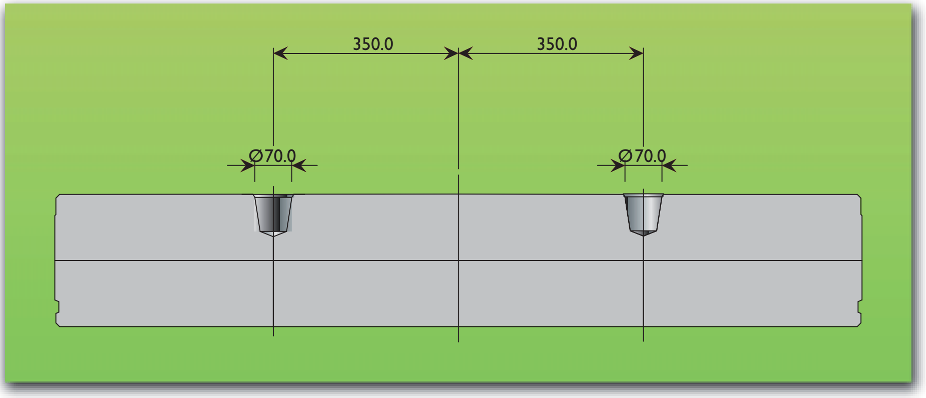


Figure 5.45 Conical insert for the vacuum system.

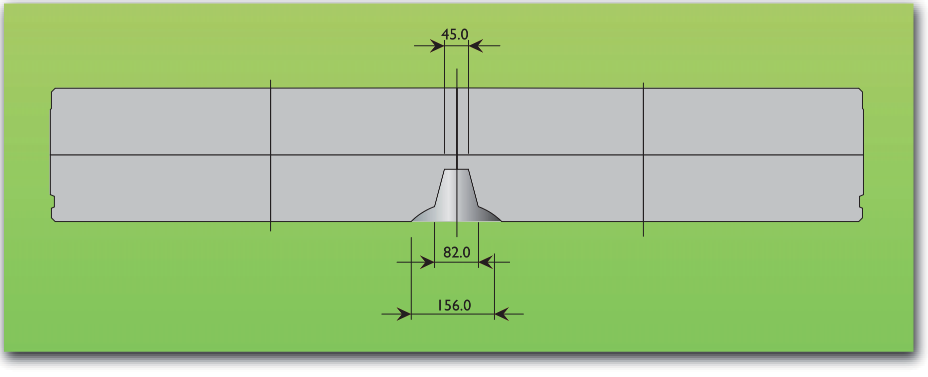


Figure 5.46 Mechanical connection of the erector.

- choice and positioning of the seal;
- accurate assembly of the ring, aligning the segments, and avoiding any possible damage, and
- filling of the ring void with suitable material.

The waterproofing system of the joints through the use of seals (or gaskets) is examined in this section.

The sealing elements always work in pairs as they are positioned in special grooves placed on each single side of all the segments close to the extrados and, therefore, they come into contact when the segments are assembled to form a ring.

There are basically two types of seal:

- Compression seals: they are compressed, one against the other, in the short term, by the connectors (for both segment and rings) and, in the long term, by stresses acting in the ring.
- Compression and swelling seals: the basic principle is the same as described for the compression seals with the addition that a part of the seal, which physically swells in the presence of water, develops a very high-pressure sealing capacity. This type of seal is more delicate as it must be protected until it is assembled to prevent undesired swelling which would make it unusable.

All of the seals that can be used have similar geometries and are, therefore, only different in terms of their "width", "height", and hardness of the used "rubber" (EPDM). The seals can also have an added layer of material on the surface that reduces friction in case the seals slide during assemblage.

Figure 5.47 shows a range of seals with their characteristic shapes and dimensions. The choice of the seal depends on the following factors:

- the purpose of the tunnel;
- the expected life of the tunnel;

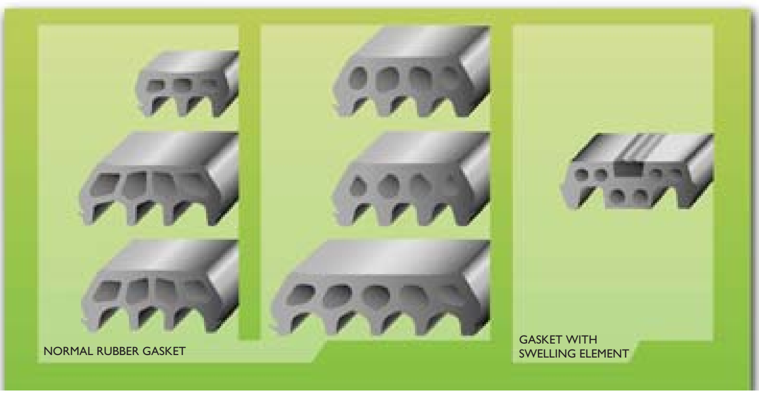


Figure 5.47 Typical shape of different gasket (or seal) types.

- the maximum and minimum pressure that the seal will be subjected to;
- the direction of the pressure (i.e. from the outside towards the inside or vice versa);
- the lining construction tolerance, and
- chemical actions due to water, the atmosphere, and the longitudinal injections.

In order to make a practical choice, it is necessary to clearly understand how such elements work and to be able to interpret the various diagrams that the producers supply for each profile in their catalogues.

The functioning of a gasket during its various work stages can be summarized as follows (see Fig. 5.48):

1. The pair of seals is moved together during assembly.
2. The two seals are pressed against each other and they deform. The squeezing, during the assembly and pressure stages due to the advancement of the TBM, is completed, i.e. the concrete segment surfaces are placed in contact, due to the predominant force of the thrusting jacks.
3. The pressure from the jack is released and the seals tend to open due to elastic reaction. This movement is opposed by the presence of either the bolt or pin connectors.
4. An equilibrium is reached which, if the seal has been well dimensioned, is compatible with the hydraulic pressure it must resist.

The diagrams supplied by the producers (see Figs. 5.49–5.50) illustrate the connections between:

- the gap and the reaction force of the pair, and
- the gap with the various, imposed off-set and seal pressure values.

The gap is defined as the squeezing value of the pair of seals while the off-set is the shifting of the pair of seals. The greater the gap and off-set, the lower the sealing capacity (or pressure) of the pair of seals.

A big off-set and/or gap can be caused by a defective assembly of the ring. In fact, the seal conditions dictate the acceptable limits that should be used as a reference in tunnels to express a critical judgement on the correct assemblage of segments and rings.

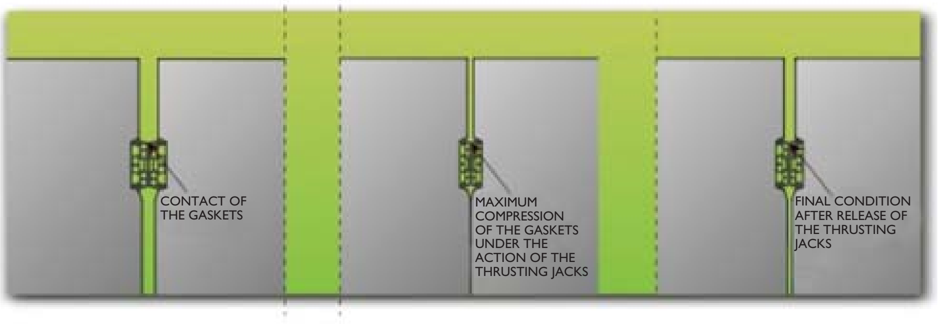


Figure 5.48 Working sequence of the gasket.

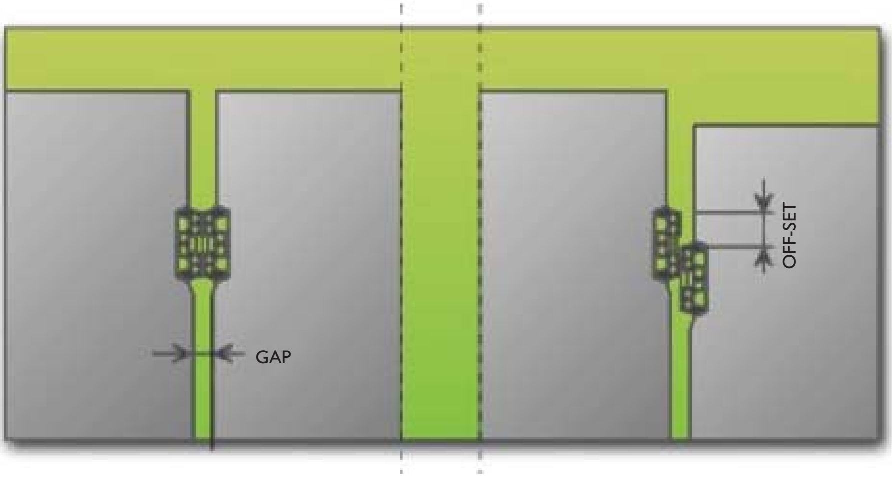


Figure 5.49 Geometrical definition of gap and off-set.

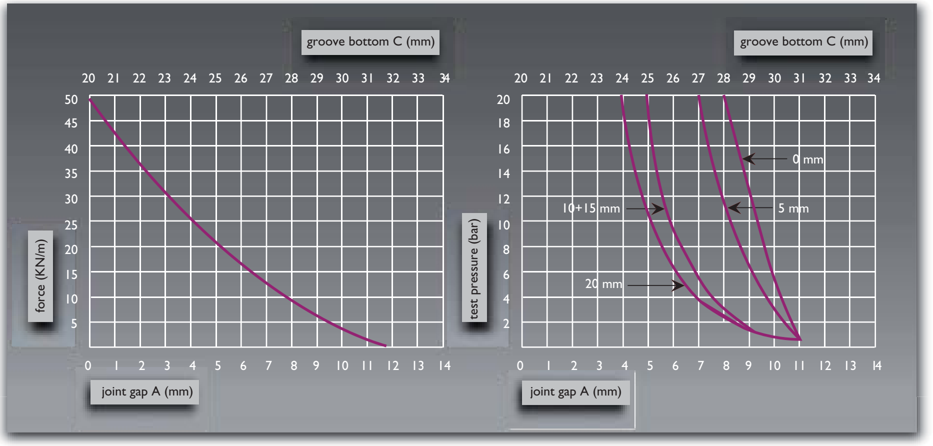


Figure 5.50 Typical diagrams gap-force and gap-working pressure with different value of the off-set for a gasket.

The diagrams that describe the functioning of a pair of gaskets are furnished by the suppliers, and are produced in the laboratory using specific geometries for the grooves that have to house the seals. The geometry of the test groove (see Fig. 5.51) should be furnished by the suppliers and should be rigorously reproduced on the segment because only in this way is it possible to ensure that the foreseen pair of gaskets will work, conforming to the curve used for its choice.

Finally, the choice should be made on the basis that the examined pair of gaskets will guarantee the design pressure seal, in the presence of the maximum gap and offset values, and with adequate safety margins.

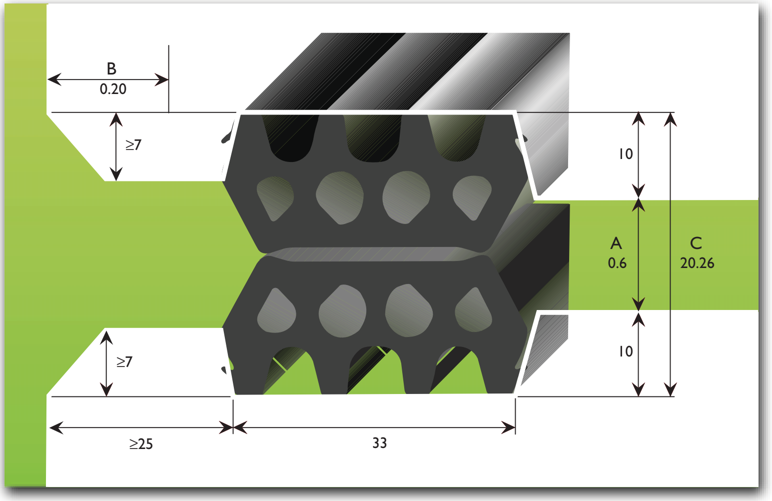


Figure 5.51 Example of specific geometry of the gasket's groove.

5.3.2.4 The thrust-pressure distribution elements

Load distributing pads between subsequent rings can be positioned in correspondence to the sides of the segments so that they are aligned with the thrust jack (see Fig. 5.52). These elements, made up of reinforced bituminous material, of a thickness of 1.0–1.5 mm, ensure that the contact between the jacks and the segment and between the segments and the subsequent ring occurs in a well-defined zone, from which the reinforcement lining descends on the edges.

It is also possible to arrange concrete protuberances on these sides (of the segments), which are inserted into the recesses in the sides of the segments of the already installed rings and which contribute to facilitating the alignment during the assembling stage.

5.3.2.5 Geometry of the segment corners

The corners of the segments are extremely delicate points which can be broken, sometimes seriously. This results in not only an aesthetic defect, but also often in the loss of function of the element, usually the waterproofing of the joints. For this reason, the corners and the edges should always be a little indented compared to the theoretical line. Figure 5.53 shows the various tapering shapes of the corners that are usually performed in the different directions.

5.3.2.6 Codification of the segments

Different symbols that help the workers during the assembling stage can be placed at the segment intrados and therefore on the internal surfaces of the moulds. The following elements are usually foreseen:

Figure 5.52 A segment with load distributing pads.

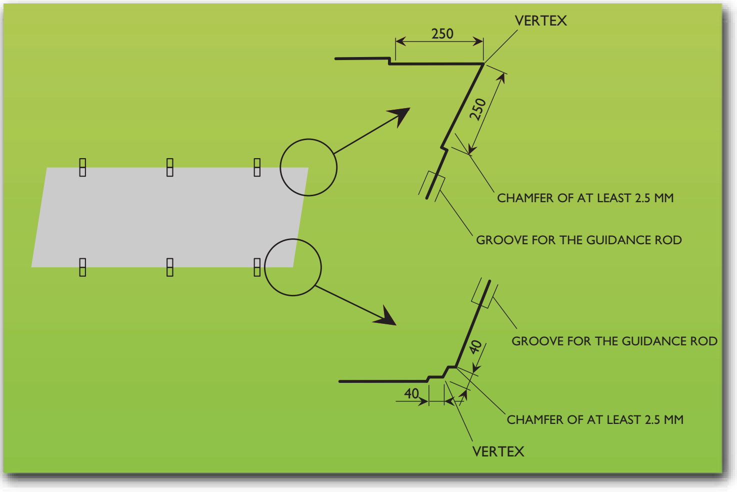


Figure 5.53 Detail of the chamfer at the segment corner.

- name of the segment;
- labels that allow an easy alignment of the segment (i.e. the longitudinal connectors) during assembly, and
- symbols to indicate possible drilling axes that do not interfere with the steel cage.

5.3.3 Ring assembly

In a broad sense, the assembly process begins with the supply of the segments at the portal and ends with the exit of the ring from the tail of the TBM. The segments are brought into the tunnel in wagons that move either on wheels or tracks. These wagons carry the segments into the back-up where they are lifted by a system known as "segment feeder" which takes them to the erector positioned inside the shield. The arrival order of the segments at the erector must be programmed to respect the assembling order; therefore, the first to arrive is the first to be assembled. In order to make this process easy, the segments are marked with letters and/or numbers which clearly give the assembling sequence.

The kinematics of the segments during their movement in a tunnel is constrained because of their dimensions in comparison to the maximum free width in the back-up and vice versa. Figure 5.54 clearly shows the constraint that exists between the dimensions of the segment and the space that is available for manoeuvring. The space constriction can put the integrity of the elements at risk during their transport, especially for small/medium machines (with excavation diameters below 7 m). The segment can also be delivered by the erector with the intrados facing downwards (segment above) or facing upwards (segment below).

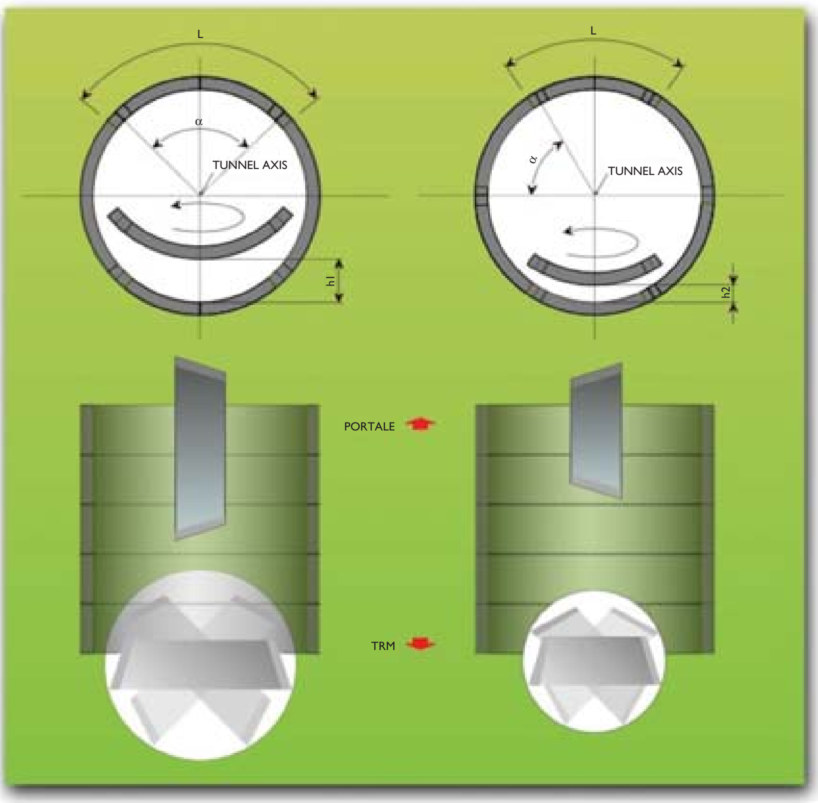


Figure 5.54 Constraints on the movements of the segment inside the back-up.

Figure 5.55 gives a simplified illustration of the scheme for assembling the individual segments to form the ring. In short, assembling of the first segment is followed by the others, one on the right and one on the left until reaching the counter-crown elements, and then the crown element is installed. The minimum (necessary) number of jacks are removed during these operations, to allow the segment to be assembled. The stability of the already positioned segments, before the ring is complete, is provided by the jacks and connectors/bolts. This is a potentially hazardous stage for the safety of the personnel because it is in this particular stage that segments can accidentally fall in the TBM. It can also be observed that the joints between two adjacent segments of a ring are never in the same angular position as the joints between previous or subsequent rings. A lining is like a brick wall in which all the elements are staggered so that there are no longitudinal weakness lines, which can otherwise be identified as hinges which, if present in a great number, would make the structure weak.

A detailed analysis of this operation shows: (1) the relationship that exists between the shape of the segments and their connection-system and (2) some geometrical constraints that exist between the distance covered by the pressure pistons of the machine and the inclination of the oblique sides of the crown required to guarantee a correct and safe insertion.

5.3.3.1 Assembling the normal segment

The joining with dowels (see Section 5.3.2.1 for dowel and bolt connections) foresees that the segment should already have the aligning connectors assembled when it is moved towards the ring, that is, the final movement of the segment is forced to follow the direction of the ring axis, at least for the last stretch of 15–20 cm, which corresponds to the length of the protruding part of the pin. In this case, if rectangular segments were used, the seal would be dragged along the final part and would be

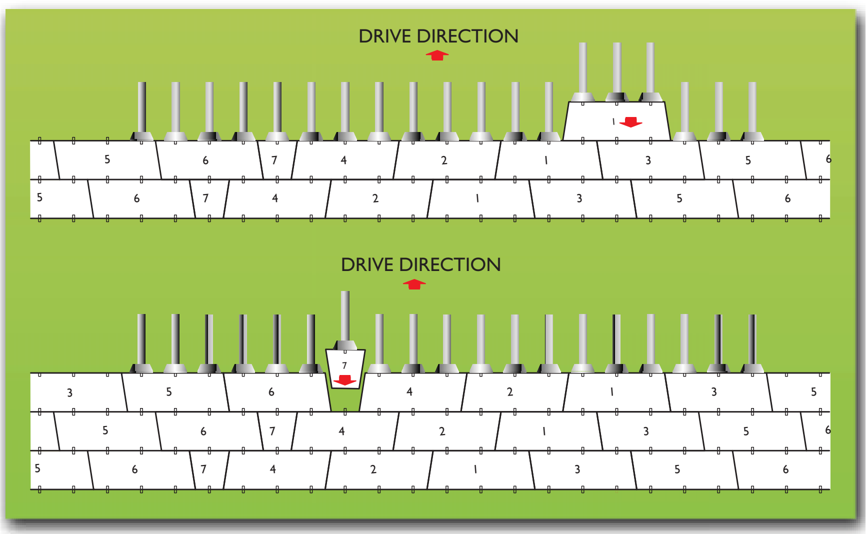


Figure 5.55 Assembling process (numbers indicate the order of assembling of the segments).

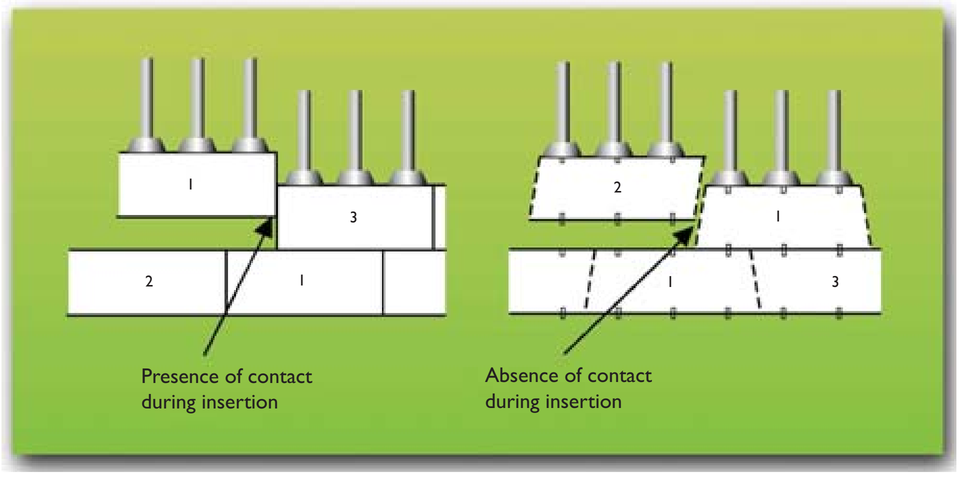


Figure 5.56 Need of the trapezoidal segment in presence of connectors.

destroyed or ripped. Therefore, the trapezoidal segments are used (see Fig. 5.56) as the practical alternative.

If bolts are used for joining, the movement of the segment during assembly can mainly be circumferential and would occur only in the last longitudinal centimetre. Therefore, the rectangular segments are adequate in this case.

Furthermore, pins do not allow connections between segments. On the other hand, it is not very efficient to use pins for connections between rings and to use bolts for joints between segments. The rings joined with pins, therefore, need the segments to be constrained together: this is obtained through the use of a guidance rod, as previously mentioned (in Section 5.3.2.1) and through a geometric effect caused by the oblique sides being further "blocked" due to the curvature effect of the ring.

5.3.3.2 Assembling the k-segment

The k-segment is the last to be inserted and, therefore, the movement that takes it into position has to be of a longitudinal type, that is, parallel to the tunnel axis, after it has been placed in the correct radial position.

Since the geometry of the joints that separate the segments is radially orientated, it is necessary that the thrust jacks have a stroke longer than the maximum segment length of a certain quantity for allowing space for the key assembly.

Figure 5.57 shows the movement of the segment-key that is required to carry out the insertion. The figure also illustrates the potential for the key vertices to collide with the sides of the two counter-key segments. To get around this problem, it is necessary to study both the inclination of the sides and the value of the additional piston stroke: the greater the inclination angle of the oblique joint, the lower the value of the additional stroke. The insertion of the k-segment is obviously a very delicate operation and, therefore, the care and ability of the operator of the erector are of great importance.

5.3.4 Geometrical pre-dimensioning of the ring

The geometrical pre-dimensioning of a ring is the first activity that permits verification of the compatibility of the lining with the chosen TBM (already existing or newly constructed) and it allows the following parameters to be identified:

- the thickness;
- the length and number of segments per ring, and
- conicity (or taper), in the case of universal rings.

The reference criteria can be divided into two groups:

- criteria related to the design and use of the work such as diameter of the tunnel and definition of the alignment, and
- criteria related to the execution of the work such as optimization of the excavation cycles and positioning of the segments, overall dimensions, and weight of the precast elements.

Each of the operative choices taken during the pre-dimensioning stage should then be confirmed in the subsequent, more detailed analysis stages.

5.3.4.1 The thickness

The ring thickness can be initially identified on the basis of criteria of experience and, in particular, using databases taken from the literature. The most efficient, in this sense, are those that have been made available by AFTES in its recommendations which are summarized in Figure 5.58 and integrated with the information available from the projects referred to in Section 8.

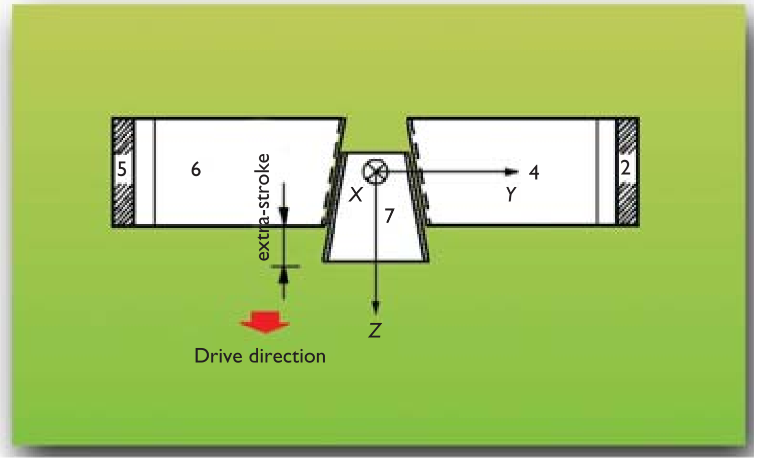


Figure 5.57 Radial movement of the k-segment before insertion.

(a)

(b)

Figure 5.58 Relation between internal diameter and (a) lining thickness, (b) number of segments.

5.3.4.2 The length and number of segments per ring

The choice of the length of a ring and the number of segments depends on several factors:

- longer rings reduce the number of assemblies that have to be carried out, but the single elements are more cumbersome and heavy and also less suitable for alignments with many curves having a small radius; the incidence of the accessories per metre of tunnel is reduced;
- a greater number of segments per ring reduces their dimensions and weight, even though their assembly time is increased as is the cost of the accessories per tunnel metre, and
- larger segments increase the risk of damage during movement in the assembling stage.

The average length of the rings is between 0.60 m and 2.00 m, even though the typical values in urban environments for road tunnels are usually between 1.2 m and 1.7 m, given the presence of curves with reduced radii.

The number of segments can also be directly connected to the pressure system of the TBM and, in particular, to the number of pressure jacks. A good rule is to avoid the pressure shoes from resting on a joint between segments and assure that the number of longitudinal connections are equal to the number of pressure shoes (in order to have a correspondence between the ring rotations and the number of shoes).

The number of connections can be:

- 2 or 3 for generic segments, and
- 0 or 1 for the crown.

The number of shoes that the TBM can have, all equally distanced, assuming the choice of having a ring with 6 normal segments plus a key, will be:

$$2 \times 6 + 1 = 13;$$

or

$$2 \times 6 = 12;$$

or

$$3 \times 6 + 1 = 19;$$

or

$$3 \times 6 = 18.$$

A key without longitudinal connections should at least have bolt joints for connection to the other segments. The dimensions of such a key can be obtained from the two counter-key segments. Instead, a key with a connection has an extension equal to the spacing between one connection and another.

An abacus could also be furnished for calculating the length and number of segments, as well as for their thickness, making use of the available examples, thus becoming a reference for a first choice. Such an abacus can be obtained using the information from the Case Histories reported in Section 8.

The lining thickness and number of segments as a function of their length are depicted, respectively, in the upper and lower parts of Figure 5.58.

5.3.4.3 The conicity of the universal ring and its rotations

The universal ring is a cylinder with two converging ends at a distance R_a from the cylinder axis; R_a is called the design radius of the universal ring. A sequence of rings positioned without any relative rotation among them makes up a lining that follows a curve of radius R_a. The value of R_a must be lower than the minimum radius of the alignment, R_c, because when the TBM moves along the alignment, it can deviate, even in minimum radius sections; and the ring should be able to help the TBM return to the right trajectory, carrying out corrections with curves that could have values lower than the minimum radius, R_c, of the alignment, even for very short stretches.

It should be added that rings can only rotate by an angle that is equal to a multiple of the angular distance between one longitudinal connection and another, and that not all the physically possible rotations are admitted, because of the continuity of the joints.

Figure 5.59 gives a view of a ring highlighting the positions of the ring in reference to the point, *k*, where the key is located, when the average length is on the horizontal diameter.

Figures 5.60 and 5.61 show a sequence of rings under development that is not allowed and a sequence that is allowed, respectively.

In order to use a correct sequence at the time of assembly, the positioning matrixes should be made available to the TBM driver who should respect them to follow the correct rotation. It is noted that these matrixes allow the alignment to be followed and prevent the formation of hinges on the structure.

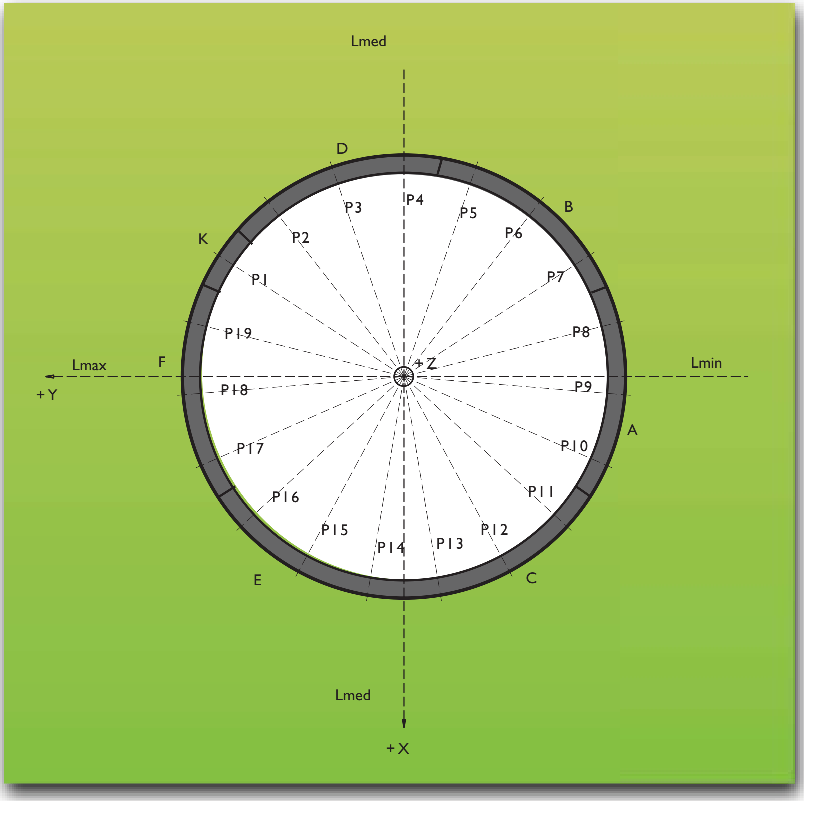


Figure 5.59 Position of the ring with respect to the location of the key segment.

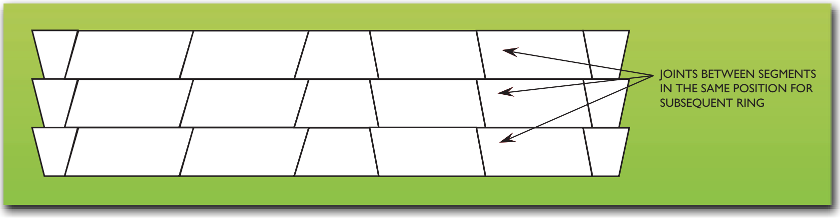


Figure 5.60 Not allowed sequence of rings (unfolded view).

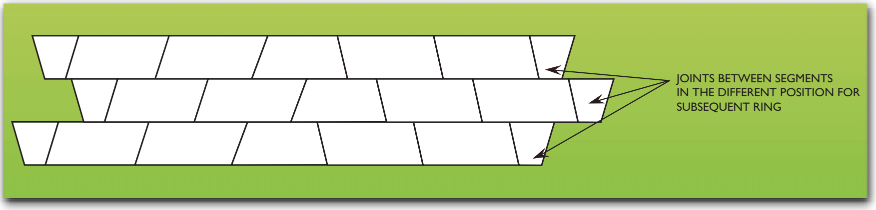


Figure 5.61 Allowed sequence of rings (unfolded view).

The following is an example of the positioning matrix:

Position	P1	P2	P3	P4	P5	P6	P7	...
P1	x	0	x	0	0	x	0	...

x: allowed position; 0: not allowed position

On connector P1 can be positioned connectors P3, P6, but not P2, P4, etc…

Finally, Figure 5.62 indicates how it is possible to obtain the conicity value ΔL, once the mean length, L, of the ring and the design radius are known.

5.3.5 Design criteria and structural verification

Once the geometrical pre-dimensioning of the lining structure has been carried out according to the steps defined in Section 5.3.4, it is necessary to confirm what has been identified by conducting structural dimensioning and by specifying the characteristics of all the elements as well as indicating the advance rate of lining or producing the construction design in relation to the design stage that is being followed.

The verifications are necessary for the ring as a whole and for each of its components, i.e. the segments, in consideration of all the work conditions listed below:

- Single segments:
  - prefabrication;
    - extraction from their moulds;
    - handling;
    - storing;

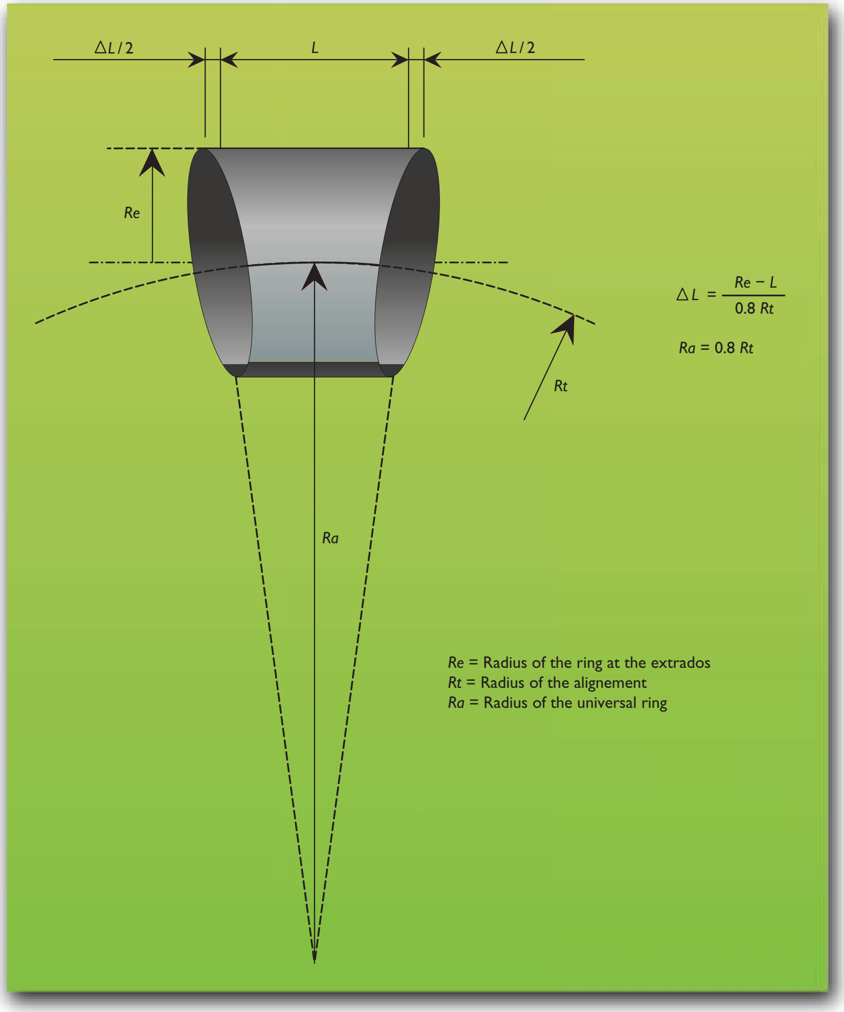


Figure 5.62 Definition of the value for the conicity.

- transport to the tail of the shield;
- assembly of the ring;
- advancement of the TBM.

- Ring as a whole:
  - longitudinal injections;
  - exit of the TBM;
  - long term stability of a;
    - ring;
    - segment;
    - joint.

In order to complete the collection of all the relevant information, it is necessary that the following essential characteristics of the TBM be available:

- Excavation cutter-head:
  - excavation diameter.
- Shield:
  - total length;
  - diameter (behind the cutter-head and in the tail).
- Thrust system:
  - number of jacks;
  - number of thrust shoes;
  - dimensions and area of the thrust shoes;
  - eccentricity of the jacks compared to the mean radius of the ring;
  - maximum distance covered by the pistons (stroke);
  - maximum total thrust force;
  - maximum acting pressure for each shoe divided into normal work conditions; and exceptional conditions.
- Segment lifting system (segment feeder and erector):
  - vacuum (geometry of the lifting area);
  - mechanical (lifting points).
- Injection system:
  - injection points;
  - maximum pressure of the injection.
- Back-up system (referring to the heaviest wagon):
  - total length;
  - number of bearings;
  - load for each bearing.

5.3.5.1 The forces at the prefabrication stage

Classical verifications should be carried out for each element produced in a precasting plant. The forces that act during each stage are summarized here, together with the resulting structural schemes.

Extraction from the moulds

The acting forces that should be considered are:

- self weight,
- load increasing due to humidity deriving from not completely hardened concrete and from steam,
- adherence to the moulds during extraction, and
- increase in the mass forces due to dynamic vibration effects.

The static scheme that can be adopted (see Fig 5.63) is that of a curved beam (with the curve facing downwards) on two bearings, which is clearly a conservative scheme in relation to the instruments that are used (generally erectors with vacuum lifting).

The main verification that should be carried out is a "first crack verification". The result should lead to the definition of the minimum strength value that the concrete should have before it can be removed from the moulds. This type of verification cannot, in general, be a determinant for the amount of steel required for reinforcement.

First storing

After their extraction from the moulds and in relation to the prefabrication plant requirements, it is possible to begin stacking up the segments.

An initial storing stage can thus be identified, for use immediately after the extraction, and before reaching a definitive storage stage.

The acting forces that should be considered are:

- self weight;
- load increasing due to humidity deriving from not completely hardened concrete and from steam, and
- increase in the mass forces due to dynamic vibration effects (a value greater than 50% with respect to that used in the extraction verification is assumed).

The static scheme that can be adopted (see Fig. 5.64) is that of a curved beam resting on two bearings, with the curve facing upwards, loaded by the weight of a segment placed on top.

Again in this case, the main verification that should be carried out is the first-crack verification. The results should confirm that, with a strength value imposed for the removal from the container, no damage occurs. This verification should also specify the geometry of the bearing points in detail.

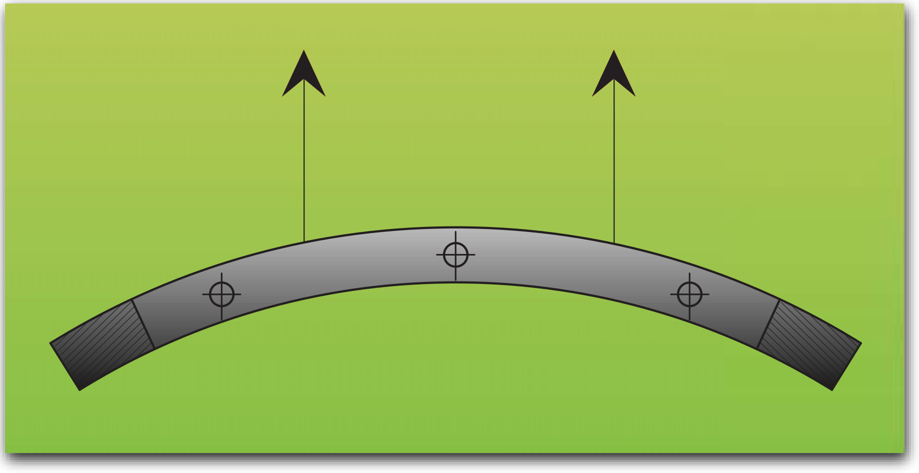


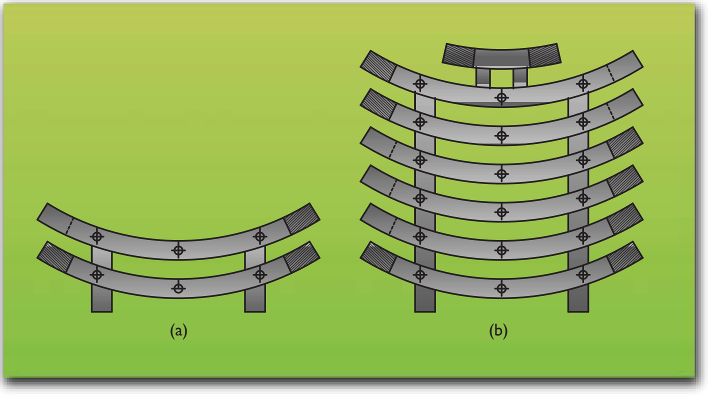
Figure 5.63 Conservative static scheme for extraction from the mould.

Final storing

In this case, the only acting force is the self weight and the static scheme is identical to that of the previous case, with the beam, however, being loaded by the weight of all the segments that make up the ring. This is certainly a limit case which depends on the requirements of space and dimensions of the rings.

The main verifications that should be carried out are the first crack and shear verifications. The results should allow one to establish the strength value that is necessary to proceed with the operations and for avoiding the risk of damage. The indication of the geometry of the bearing points is even more delicate. The loads on the segments should be applied with a certain shift (about 5–10 cm) with respect to the bearing

(a)



(b)

Figure 5.64 Static scheme for (a) the first storing and (b) the final storing.

points, on the outside and on the inside, in order to take into consideration a certain degree of approximation in locating the separators when storing the segments.

5.3.5.2 The forces coming from the TBM

The forces that the TBM exerts at the face to excavate and advance are completely transferred to the sequence of rings. The necessity of guiding the machine leads to the application of different pressures for each group of jacks, with a general tendency to have greater forces on the lower part to counteract the natural tendency of "descending", which is caused by the dominant weight of the cutter-head.

Another determinant factor which should not be underestimated, is that the jack system could act in an eccentric manner with respect to the average radius of the segments. This is not a rare event, because it is a consequence of the construction of the TBM itself or of a progressive "shifting" between the TBM axis and the tunnel axis.

Considering that the applied pressure has the most significant effects on the result of the structural dimensioning and the appearance of damage on the segments, its verification should be carried out as rigorously as possible. The typical work condition that is subject to verification should, therefore, use the maximum pressure value, applied with the maximum possible eccentricity value.

In particular, the following verifications should be performed:

- contact pressures;
- tensile stresses induced in a radial direction, and
- tensile stresses induced in a circumferential direction.

There are reference models, which are well-known in structural engineering, and which can be used to verify the anchorage of the pre-compression cables in the head of the beams (in precast and prestressed concrete). The calculation can be conducted using analytical formulas based on the theory of elasticity, or through 2D or 3D computer models. The application of these models for comparison purposes, in particular the analytical formulas (see Leonhardt, 1975) and the 2D models, has indicated that the use of the former is sufficient for a correct and normal planning of the elements.

Verification of the contact pressures

The verifications can be done according to the process provided in Eurocode 2, point 6.7, in reference to puntual loads. The verification is satisfactory when the following inequality is valid (Eq. 5.4):

$$\text{Fsd} \le \text{Frdu} \le \text{Fmax} \tag{5.4}$$

with

$$F_{rdu} = A_{c0} \cdot f_{cd} \cdot \sqrt{\frac{A_{c1}}{A_{c0}}} \tag{5.5}$$

(reference can be made to Figure 5.65 for the definition of the symbols)

- Frdu is the maximum load at the Ultimate Limit State of the thrust jacks force
- Fsd is the maximum ULS load of the force exerted by the pistons
- Fmax is the maximum acceptable load which is directly related to the quality of the concrete, regardless of the geometry of the loading area
- $A_{c0} = b_0 * l_0$
- $A_{c1} = b_1 * l_1$
- b_0 and l_0 are the dimensions of the loading area
  b_1 and l_1 are the dimensions of the distribution area

Verification of the induced tensile stresses

It is necessary to perform the verification of the analytical calculation in two directions, perpendicular to each other:

- the radial direction that cuts the pressure shoe along its thickness;
- the circumferential direction that cuts the shoe along its development.

Figure 5.66 indicates the distribution of the tensile forces inside an element.

The tensile force "Z", identified from Figure 5.67, should be taken from lining bars positioned in the stressed area and these can be identified from the abacus in Figure 5.68.

These forces should be taken from lining specifications, and can also be derived through the use of specific numerical models.

5.3.5.3 Long term Stability

The structural analysis of a concrete ring used as a lining in a tunnel is a subject that has been abundantly dealt with in literature. There are three types of method that can be applied to define the stresses in a lining ring, and that are briefly discussed below.

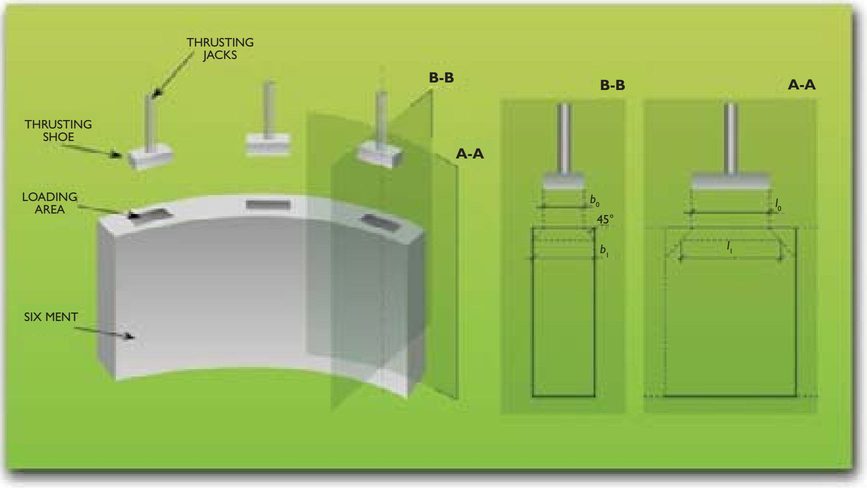


Figure 5.65 Geometrical definition of a single segment under the thrust of the TBM.

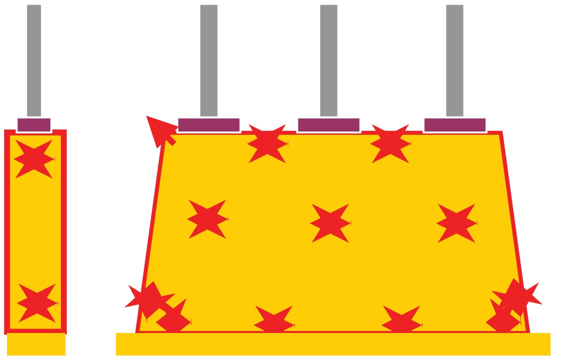


Figure 5.66 Distribution of the tensile forces acting in the segment.

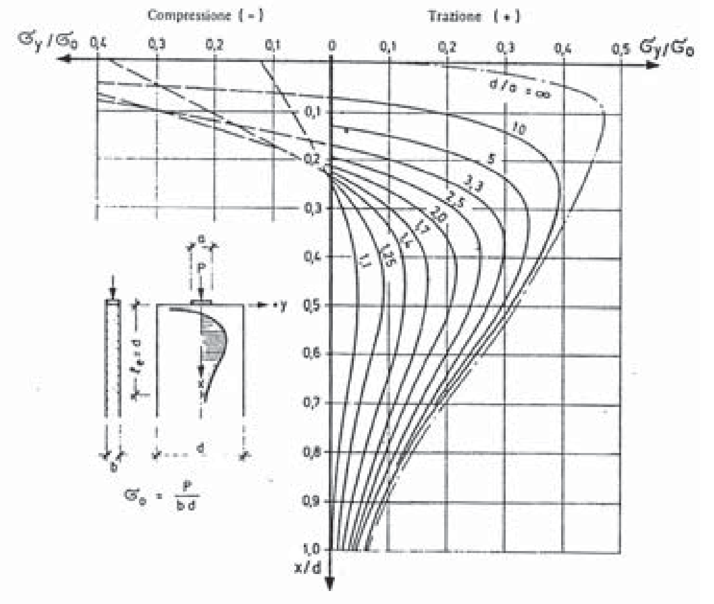


Figure 5.67 Calculation of the tensile force, z, acting in the segment.

1. *Analytical methods* define the actions on the lining based on the geotechnical characteristics of the surrounding ground and on the mechanical characteristics of the lining (stiffness, inertia, etc.), while attributing the entire geostatic load to the structure. The most commonly used method (Duddeck *et al.*, 1982) also considers different lining constraint conditions on the surrounding ground. The method supplies maximum axial force, N, and bending moment, M, and in different angular positions.
2. *Numerical methods with mono-dimensional elements* allow to: use the loads and internal ground loads) on the lining that are considered to be most appropriate,

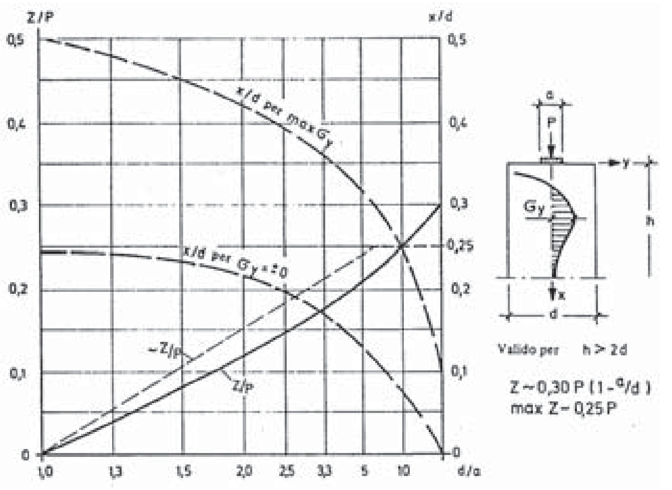


Figure 5.68 Calculation of the portion of segment where the tensile force is acting.

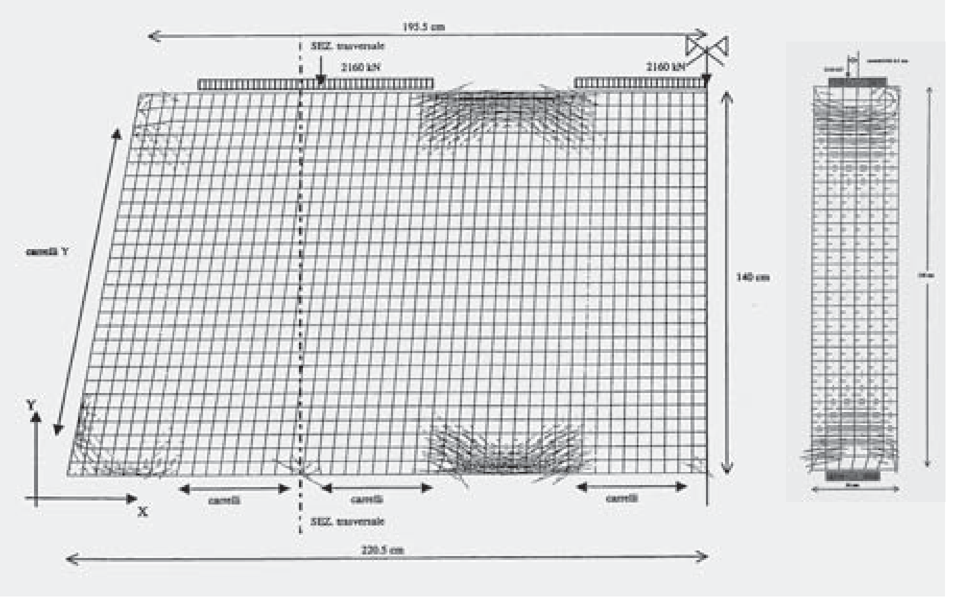


Figure 5.69 Tensile force defined by numerical analysis.

define the characteristics of the ground-structure interaction, and assign appropriate properties to the normal and tangential springs/Fig. 5.70).

3. *2D or 3D numerical methods* allow all the phenomena that are involved in the long term stability of a lining, from the moment it exits from the shield, to be modelled with more or less acceptable approximations (Figs 5.71, 5.72).

All three methods allow the phenomena, according to which the lining structure takes on loads and becomes stable in the long term, to be investigated with different degrees of detail. The more a study refers to an advanced stage of a project, the more it will be necessary to optimize and, therefore, make use of the sophisticated and time-consuming models.

The logical steps according to which a tunnel is excavated with a shielded TBM are illustrated in a conceptual manner (in Figure 5.73).

The progressive geostatic conditions indicated by the 5 sections of the tunnel shown in Figure 5.73 are expressed below:

Section 0. The ground is in the actual, in-situ geostatic condition (the acting pressure is P0).

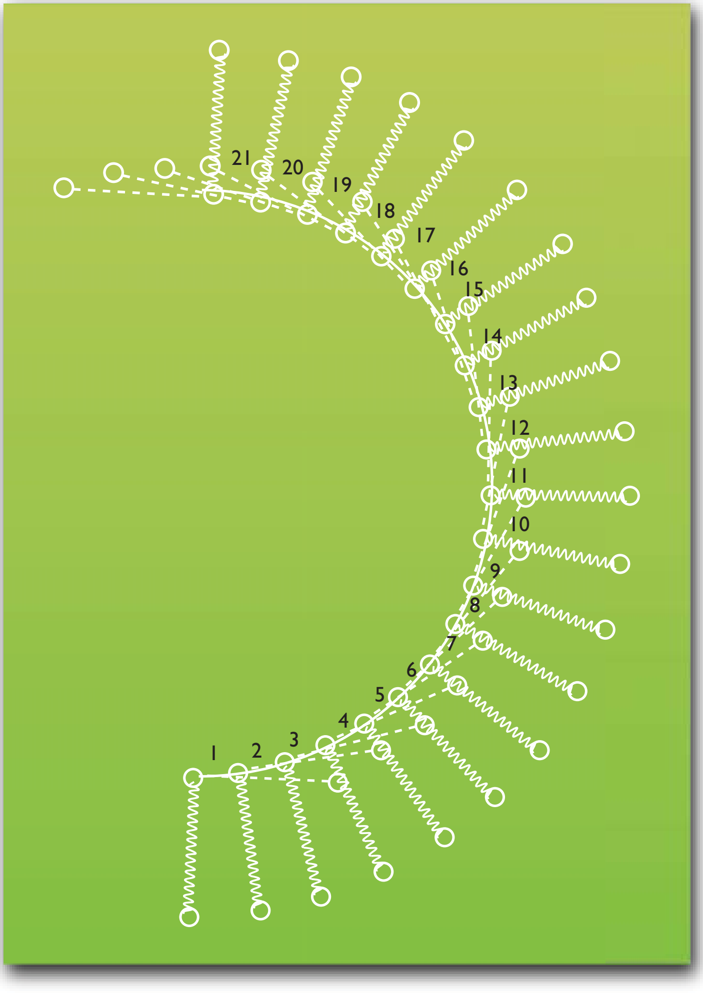


Figure 5.70 Bedded Beam Model (BBM model).

Section 1. The in-situ stress is reducing from P0 to Ps, the pressure applied at the face by the TBM.
Section 2. The internal pressure has reduced further and the convergence has stopped due to the presence of the shield.
Section 3. The ground and the lining (after exiting from the TBM) are loaded by the pressure of the longitudinal injection.
Section 4. The longitudinal injection has hardened and the long-term stabilization condition is reached.

Considering the specific theme of this section, it can be stated that a lining ring composed of precast segments has some characteristics that make it unique and which must be known and considered in order to perform a correct simulation, dimensioning, and stability verification. The special characteristics of a ring are:

1. The ring is assembled inside a shield and inserted/forced into the ground through filling the annular void with pressurized mortar.
2. The ring is made of assembled precast elements in a staggered manner like "bricks".
3. There is a section, in correspondence to the joints between the segments of the same ring, that is unable to endure traction which can therefore, only bear pressure-bending states with small variation.

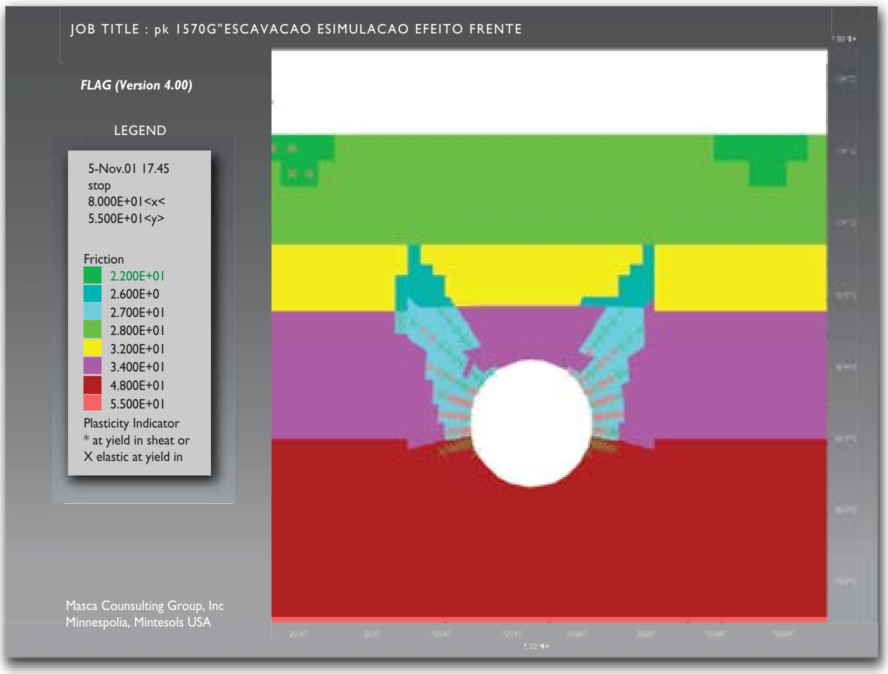


Figure 5.71 2D Finite Difference Method (FDM model).

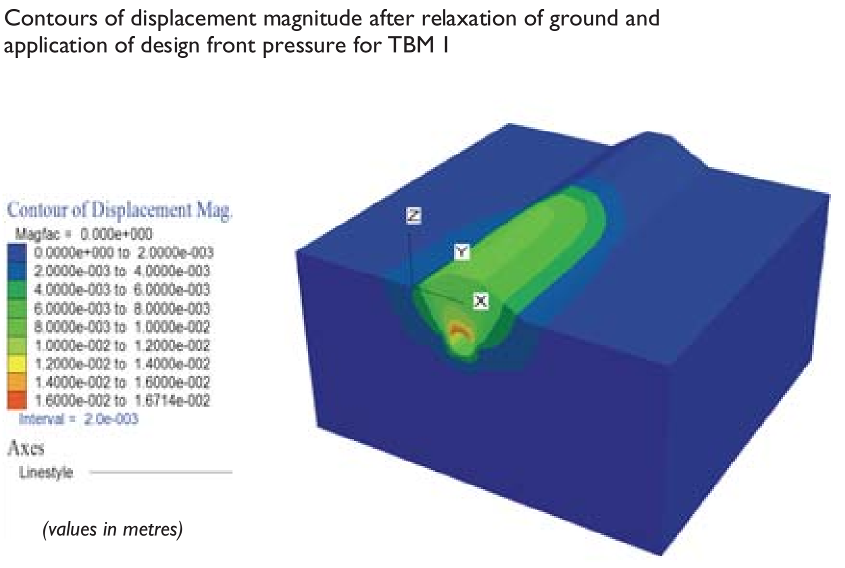


Figure 5.72 3D Finite Element Method (FEM model).

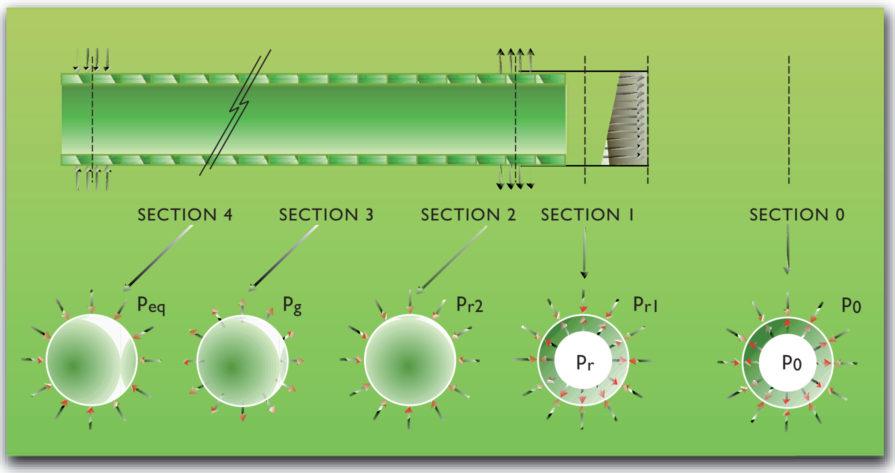


Figure 5.73 Logical steps for a numerical analysis.

It can be stated that this lining is only stable in the ground because it has been assembled on the basis of use in an excavation using a shielded TBM. The same structure, assembled in a circular tunnel excavated with conventional methods at a later stage, would be completely inadequate and unstable; therefore, a verification process should consider all these conditions. These two specific aspects are considered later on this section.

The effect of the longitudinal injection pressure

The value of the longitudinal injection pressure (see section 5.4) is generally related to the following aspects:

1. The value of the pressure at the excavation face.
2. The injection system.

The mean value of the longitudinal injection pressure will always be higher than the pressure applied to the face, and the variations in its maximum and minimum values will depend on the injection system which, however, must guarantee the maximum homogeneity of the distribution and the minimum oscillation of the instantaneous pressure value.

From the perspective of structural dimensioning, these injections can actually be considered by the structure-ground set as a radial hydrostatic pressure which acts simultaneously on the structure and on the ground in opposite directions.

This effect is determinant because at the moment the ring exits from the shield, it must be forced/blocked in the ground by this fluid which acts in order to:

1. completely fill the ring void;
2. give an external load to the ring that "encloses" it and tends to eliminate/limit any asymmetry of load on it (reduction/annulment of the bending moments), and
3. compress the ground at the boundary, eliminating any type of void that could have been created.

The injected material, when passing from fluid to solid, should maintain the static equilibrium that has been created; therefore, it should not be reduced in volume nor pressure-filtered into the ground. This situation results in the following two assumptions that have to be considered in the numerical analysis stages with 1D, 2D, or 3D elements:

1. The loads imposed on the structure can be applied with a radial distribution and values that can be chosen in relation to the natural in-situ stress state.
2. The stage where the ring exits from the TBM should be considered and both the structure and ground must be loaded by a radial pressure.

This installation stage of the ring, which must be simulated, is the one that allows the structure to be loaded by stresses characterized by high normal forces and low bending moments; these are the only stresses that are compatible with the type of structure under consideration.

The inertia of the lining ring

The lining ring does not possess a flexural inertia either equal to that of a real cylinder with a constant thickness or to that of a ring with aligned longitudinal joints. Flexural inertia is of fundamental importance to identify the acting stresses and strains, especially those that involve the joints. Its value clearly depends on the number of segments in the ring and also on the type of subsoil that interacts with the ring.

Because the various rings are continuously rotated, it is not practical to simulate the presence of the joints in determined positions, for example, with reduced thickness or special interfaces.

The simulation of this particular structure can be effectively done using the procedure recommended by the Japanese Tunnelling Association, which make it possible to explicitly consider the reduced flexure stiffness and to identify the value of the bending moments that act on the segments and on the joints.

Figure 5.74 illustrates the concept that is intended to be simulated:

1. The ring considered singularly is characterized by zones with both high and low flexural inertia, that is, the joints and the segments, respectively;
2. A sequence of rings is such that a joint in one ring corresponds to a segment in the previous and subsequent rings;
3. Such a configuration allows the excess moment that cannot be sustained by the joints in the adjacent segments to be transferred to the previous and subsequent rings.

This aspect is transferred to a numerical analysis using the following steps:

1. Correction of the elastic modulus of the ring, according to a factor ξ.
2. Calculation of the stress characteristics.
3. Modification of the value of the bending moment, increasing and decreasing the value for the segment and joint, respectively, by the same ξ factor (the normal force remains the same).

$$Ea = (1 - \xi) \cdot Ec$$
$$Mj = (1 - \xi) \cdot Mc$$
$$Ms = (1 + \xi) \cdot Mc$$

Ea = the virtual modulus of the ring
Ec = the concrete modulus
Mc = the bending moment derived from the analysis
Mj = the bending moment of the joint
Ms = the bending moment of the segment

The value of the parameter ξ varies between 0.3 and 0.5 as a function of the number of segments and the stiffness of the surrounding ground.

The structural verifications

When the stresses in the joints are available, the verification can be made using the procedure illustrated in Section 5.3.5.2 for the action of the jacks, because a joint can be considered as an external surface of the concrete which is precisely bearing a load acting on a determined area (see Fig. 5.75).

This phase that must be considered in the numerical analysis is the one which allows mainly high normal forces with low bending moments, the only one compatible with the specific type of structures to be exerted on the segments.

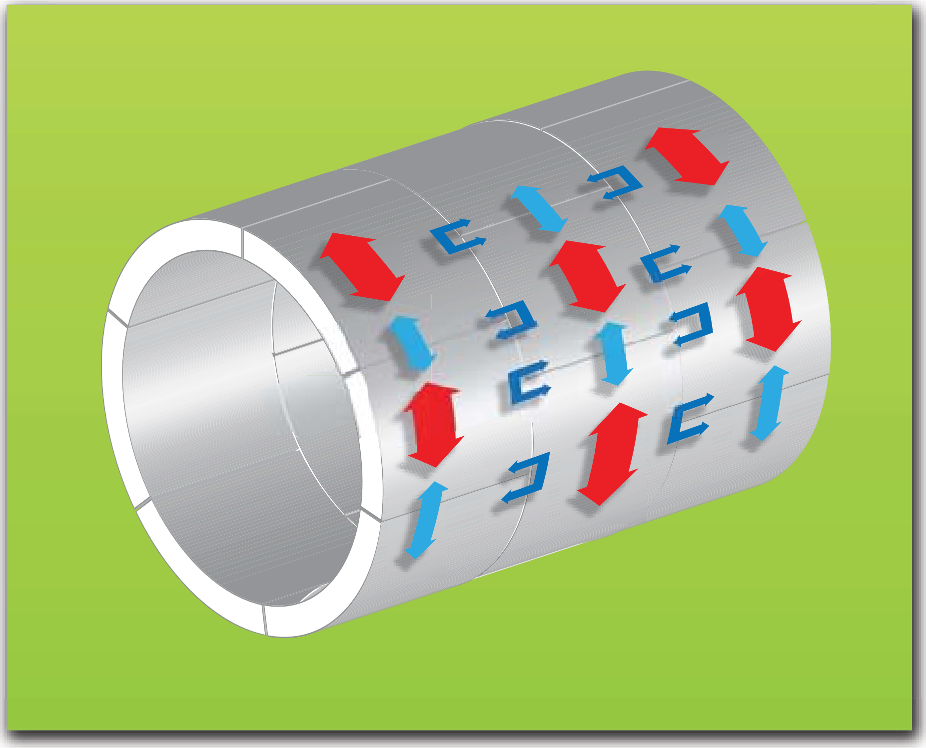


Figure 5.74 Flexural inertia for a segmental lining.

The segment section should instead be dimensioned as a normal reinforced concrete section.

The resulting steel cage is generally composed of:

- a regular, preferably electro-welded grid placed at the intrados and extrados;
- an increase of the reinforcement at the perimeter to resist the TBM thrust (long sides) and the forces exchanged in correspondence to the joints between the segments of the same ring (short side), and
- specific local reinforcement in correspondence to the bolts, connectors and erector-taking points.

As far as the universal ring is concerned, it can be noted that all the main dimensions of the segments are in fact different, but it is possible to standardize the steel cages, profiting of the possibility to vary the cover thickness.

One way of getting around this situation is to identify two types of steel cages: one for the segments having a medium to maximum width and one for the segments having a minimum to mean width.

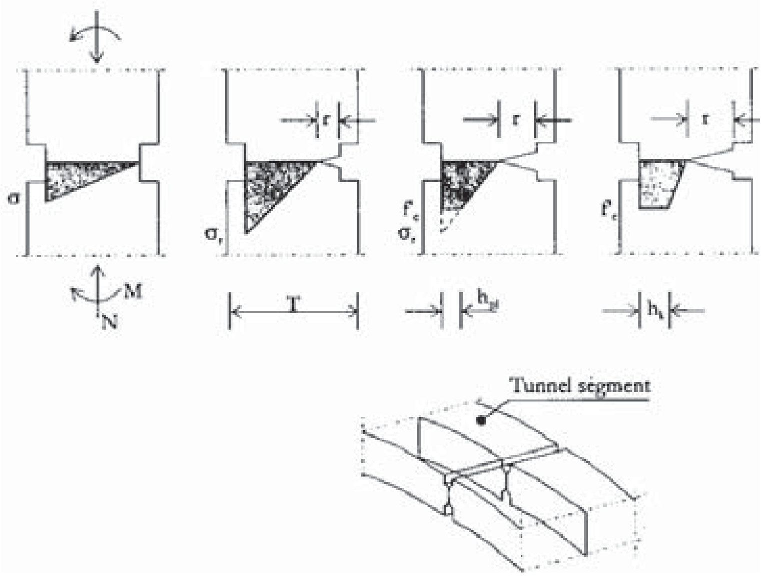


Figure 5.75 Verification scheme for a joint between segments (from de Waal, 1999).

5.3.5.4 Fibre reinforced segments

The recent development of a clear and standardized procedure for the calculation of concrete elements, reinforced only with fibre, and the possibility to speed up the prefabrication process by reducing the complexity of the steel cage or eliminate it, allowed to investigate the real possibility to use fibres as the only reinforcement for the concrete of the segmental lining.

As a result of these investigations, the best solution in terms of work and costs seems to be a mixed scheme for which the flexural resistance of the element is obtained by the use of traditional steel bars, while all the remaining steel is completely substituted by fibres

In terms of the type of fibre to be used, it is not possible to give clear instruction since the effect must be considered in terms of increased tenacity of the "new material", i.e. the fibre reinforced concrete. Moreover a preliminary choice must be made in terms of the material that constitutes the fibre: steel or plastic. Both are on the market and can be theoretically used, even if, up to now, just the steel fibre has been tested in several projects.

5.3.6 The prefabrication process

In this section, attention is only focused on those aspects of the prefabrication process that could be of interest concerning the prefabrication of segments which are assembled as part of the tunnel lining.

5.3.6.1 The materials

The main constituents of a tunnel segmental lining are:

- concrete (containing water, cement, aggregates, and admixtures), and
- reinforcing steel.

It is sufficient to follow the applicable standards for selection of both types of material, concrete and steel. In addition, the following specific aspects should be considered for the ingredients of concrete:

Cement

Preference should be given to the use of additive-free rapid-hardening cements, whose durability is only marginally affected by steam curing, provided the resistance to aggressive ground conditions is assured.

Aggregates

The aggregate sizes should perfectly suit the geometrical accuracy of the segments, the form work recesses, the reinforcement arrangements and the possible connector inserts. A maximum dimension of 25–30 mm is generally recommended.

Admixtures

In case the aggregates lack fines, the use of additional fly-ash or fillers (limestone based materials) are recommended. The origin of such products should, of course, be checked.

The use of standardized water-reducing superplasticizers is recommended to obtain increased workability in order to achieve higher strength and a perfect filling of all the spaces between the reinforcing bars.

5.3.6.2 The plant

The durability of the lining is generated first and foremost in the prefabrication plant, which should have the following basic elements:

- an efficient plant, and
- quality procedures made available and maintained for all the work phases.

The prefabrication process of the segments that are used in a tunnel is part of a production cycle that requires a high level of industrialisation and control of each single operation.

The aim of the process is to produce all the high quality segments that are necessary to satisfy the excavation rates of the tunnel(s), according to a kind of "assembly line", which produces a ring at the end of each cycle.

The process essentially consists of:

- storage areas for the aggregates that should, if possible, be protected from atmospheric agents;
- a mixing plant including systems for the automatic measuring and recording of the mix data;
- a series of moulds that may be:
  - positioned to form a carousel in line with the furnace;
  - fixed, with a steam distribution plant.
- a steel cage assembly area;
- a pre-storage area where the segments are protected from atmospheric agents, and
- areas for final storage of the segments outside the cover part of the plant.

The "carousel" plants (which are at present frequently used) are particularly efficient as the "formworks" can be moved from one place to another using appropriate hydraulic jacks that allow to carry out the specific, repeated operations in the same position. In this way, it is possible to optimize the processes of the supply of the materials, shifting of the necessary equipment, and intervention of the necessary workforce.

5.3.6.3 The moulds

Particular care is given, in the plant, to the formworks which should generally satisfy three essential requirements (see Fig. 5.76):

- enhanced workability and robust performances in order to guarantee the production of segments with a tolerance of less than millimetric;
- exact connections between the sides, to avoid the spilling of grout, and to maintain the perfect repetitiveness of the device, even though a remarkable quantity of re-employment is requested;
- easy opening and closing of the sides and of the covers to allow a lowering of the preparation and dismantling times, using specifically studied mechanical devices for this purpose with the assistance of hydraulic cylinders for the larger sized elements.

5.3.6.4 The working cycle

The elementary operations that are performed to prepare a segment for transport into the tunnel can be summarized as follows.

Preparation of the moulds

Before carrying out a new casting, it is necessary to perform the following basic operations:

Figure 5.76 Example of a typical mould.

- an accurate cleaning of the residuals of the previous casting, not only inside but also in correspondence to the mechanical devices that make it possible to move the sides and covers;
- placing of lubricant on the internal sides of the moulds, and
- positioning of all the accessories that have to be present in the concrete (nuts for the connectors or bolts for the TBM hardening system).

Construction of the retaining ring

The steel cages are assembled at the same time. This ring should clearly have a curved form with a radius that is derived from tunnel intrados radius with the addition of the cover-side.

One system that is often used to avoid the mechanical rolling of the steel bars placed at the intrados and extrados is to bind them straight onto the support made to assemble the reinforcement. The curved shape is in fact obtained by forcing the bars into the support and then binding them and welding them together.

A serious problem could occur when the steel cage is removed from the support. Because of the elasticity of the steel, the curved steel cage tends to straighten, assuming a different and lower curvature than the designed one. Therefore, the coverside is not maintained in the centre of the segments by defect and by excess at the intrados sides and vice versa at the extrados side (see Fig. 5.77).

Insertion of the steel cage into the moulds

The steel cage is let down from a bridge crane into the mould and is positioned with the help of spacers to ensure/control that the cover-sides is maintained on all the edges of the segments.

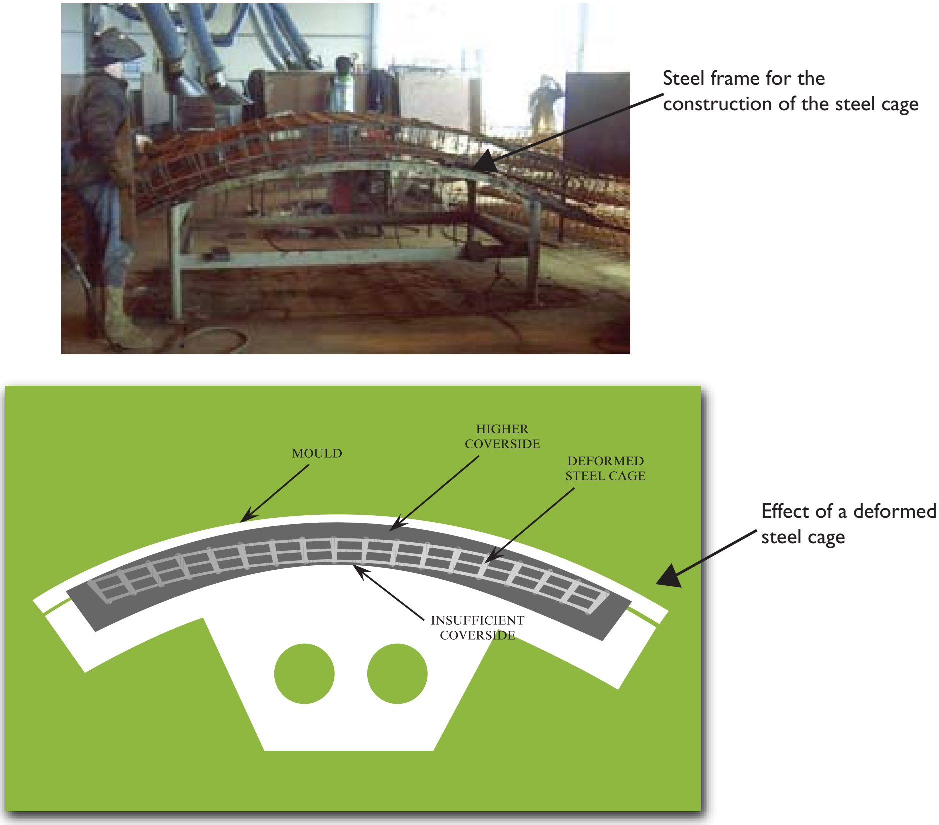


Figure 5.77 Variation in the cover-side due to imperfect geometry of the steel cage.

Closure of the doors

All the sides are closed, ensuring that the "accessories" that are present, and which must be covered in the casting, are all in position and are not in contact with the reinforcement bars.

Casting and the beginning of hardening

The casting takes place from the central upper window of the mould (on the segment extrados side). The vibration process is started at the same time as the casting to ensure that all parts of the moulds are filled with the concrete.

Smoothing

It is normal that air bubbles form, during the vibration, because of the geometry of the upper door. These bubbles give an extremely irregular shape to the segments at the extrados. This incidence is particularly problematic and difficult to eliminate:

the negative effects are not so much an aesthetic problem, which is not particularly significant as it involves only the tunnel extrados, but rather a problem concerning the consequent and inevitable waste of grease from the TBM tail brushes which is very expensive and highly polluting. In addition, attention should be given to anomalous wear on the brushes. In order to get around these problems, apart from studying a suitable mix and using suitable admixtures, it should be foreseen to open the upper door of the mould as soon as the casting has started to harden and to carry out a complete smoothing of the fresh concrete surface.

Steam-curing cycles

The steam-curing cycle follows the typical steps of prefabrication with steam-accelerated hardening; These steps are simply mentioned here to complete the explanation of the process (see Fig. 5.78);

- insertion of the mould with the fresh concrete into the furnace;
- heating with controlled positive thermal steps (20–25°C/h);
- treatment at a constant temperature (55–60°C);
- cooling with controlled negative thermal steps (about 20°C/h);
- cooling from the outside, and
- extraction from the mould.

Extraction from the mould and possible turning over

The extraction from the mould is usually performed through the use of vacuum lifter to reduce the risk of damaging the segments. Sometimes, just after the extraction (and

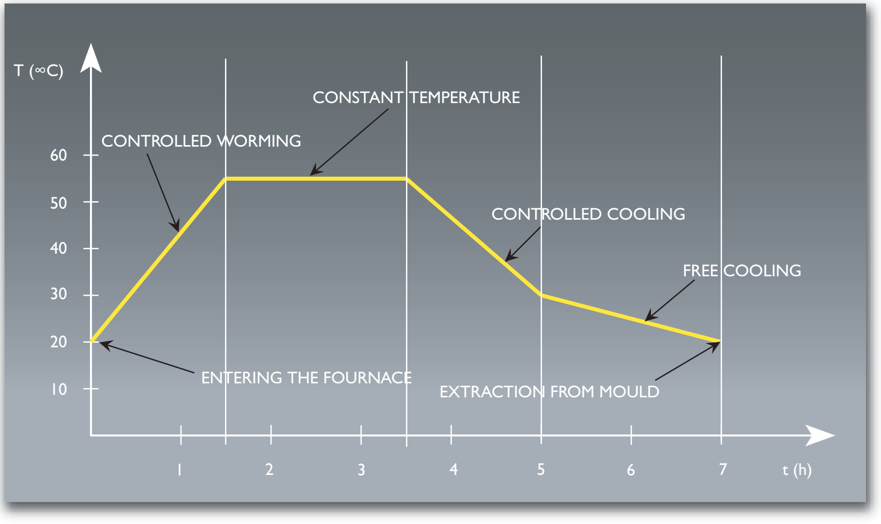


Figure 5.78 Typical steam-curing cycle.

Figure 5.79 Segment extraction from mould and rotation.

based on the adopted logistic of the plant) the segment can be turned over through a special machine (see Fig. 5.79)

Positioning of other accessories

Once the segment has been turned over, inside the plant, and is therefore protected from atmospheric agents, it is completed with the addition of the gaskets, bituminous pads, and possibly plugs to close the holes.

Storing

This operation should be conducted in two stages:

- a first storing inside the plant protected from atmospheric agents for at least 24–48 hours, taking care to place no more than two segments one on top, and
- final storing in the yard, where the segment is carried to an area where it will be left to harden completely (typically 28 days) and where it can be piled up in a sequence that corresponds to the geometry foreseen at the design stage.

5.3.7 Ring monitoring

The monitoring of rings is only one part of a more complex and larger operation, of monitoring a tunnel in an urban area, which has been dealt with in other sections of this book. Nevertheless, it is important to focus on the particular aspects of mechanized tunnelling compared to a traditional method for the specific subject related to rings.

Monitoring has the purpose of ascertaining the stability of an excavation, confirming the installed safety factors of the structures (in particular, the first stage and final lining), and confirming the design hypotheses in order to be able to adapt the interventions to the real conditions that are encountered (i.e. to put into practice the previously defined countermeasures).

In a traditional excavation, these aims are reached by putting the instrumentation into the structure and taking note of the readings on the structures that are gradually installed from the face. Instead, these operations are particularly difficult for mechanized excavation due to the almost physical impossibility of operating because of the obstacles encountered in the back-up.

One of the prevailing factors in the understanding of a monitoring project is the possibility to connect the effects of the excavation process in the excavation face zone (that produce the perturbations) to the response of the support structure.

The first parameter to be checked is the convergence, which is easy to obtain and which can then be used to interpret all the remaining data obtained from the set of other instruments. This parameter is in fact less significant than all the others involved in mechanized excavation. It can be easily obtained with a certain systematic nature and precision even at remarkable distances from the excavation face. It is, therefore, almost completely unconnected to the excavation, to the longitudinal injections, and to its subsequent hardening, that is, once stability has been achieved.

Therefore, the way in which the readings of the instruments on the final lining of a mechanized excavation are read, is in fact the opposite to the process used during excavation with traditional methods.

Control of the stability of an excavation is entrusted to the monitoring of the machine parameters and of the effects of the surface (settlement and displacements of the already existing buildings).

The verification of stability and safety of the support structures is carried out after the excavation has been completed and has become stable, through special instrumented sections for measuring stresses in the reinforcement bars of the segments, stresses in the concrete, and contact pressures at the extrados of the lining.

From this point of view, the precast concrete-segment-lining instrumentation and monitoring schemes should only satisfy the following specific aims:

- to check the present and final condition of the lining in terms of forces and deformations with respect to the design hypothesis, and
- to gain greater knowledge of the magnitude and distribution of the external actions that have an impact on the lining, and the internal load of the segment.

Fig. 5.80 shows two typical monitoring sections of the structures from which the information connected to surface movements is also acquired in order to have a more complete database.

It is necessary to underline that it is particularly difficult to manage the instruments that are immersed in the concrete during the castings in the prefabrication plants. These should be arranged and positioned while in the prefabrication plant and repeated readings should be performed for the entire period before the segments are taken into the tunnel in order to be able to follow the evolution of the measurements and to prevent the risk of attributing the effects to the wrong causes. The following readings should, therefore, be performed as a minimum, to verify a baseline:

- just after removal from the mould;
- before being placed in the yard for the final storing;
- before being transported to the portal;
- before the ring is installed;

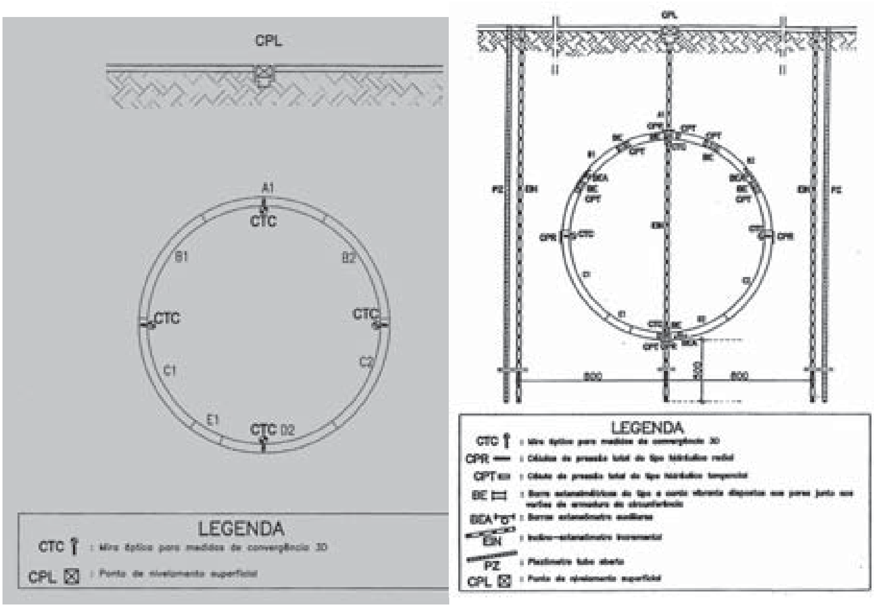


Figure 5.80 Current and special monitoring section.

- just after the installation;
- after coming out of the TBM tail, and
- once a day until the longitudinal injection has hardened substantially.

5.4 BACKFILLING OF THE TAIL VOID

The difference in size between the minimum diameter of cylindrical cavity cut by a modern tunnel boring machine and the outside diameter of the precast concrete lining, the tail void, is typically in the range of 150 mm to 370 mm, depending on shield size and the minimum radius of curve to be negotiated.

The backfilling of the void around the lining is essential for the success of the tunnelling process. It is a critical operation that performs the following fundamental functions:

- Reduce surface settlement above the tunnel. If the void is not properly filled with grout, the ground will move into the void, resulting in settlement. Typically the volume (per meter of tunnel) of the tail void is in the range 3% to 16% of the internal volume of the tunnel. There can be high surface settlements if the backfilling is ineffective and the tail void closes as a result;
- Ensure uniform contact between the lining and the ground. The ground loads the lining and also provides resistance to distortion; consistent filling of the tail void will prevent uneven loading;

- Hold the ring in place during shield advance. If the lining gets surrounded by liquid injection, it can float upwards, according to Archimedes' law. This can lead to stepping on the circumferential joint, loss of plane and general damage to the lining;
- Carry the load transmitted to the lining by the TBM back-up;
- Reduce seepage and loss of fine particles from the surrounding ground where the gaskets is ineffective due to damage or because of stepping of the lining.

As a total result, the effective backfilling by grouting helps to minimize settlements, by holding the rings in place during shield advance, and assures long term stability of the structure submerged into the ground.

The backfilling system could be classified according to the injected mixture.

5.4.1 The methods

From the point of view of methodology, it is possible to specify three main types of injection used in practice, listed below in a chronological order:

- Radial injection through holes provided in the concrete lining.
- Longitudinal injection directly through the shield, advancing simultaneously with TBM.
- Longitudinal injection through the shield, with 2-component grouting systems, simultaneously with TBM advance.

Traditionally, grout has been injected through grout holes in the tunnel lining: the segments are provided with holes fitted with screwed connection pieces and closed during ring installation by plugs. Another alternative was the introduction of plastic no-return valves integrated into the segments.

This worked adequately prior to the introduction of modern, pressurized face machines. With the open face shields, unstable ground had to be stabilized by compressed air or ground treatment. The pressurized TBMs can apply large face-support pressures at the face, but little or none of this pressure is transmitted to the tail void.

Grouting through the tunnel rings cannot be carried out until the grout holes have passed beyond the position of the tail seals. As a result unstable ground is likely to collapse onto the lining before grouting can be carried out, generating important settlements. This system was very ineffective with respect to subsidence containment and risk management of structures located within the area of influence of excavation (Fig. 5.81).

In order to overcome this problem it has become common practice to have grout pipes built in the tail skin, with the injection points at the end of the tail skin (see Fig. 5.82). With this arrangement it is possible to grout simultaneously with the advance of the shield machine, turning the annular void into just a "virtual void".

Simultaneous backfilling grouting was carried out in shield tunnelling for the first time in 1982 in the construction of No. 4 line of the Osaka subway, in Japan, resulting in a considerable reduction of subsidence.

For SS or EPBS excavation, backfilling grout injection occurs in a continuous way during the TBM advance to make the overall system watertight. The grout is injected by two or more pumps placed on the back-up, generally through six pipe lines ending

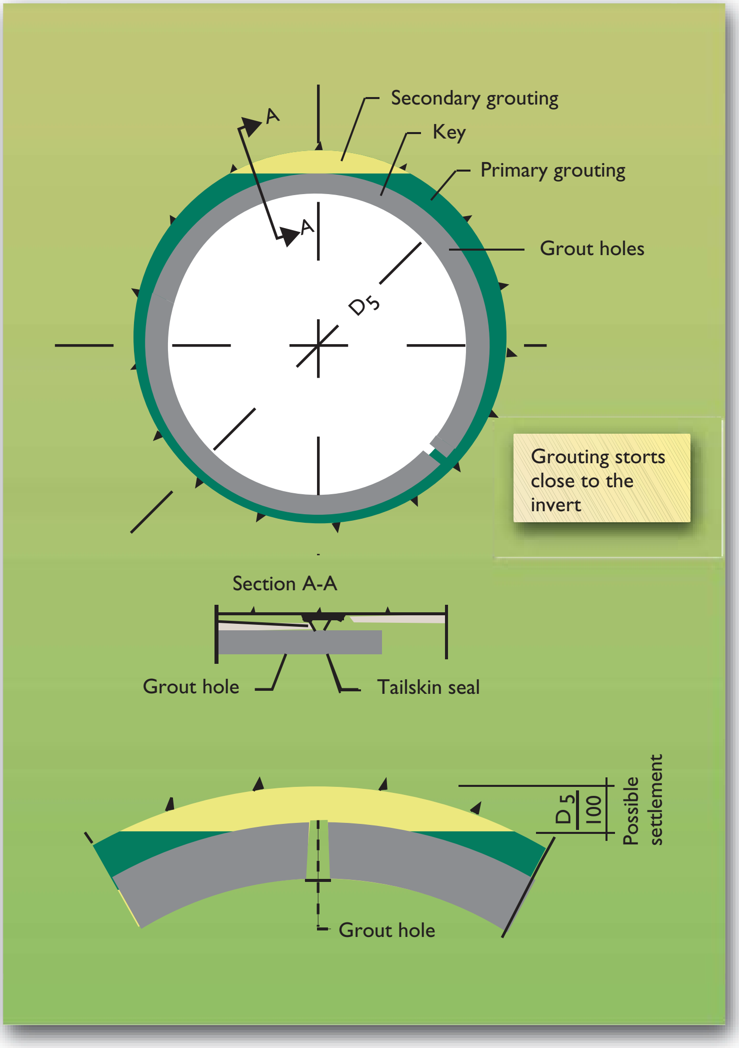


Figure 5.81 Radial backfilling through the lining.

above the rows of steel-wire brushes, and positioned along the perimeter of the rear shield. Injection occurs at the shield edge, directly on the extrados of precast segments of the final lining (see Fig. 5.82).

Each injection line often has a reserve pipe, in order to switch the flow in case of main-line blockage, i.e. due to clogging of the grout: double-line system allows standard working condition for the machine also during cleaning and maintenance operations of occluded line.

The above injection method has been introduced in many regions of the world, such as Asia, Europe, and America, reducing the associated settlement with shield tunnelling. The inclusion of reserve pipe is recommended for mechanized tunnelling in urban areas.

5.4.2 The mix design

The next step is to optimize the properties of the backfilling material. Nowadays, from the point of view of grout mixture, it is possible to define the following terms:

- Inert mortar, with no cement content in the mixture.
- Mortar, with low cement content in the mixture.
- Cement based mortar.
- 2-components mortar.

The grout must provide effective support to the lining, and prevent it from moving during TBM advance. In particular, the grout must prevent the lining from floating due to the fluid pressures exerted by the grout itself and by the water, if any. The use of mortar-grouts that can achieve this goal by their rheology is becoming more common.

The recent applications of 2-component grout use liquid A (cement, clay and/or bentonite, water, retardant additives) and liquid B (water-activated additive): After

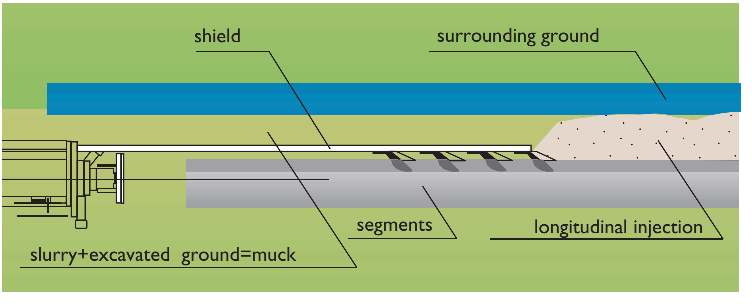


Figure 5.82 Longitudinal injection system.

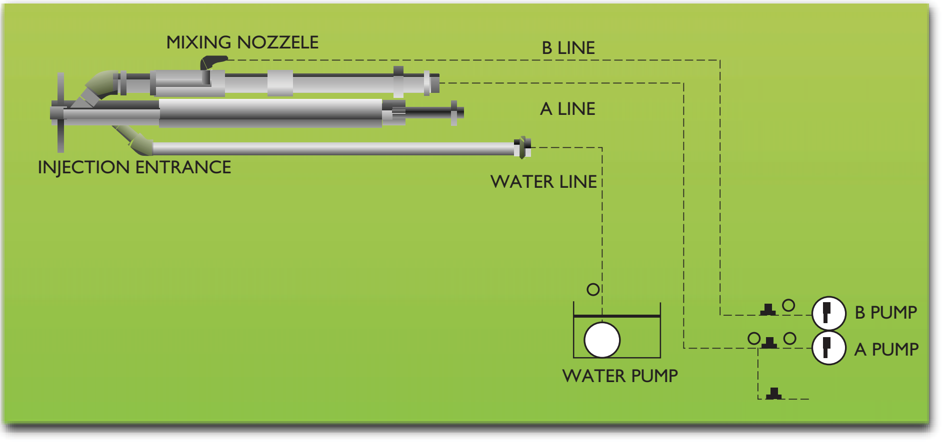


Figure 5.83 Equipment for simultaneous injection with 2-component grout system.

mixing the two liquids, the grout assumes a semi-solid (plastic state) in a few seconds and keeps this state for about half an hour.

Then it starts to become hard, generally reaching a compressive strength of 0.05–0.1 MPa within 1 hour. The gelling and hardening time can be set to meet the specific project requirements. If the two liquids are injected together through one pipe, the pipe is often clogged due to the short hardening time of the liquids. Therefore, the injection system should be equipped with two extra mechanisms, one for mixing the two components just before entering the tail void, and one for cleaning the pipe (see Fig. 5.83).

In Europe, examples of this method for backfilling are the Botlek rail tunnel (1998–2001), where some tests with a Japanese 2-components grout (ETAC) were performed by injecting directly through special made pre-installed openings in the lining, Genova Metro line (1993–1994), or Naple metro line (2005-today) and Castellanza railway tunnel (2005–2006), where the injection of the 2-components mixture passed through the shield, by a system similar to the one shown in Figure 5.83.

In this way, the system delivers a more efficient tunnel boring process with a fast and uniform support action on the tunnel lining (even if clogging risk for the injection system is always present) and, as a consequence, a better control of grout injection operation and, indirectly, of the potential subsidence.

The injection process requires a considerable amount of grout, i.e. several cubic meters per lining ring. The equipment required for preparation plants with storage silos, mixers etc. are normally located outside the tunnel.

The grout is then transported to the injection point by track-bound or other vehicles and passed on to the grout pumps. Transfer of grout can also be by pumping.

5.4.3 Performances required for the injection

The potential delays in starting, and during performance, on the injection activity could have negative consequences on the mortar characteristics (anticipated hardening process, loss of fluidity, loss of future resistance capacity, segregation phenomenon on the circuit pipes inside the shield part, whose obstruction could cause serious problems. For these reasons, the design of the grout mixture has to be properly studied in order to provide the adequate mechanical characteristics.

A correct design of a backfilling grout mixture, should consider the various parameters that refer to three main application fields: 1) practical aspects, linked to workability of the grout to be injected; 2) the effectiveness of the injected grout with respect to settlement control and interaction with the lining and the boring machine; 3) economical consideration linked to locally available, and probably cheaper material versus the ideal mixture for the required performance.

The following are some of the practical aspects or “requirements” regarding good pumpability of the grout:

- High workability conservation during storing and transporting time.
- Mixture stability during storing and injection time; in particular the mixture should not be affected by segregation phenomenon.
- Smooth flow of the mixture during injection.
- Good pumpability through narrow pipes and, eventually, at great distance.

All these requirements are very important to allow effective tunnelling operations, in order to avoid blockages of the pipes due to clogging, or unsuitable working conditions of the injection system.

For an effective backfilling action the main requirements for the mixture are:

- Perfect filling of the tail void.
- Slightly deformable resulting material, once the grout has hardened completely.
- Slightly abrasive material.
- Low permeability of the grout, in order to avoid loss of the fine size components in the surrounding soil during injection.
- No reactant material to wash-out action.
- Short setting time, compatible with injection timetable with respect to the TBM advancing mode.

In addition to the requirements mentioned above, setting time has a fundamental role in effective backfilling with grouting. The use of slow-setting grouts could possibly have great, practical benefits for efficient tunnelling: by using a slow setting mortar, the risk of blockages in the grout lines and pipes, or of damage to the tail seals during temporary stoppages, is greatly reduced. However, several limitations may need to be considered.

Bezuijen *et al.* (2002) provide the results of a study on the flotation forces, according to Archimedes' law, that were experienced. The forces exerted by the grout in a field case were measured. On initial injection the pressure gradient between the top and bottom of the rings was very close to that obtained by treating the grout as a liquid with a density of 2190 kg/m^3. After nearly 11 hours the pressure gradient was approximately equivalent to that which would apply if the rings were surrounded by water. This study demonstrates the magnitude of the buoyancy forces exerted on the ring. The study also identifies methods to estimate the yield stress that must be achieved in the grout in order to resist the buoyancy force.

If the mortar hardening will start after 8 to 10 hours or more after its injection, more dangerous phenomena could occur: one of that is the floating of the lining tube (i.e. some offset between the shield and the first few rings immediately behind the TBM). Actually, a first consequence of this offset position will be an abnormal pressure on the upper shield brushes and, at the same time, possible flow out of slurry in the bottom contact ring-brushes. This may lead, in turn, to loss of pressure in the chamber and, therefore, to the instability risk mentioned above.

In addition, changing damaged shield brushes requires special maintenance actions, very complex and onerous from point of view of time and space.

Possible damages to longitudinal junctions between contiguous segments could also occur, with all the consequences in terms of waterproofing action of the tunnel and global stability of the final lining.

If the setting time is not well calibrated with TBM advancing speed, it is possible to create unforeseen overloads on the lining, which will cause an additional bending moment on the segments, acting in the plane parallel to the axis of the excavation.

Segmental lining behaves similarly to a continuous beam that is subjected to a distributed load, generated by the injection pressure. If the grout hardens too slowly the beam is free to deform, being immersed in a fluid with constraints only at the terminal points, represented by the grout already hardened from one side and the boring machine support from the other side.

This occurrence may create serious structural damage to the lining because it is difficult to predict during design stage. To reduce the forces on and the moment in the rings, it is necessary to limit the length of the lining that is surrounded by not hardened grout.

Vertical gradient (and thus the loading on the lining) is highest behind the TBM, where the grout has the lowest viscosity and yield stress.

The grout applied influences the loading on the lining, in particular the distribution of the loading perpendicular to lining axis. It is of importance that the unsupported part of the lining (where buoyancy forces dominate) is kept as short as possible to reduce the moment in the lining, and high vertical forces at the tail skin and at the last ring where the grout is already hardened. This can be achieved in three different ways:

1. The grout has a relatively high initial shear stress. In a situation with a high initial shear stress of the injected grout, the shear strength in the not-yet-hardened grout is already sufficient to prevent upward movements of the tunnel lining;
2. subsoil and grout allow for a rapid consolidation of the grout, resulting in an increase of allowable shear stress in the grout and, as a result, in only a limited unsupported length of the lining;
3. the grout used hardens quickly. This also leads to a reduced unsupported length of the lining.

These three approaches can be used for tunnels excavated in sand, while in clay the second is not available, because the low permeability of the clay prevents consolidation of the grout.

In conclusion, grout properties in combination with the soil properties influence the loading on the lining directly behind the TBM. It is, therefore, necessary to select a grout taking into consideration the soil properties at the location and desired grout properties (yield stress, bleeding and hardening parameters).

It is necessary to take into account the evaluation of subsidence for a correct risk management and control during tunnelling works. Rheologic properties of the grout are mainly linked to this topic, in terms of 1) permeability of the mixture with respect to the surrounding soil one, 2) deformable potentiality after hardening phase, and 3) loss of volume during setting further to consolidation phenomena.

Bezuijen and Talmon (2004) point out, based on field measurements carried out for tunnelling in sandy material, that it is possible to quantify a volume loss due to bleeding, of up to 5% of the injected ground leading to a reduction of the grout layer, which will, in turn, cause a consistent pressure decrease, according to the formulation proposed by Verruijt (1997).

For all that concerns strength properties of the mixture once hardened, usually high performance is not required. Once injected, the grout is well confined by the surrounding subsoil and, in this condition, cracking the grout that is under compression stress is not so likely.

However, in case specific situations, for example, during deep tunnelling or obviously high stress conditions in the ground, special characteristics for backfilling grouts may be explicitly required.

Typically, from 3 to 10 Mpa is a sufficient value to grant satisfactory performance from this point of view. Instead, peculiar attention should be given to shear resistance

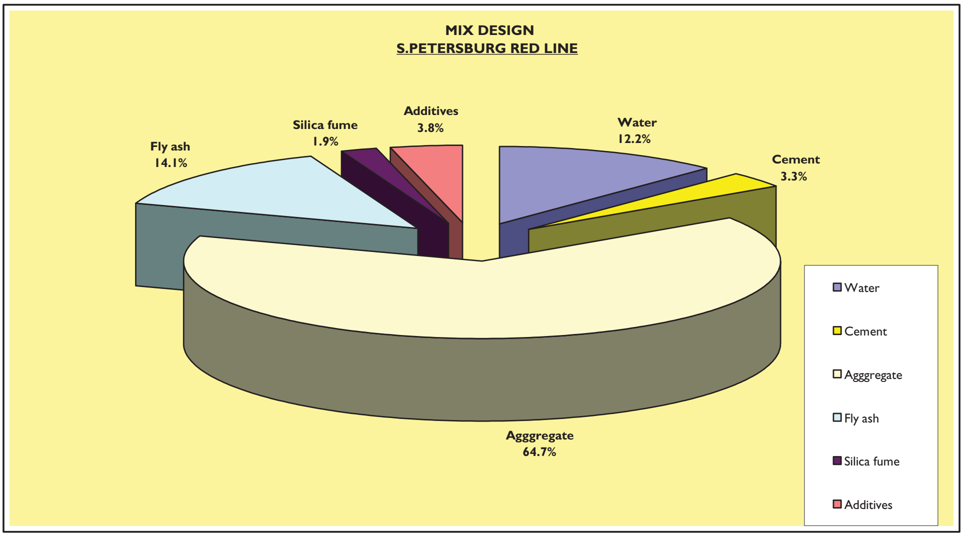


Figure 5.84 Mix design of a cement based grout, AV – Penetrazione urbana del nodo di Bologna.

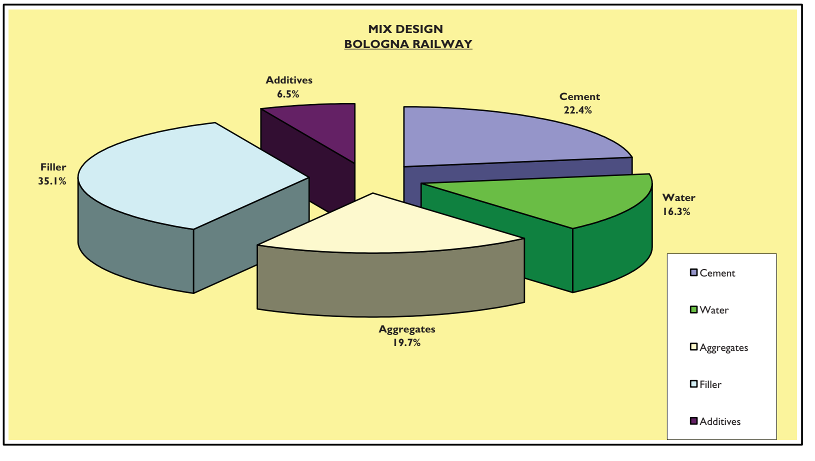


Figure 5.85 Mix design of a 2-components based cement grout, Castellanza Railway tunnel.

of the grout during injection phase and before hardening, because this parameter severely influences ring floating occurrence: high viscosity properties involve the arise of shear strength in the injected mixture, which behaves as a Bingham fluid, that can resist, thanks to internal friction, buoyancy force.

The wide variety of mixes used in practice (see Table 5.20) indicates that there are many different ways of achieving the same basic goals. The fast-setting, cement-based grouts have obvious attractions in terms of providing rapid, short-term strength

Table 5.20 Backfilling grouting, some examples of mix design

TUNNEL			*EOLE*	*ST. PETERSBURG*	*PORTO*	*TORINO*	*BOLOGNA*	*CASTELLANZA*	
REFERENCE			*section 8.1*	*section 8.2*	*section 8.3*	*section 8.4*	*section 8.6*	*appendix 7*	
COMPONENTS			*No cement*	*Low cement content*	*Medium cement content*	*Medium cement content*	*High cement content*	*2-components*	
								comp. A	comp. B
Cement	Kg/m^3		=	70	200	220	370	341	=
Water lime	Kg/m^3		=	=	60	=	=	=	=
Silica fume	Kg/m^3		=	40	=	=	=	=	=
Fly-Ash	Kg/m^3		400	300	120	380	=	=	=
Active Fly-Ash	Kg/m^3		200	=	=	=	=	=	=
Filler	Kg/m^3		=	=	=	=	580		=
Bentonite	Kg/m^3		40	=	=	=	=	43	=
Aggregates	Kg/m^3	(0–2 mm)	=	=	=	=	880	=	=
		(0–5 mm)	1000	1380	1530	=	=	=	=
		(0–6 mm)	=	=	=	1250	=	=	=
Water	l/m^3		250	260	230	250	270	812	=
Additives	%	plasticizer		4.1	2	4	3		
		retarder			0.1				
		air-entrainer					4		
		stabilizer						5	
		jellying agent						4	
		hardener							70

to hold the ring in place during TBM advance. However, this type of grout has the disadvantage of requiring frequent flushings of injection pipes and the constant risk of pipe blockage.

The potential for sand-based mortar grouts has been recognized. The problem with this type of grout is how to achieve a mixture that is easily pumpable, has a very long effective setting time, and yet holds the ring in its place as soon as it is placed into the annulus around the ring. Ideally, such a grout should be developed from the cheapest, locally available and consistent materials.

5.4.4 Control of the backfilling process

Backfilling injection is a dynamic process that involves several technical parameters, which have to be monitored and systematically compared with the design values. The procedure for monitoring and control of the injection process (with the objective of minimizing the potential risks) involves three main components, which are described below.

1. *Control of grout volumes:* injected quantities must be correlated with the theoretical voids, calculated on the basis of a single advance (Eq. 5.6).

$$V_{VOID} = \left(\frac{D_{EXC}^2}{4} \cdot \pi - \frac{D_{EX-LIN}^2}{4} \cdot \pi\right) \cdot L_{RING} \tag{5.6}$$

   Where V void: volume of the void.
   Dex x: diameter of the excavation section
   Dex-LI: diameter of the lined section
   Lr: length of the ring (parallel to the tunnel axis)

Systematic check of the backfilling-grout volume is fundamental for controlling the ground surface settlement.

The theoretical grout volume (expressed by the above equation) to fill the gap between the excavation profile and the lining extrados, considering unworn excavation tools, is calculated as the difference between the global area of excavation and the section at the lining extrados, evaluated on the total ring length.

The quantity of the injected grout will vary as a function of 1) the real stroke of advance, 2) the permeability of the mixture/subsoil, and 3) the wearing of reamer cutters (which reduce the excavation diameter).

In fact, the real stroke of advance can vary by a few centimeters from its theoretical value, because of the trend of the alignment, the permeability of the soil can increase the quantity of grout to be injected, taking into account also the grout fluidity during the injection phase, and the wearing of the cutters reduces the overall quantity to inject.

A grout volume significantly higher than the theoretical one may represent an over-excavation condition or a dispersion of the grout in a natural pre-existing cavity presents in the surrounding ground.

2. *Control of backfilling pressures*: check whether the final pressure is consistent with the design reference, which is correlated to the confinement pressure.

Today the grouting systems are generally not only volume-controlled, but are also pressure-controlled. Ideally, the grout pressure should be measured in the zone to be grouted (the Tail void). However, for practical reasons, pressure-measurement is carried out only in the grout pipe in the shield or in the concrete segment.

Backfilling injection should always occur at the same time as the boring progress, and should follow the procedure indicated in the advancement specification. In particular, the advance speed of the machine needs to be adapted and calibrated to the backfilling operation, in order to respect the pressure range foreseen at design level.

Just before starting the new-ring excavation, the grout must be pumped in order to avoid presence of void behind the shield.

The minimum and maximum pressure values, valid for each homogeneous tunnel stretch, will be set together with face-support pressure. In the case of backfilling injection, different values of pressure will be reported as functions of the nozzle position on the shield' contour. Pressure values are evaluated on the base of lithostatic load calculated at the considered depth, plus water pore pressure, if present.

If one of the limits is overcome during excavation mode, it is advisable to check the final quality of the backfilling grouting by the execution of a survey by drilling cores.

Once the excavation of the ring is completed, backfilling injection must continue until the pressure value for each nozzle's position is reached, eventually using the automatic control injection system, before starting a new cycle.

In order to protect the segment from excessive pressure, the pumping plant must be equipped with a safety valve, which stops injections automatically when the maximum value set for the pressure is exceeded. Improper pressure could be caused either by incorrect movements of the ring due to injection (mainly in the case of unbalanced pressures among the six injectors) or by insufficient backfilling.

3. Check that the *mortar properties conform to the design characteristics*. It would be appropriate to develop testing methods that will allow the performance of the grout to be assessed prior to use. Some useful tests to apply in practice are listed below:

   - Grading curves.
   - Cube test, to obtain final compressive strength of the hardened mixture.
   - Segregation.
   - Bleeding attitude under pressure.
   - Workability of the mixture.
   - Flotation (in order to measure the actual effectiveness of the grout, the change in level of several rings immediately behind the tailskin is measured before and after shoving).
   - Viscometer test, to assess and control general grout quality.

If any of the mentioned conditions are not maintained, it is convenient to perform a second or a third stage of injection, in order to manage correctly the development of surface settlements for long-term stability evaluation.

6

The control of tunnel construction

> "The correct choice of machine operated without the correct management and operating controls is as bad as choosing the wrong type of machine for the project"
> (BTS/ICE, 2005)

This section provides the details about the use of the Plan for Advance of Tunnel, PAT, starting from updating and implementation of the Protocol, discussing the use of PAT for excavation control in the cases of Slurry Shield and Earth Pressure Balance Shield, and concluding with the description of an integrated real-time monitoring system.

6.1 UPDATING AND IMPLEMENTATION OF PAT

6.1.1 Assessment of construction risks

As indicated in Section 2, the analysis and management of risks constitute a fundamental principle of the design and construction approaches, which is also the "leading theme" of this book. The "risk register", also defined in Section 2.3.1, is equally fundamental for the construction control phase and needs to be completed with the inclusion of "construction risks", prior to beginning the actual excavation activities.

Hereafter, the text will deal with the methods for identification of risks associated with the various types of TBM and, in particular, the relevant mitigation measures, which will allow tunnel excavation in urban areas with the awareness of having reduced the levels of the various risks to the acceptable residual values.

As a general framework, it should suffice to mention the main hazards foreseeable in the excavation of a tunnel within a city: a face collapse event, the creation of a "chimney" running to the surface, and the occurrence of surface subsidence capable of endangering the safety and the stability of the pre-existing surface structures along the tunnel route. The risks related to such hazards can endanger the whole project, since they could entail dramatic consequences for persons and properties, with catastrophic impacts on the social, financial, and temporal aspects.

The tunnel design ought to have already foreseen the necessary countermeasures to overcome such potential damages, by keeping them as low as reasonably possible (see Section 2.4). It is essential, in the construction phase, to perform the work by implementing the "theoretical" concepts expressed by the design.

If an adequate excavation method has been chosen and an adequate face pressure is applied (see Section 5.2), the surface settlements are reduced to below the acceptable values. If the volume of the extracted material is strictly kept under control, the face collapse phenomenon can be avoided, also in case of "instability". Should any of these controls not suffice, the ground would have to be consolidated or the structures subject to risk would need to be otherwise protected (see Subsection 5.1.8).

There are some key questions that need to be answered.

How is it possible to guarantee that the pressure will be maintained at "adequate" levels? How can it be guaranteed that the control of the extracted volumes and weights is performed regularly and precisely? How to monitor the trend of the parameters, how to interpret the monitoring data, and how to react when faced with an emergency situation? When and how should the decision be taken as to whether the TBM has to be stopped and remedial/consolidation measures have to be implemented from the surface?

There is just one answer to all these questions: risk analysis and management must be implemented in the construction phase, setting in place a series of "operating procedures" which constitute the mitigating interventions for construction risks.

If correct management and control procedures are available and are followed, even in a potential face-collapse situation, there is the possibility of reacting, applying the identified countermeasures, which were previously designed and are in place ready to be used, thus avoiding the worst scenario (Fig. 6.1).

The causes of any accident in tunnelling operations are complex, but certainly the lack of an adequate control of the TBM is a contributing factor. The complexity of the underground geology and hydrology could be, and generally is, one of the causes of potential face collapse. However, for moving from "potential" to "actual" collapse, it

Figure 6.1 Some pictures of collapse consequences.

is necessary to recognise that someone forgot to do a proper control of the excavation process. Often, the insufficient control of the extracted material, together with an over-excavation above some rings, is sufficient to cause a chimney to extend up to the surface; but the over-excavation itself, if accompanied by a rigorous control of the extracted volume, is not sufficient to cause major damages.

The technical means now available, particularly the accessibility to all the excavation and monitoring parameters and their recording (exactly like the "flight recorder" of an airplane), supply the data that are essential for interpreting, consciously and effectively, "what is happening". The aim is that the analysis, interpretation and management of the operational data, together with the monitoring readings, make it possible to implement the countermeasures in the shortest possible time, in order to avoid "the plane crash" as a consequence of the accident, and not to use the "black box" just to investigate the causes of the crash!

6.1.2 PAT update

Section 2.6 defines the concepts and the implementation mode of the Plan for Advance of Tunnel, PAT, which is the acronym for the particular design/construct-control procedure first used in the Porto Metro. So, in Portuguese it was given the name "Plano de Advance da Tuneladora".

The concept of PAT is a blend of the principles of the "probabilistic design" and the "flexible design". In practice, the PAT represents the perfect spiral: Design – Construction – Design Adjustment – Design Re-elaboration – Construction follow-up – Design Re-adjustment (Fig. 6.2).

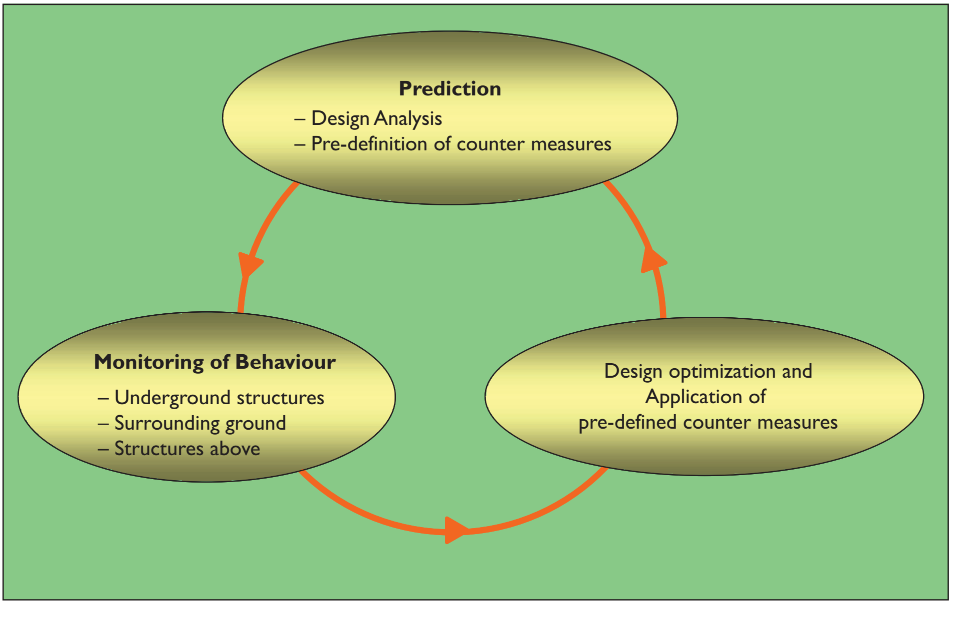


Figure 6.2 Iterative Design Schema.

PAT constitutes a dynamic tool for interrelating design and construction of an underground work, with particular reference to mechanised tunnelling in urban environment. PAT has been iteratively improved by the Authors by using experience gained from its application to various projects.

The practical and prime purpose of PAT is to validate, and eventually update, the main parameters that allow a full control of the excavation and that were defined during the design stage. Collection and analysis of the data obtained during TBM excavation allows evaluation of correctness of the design forecast (with specific reference to the key parameters) as confronted by varying geological, geotechnical, and hydrological conditions actually met by the excavation. These data are compared on-line, in real time, with the readings of the surface monitoring instrumentation.

For all works performed in an urban environment, priority is given to the necessity to limit the subsidence caused by the excavation (due both to the face plasticization and the annular void between the excavation and pre-cast lining) and to make them compatible with the pre-existing structures on the surface.

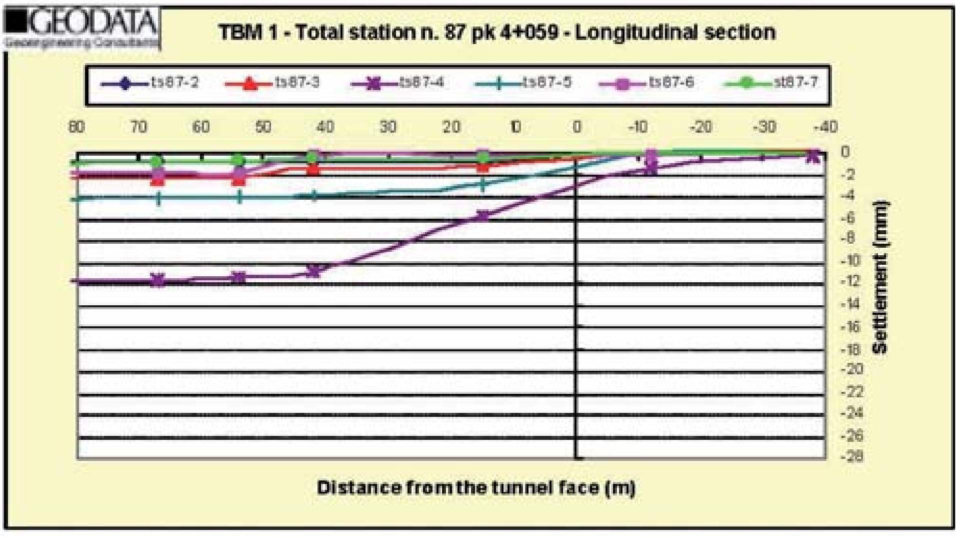
During excavation, the degree of effectiveness of the face support and the lining back-fill is provided by the subsidence response, which has been continuously monitored and interpreted by using adequately instrumented stations (Fig. 6.3).

In particular, it can be stated that:

- The correct evaluation and implementation of the face pressure is prevalently reflected by that portion of the subsidence, the so-called "*pre-subsidence*", which is the settlement measured before the face arrives underneath the monitoring station.
- The balance of subsidence (until final stabilisation) depends on the correct backfill of the annular void around the lining and, therefore, has to be controlled through pressure and volume of the grout. It no longer depends on the face pressure. It should also be noted that in practice, there is also a portion of subsidence defined as "physiological", connected with the usual truncated-cone geometry of the TBM, which can cause above-the-shield ground settlements (depending on the fragile or plastic short-term behaviour of the ground, besides the excavation advance-rate). Strictly speaking, this percentage of "physiological volume loss" is still connected to the pre-subsidence phenomenon. A large value of pre-subsidence will most likely mean that the ground has been largely disturbed, so that it will tend to settle on the shield to an extent which is greater than the one associated with a non-disturbed ground.

During the detailed design, the face pressures are calculated using numerical modelling and analytical methods (see Section 5.2). The experience previously gained from similar works is also used for developing the design concept, for selecting the construction method, and for analising and controlling the construction.

On the basis of the geotechnical model elaborated in the initial design phases (base model), the tunnel route is divided in 'homogeneous' zones, i.e. lengths in which the same face pressure is to be applied. The subdivision is normally carried out on the basis of the foreseeable geotechnical characteristics of the ground



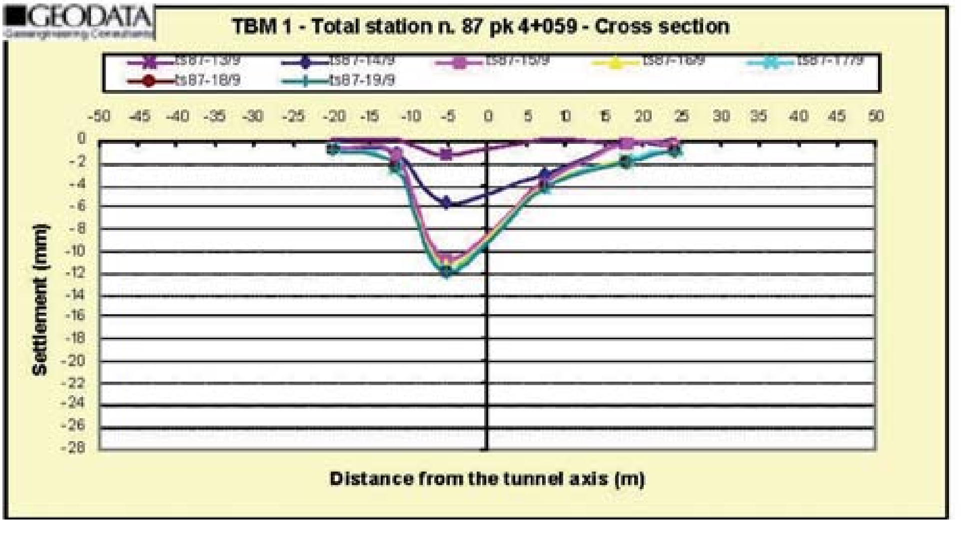


Figure 6.3 'Nodo di Bologna' Project: settlement recorded by station No. 87.

to be excavated as well as the conditions of the surrounding environment along the route (cover, water table presence, surface or deeper loads, and presence of interfering structures).

Afterwards, as the excavation progresses, starting from the "learning-curve" zone, the base model is regularly verified and/or refined on the basis of the data obtained during the work and, particularly, focusing on the key parameters that control the excavation.

This work (or process) is what constitutes the PAT, which is performed in advance of the excavation of the subsequent zone. The Plan deals with sections of the tunnel, whose length (preferably constant – say about 300 m) is defined a priori as a function of the complexity and variability of the conditions to be faced as well as of the foreseen progress rate of the TBM.

The presence of specific structures along the route can, in some cases, extend or reduce the length of the PAT zone.

6.1.3 PAT implementation

PAT, besides being a design and construction control method, is characterised by a *physical* "hard-copy" document, generally a technical report, describing the conditions which led to the modifications as well as the modifications themselves, accompanied (usually) by a synthesis sheet representing the geo-mechanical *profile* of the related zone, with all the information necessary for the synthesis.

Joining together each single *profile* with the adjacent ones provides, at the end of the works, the "as built" profile of each tunnel, with the history of the whole construction.

PAT is usually prepared by the Designer/Consultant responsible for follow-up of the works, interacting with the Contractor to define and select the agreed countermeasures. It is then discussed with, and subsequently approved by, the Engineer during the periodical technical meetings held to simplify and expedite the contractually-required (if any) approvals, prior to implementing the agreed actions. In particular cases (for example, Section 8.3, Porto Metro), the technical meetings may be held on daily basis.

6.2 EXCAVATION CONTROL: THE CASE OF SLURRY/HYDROSHIELD (SS/HS)

The main technical aspects characterising the tunnel excavation performed with Slurry Shield or Hydro-shield technology are described here, together with the control and management procedures to be used for a correct work performance.

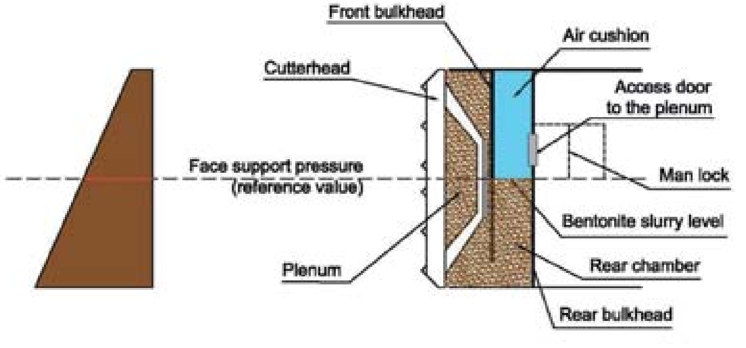


Figure 6.4 Schematic representation of a Hydroshield.

6.2.1 Monitoring and control

The Parameters to be monitored are:

1. Face-support pressure (the reference value of which is indicated in Fig. 6.4): the control will proceed by means of compressed-air pressure.
2. Quantity of mucked, solid materials: it will be determined as a function of the difference between the outlet and inlet flows, density measures, and slurry level in the chamber.
3. Slurry characteristics: density, yield value, viscosity, and quality/cake-thickness of the filtered material (checked at the treatment unit laboratory).
4. Segments mortar grouting: it deals with control on volumes and pressures recorded during the backfilling around the segments and automatically checked by sensors connected to injection pumps.

6.2.1.1 The control of air pressure in the plenum

Compressed air in the chamber performs several functions. It regulates the pressure transmitted by the slurry to the ground at the face, thus controlling the face stability

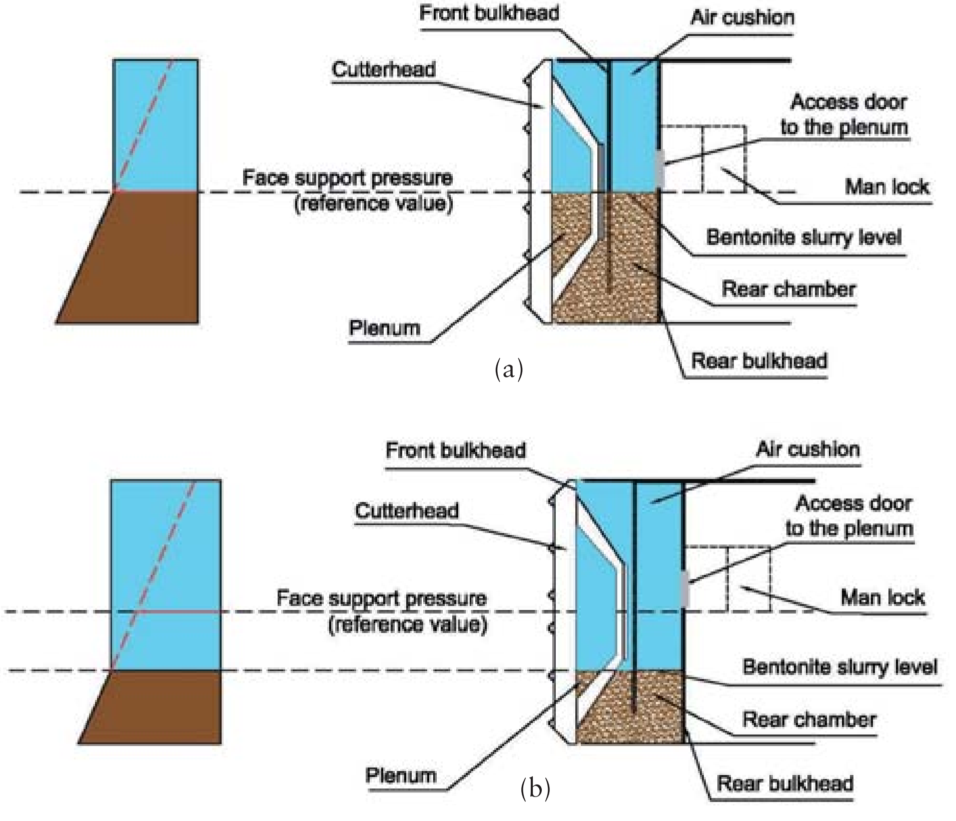


Figure 6.5 Compressed-air face-support pressure; (a) with bentonite slurry level in the middle, and (b) with the bentonite slurry level at the bottom of the plenum.

and, at least partially, the surface subsidence. On the other hand, it is an essential tool for the maintenance operations to be carried out under pressure in the chamber, because, then, it is the only means to apply pressure at the face, besides supplying air for breathing to the maintenance crew.

A comparison of Figures 6.5 a) and b) shows that fixing a pressure value of the compressed air signifies the fixing of the "average" pressure that the slurry can exert on the ground at the face.

During the stoppages programmed for control and maintenance of the head (e.g. replacement of excavation cutters), it is necessary that the crew enters the chamber that is under pressure, which cannot be lowered because of the potential risk of face instability or increased surface subsidence.

Depending on the type of maintenance required, the chamber can be emptied down to various levels and in any case, lowering the slurry to the level of the transfer door of the hyperbaric chamber (see Fig. 6.6). Unless it becomes necessary to intervene in the lowest part of the chamber (the slurry sucking area), it is not necessary to empty the chamber completely and it is recommended not to do so, unless it is absolutely indispensable.

A reference pressure is fixed by assigning a threshold pressure to the automatic system regulating the compressed air. The reference pressure can be regulated within very precise limits (it can be fine-tuned to 5 kPa or even less). When the pressure increases beyond the threshold pressure, the system opens the discharge valves and lets the air out, until the required pressure is re-established. When the pressure gets lower, the valves on the main supply circuit from the compressors open and the pressure is quickly returned to the established level. The system has to be well balanced to avoid the fluctuations that are too big or too long, that could lead to instability. This control system through the compressed air partly substitutes the control through the input pumps on the slurry lines and the output pumps on the muck lines, which are used in the Japanese "slurry machines". However, as shown in Fig. 6.7, the control of the inflow and outflow given by the pumps is extremely important. In fact, if the balancing of the pressures is obtained when the level of the bentonite slurry is different from the reference level, then the situation needs to be investigated further.

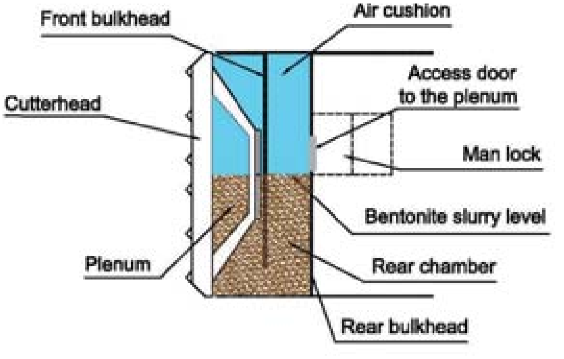
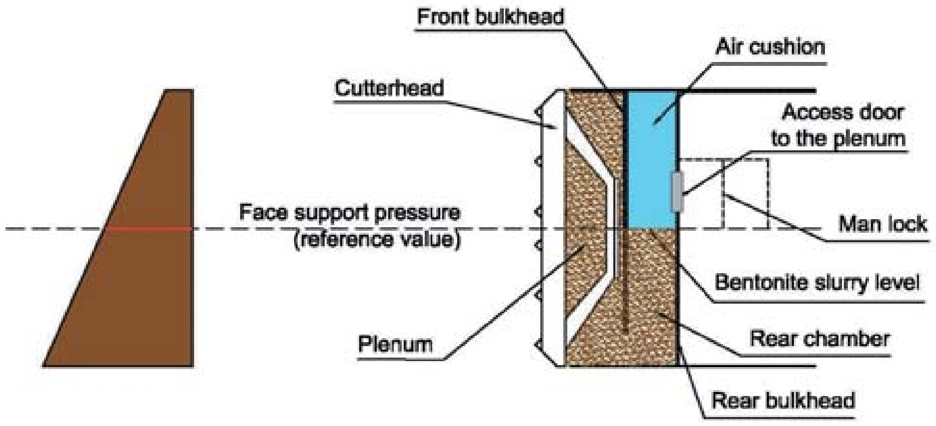
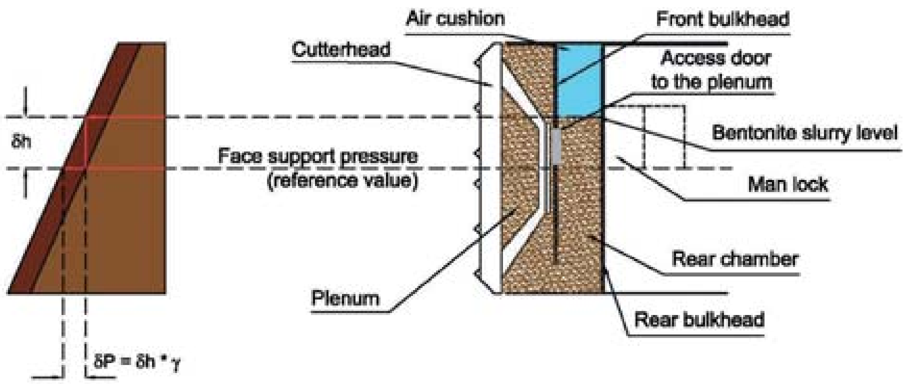


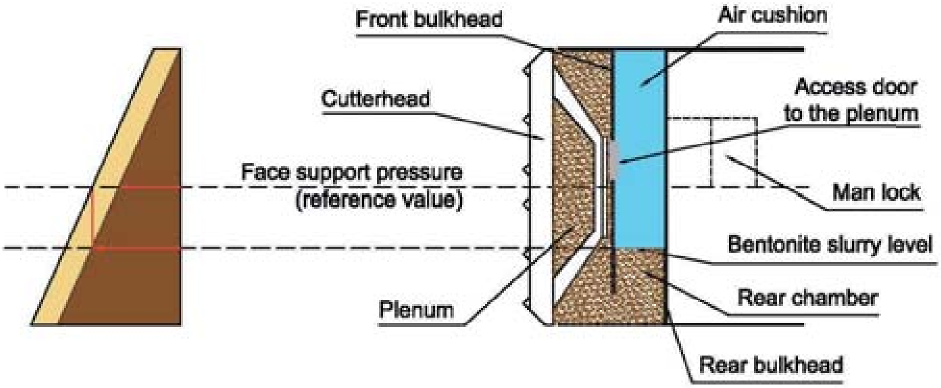
Figure 6.6 Entrance into the plenum through the man lock.



(a) Normal level: face-support pressure = air pressure



(b) High level: face-support pressure = air pressure + δP



(c) low level: Face-support pressure = air pressure – δP

Figure 6.7 Control of the slurry level in the plenum.

6.2.1.2 The control of the slurry level

With reference to Figure 6.7 (a, b, and c), the bentonite slurry level in the rear chamber is defined as the distance of the interface slurry-air from the bottom of the excavation chamber. The reference pressure (i.e. the compressed air pressure in the chamber) corresponds to the pressure that the slurry exerts on the face when at this level. Should the level vary, the air-regulating system comes into action for maintaining the slurry-air-system pressure at the reference level. If the level rises, the volume available for air diminishes and, thus, the pressure increases: the regulating valves will open to re-conduct the pressure to the equilibrium value. In this manner the regulating system keeps the chamber-air pressure constant. However, what happens to the pressure that the slurry exerts on the face? Figure 6.7b shows that the pressure exerted by the slurry on the face is elevated, by $\delta P = \delta h \times \gamma$, where δh is the increase in the slurry level and γ is the slurry density. On the contrary, if the slurry level drops, the system will maintain the air pressure constant by calling for the necessary quantity, but the pressure on the face will diminish, with the magnitude indicated (again) by the product of the level-decrease and slurry density.

Therefore, the primary necessity is to concurrently control both the air pressure and the bentonite slurry level in the plenum. The required adjustments are made by using the input and/or output pumps of the slurry and mucking circuits, respectively, which can operate in automated, semi-automated, or manual modes. The following precautionary principles should be followed when using the pumps:

i. Operating on the output pump only, can have dangerous consequences, when the action is required to react to an *increase* of the bentonite slurry level. Increase of the output volume, to re-adjust the level to the reference mark, increases the quantity of material being pumped-out. If one reflects that a possible (and dangerous) cause of the level increase could be a face collapse, with consequent tendency for the material to invade the chamber, an increase of pumping-out would only *favour* a dangerous event.
ii. The same effect (i.e. lowering the bentonite slurry level) could also be achieved by reducing the rate of "pumping-in". However, the time required for adjusting the pumped-in quantity is considerably longer than the response time for the outflow. The reason is that, while the 'out' pump is located on the TBM back-up and the reaction can be nearly immediate, the 'in' pump for slurry supply could be located a few km away from the face, with a reaction time necessarily much longer (Fig. 6.8).

The variation of the bentonite slurry level also has a negative influence on the system stability. It should be noted that the "plenum-cutterhead" system is not *static*. The head turns slowly (typically in urban environment and in soft ground at about 1 rpm), and can also reverse its sense of rotation. In several instances it *must* reverse to reduce the "rolling" effect, with consequent dynamic influence on the slurry level. Also, the slurry is pumped into the chamber and extracted from it together with the muck, with flow quantity variation, which causes turbulence in the slurry. All of the above events induce oscillations in the slurry level (some can be substantial) which, in turn, can induce oscillations in the face pressure of the order of + or –25 kPa. (see Fig. 6.9, showing parameters from the St. Petersburg job site, discussed in Section 8.2).

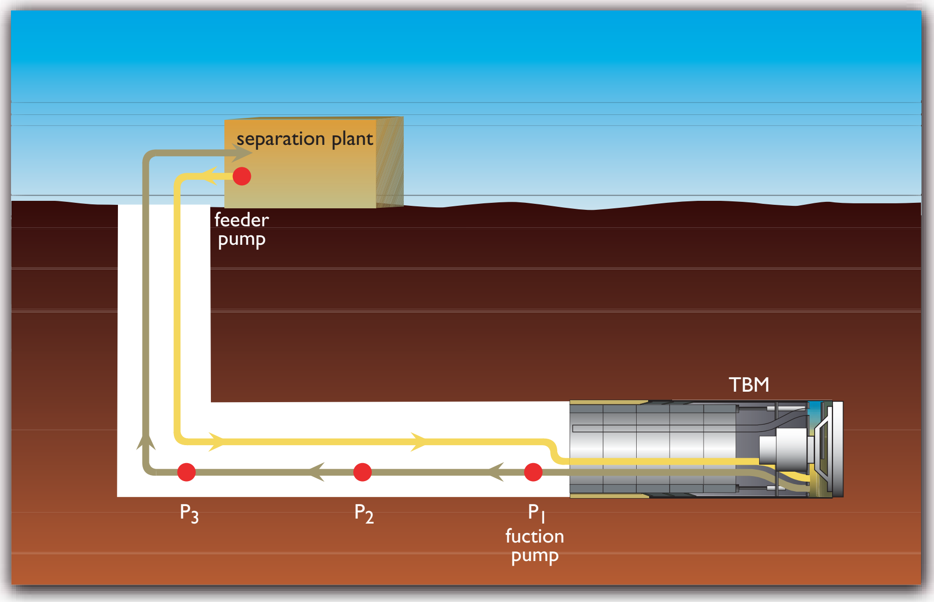


Figure 6.8 Slurry circuit in a Hydroshield System, showing the feeding and suction pumps and circuit.

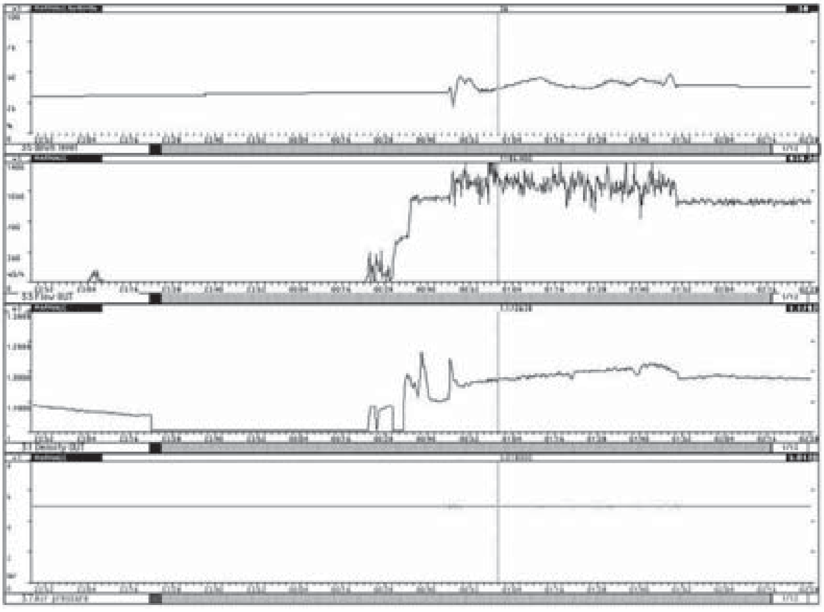


Figure 6.9 Excavation parameters of the Slurry Shield used in St. Petersburg, where: a) Bentonite slurry level, b) Flow out, c) Density out, d) Air pressure.

All of this has to be taken into account in determining the reference values and the allowed range of operation. (A point worthy of note is that these oscillations nullify in practice the efforts made to achieve tighter air pressure controls, often lower than 5 kPa, which are touted as a special advantage of this system as compared with others).

6.2.1.3 The intervention at the face under pressure – the hyperbaric chamber and its use

The use of "closed" machines with the face under pressure creates the problem of how to intervene for maintenance and repair of those parts that are accessible only through the plenum, which is, and has to remain, under pressure to ensure the correct face support and the surface subsidence control.

The necessity of intervention can be also very frequent: a case in point is the control and replacement of the excavation cutters, whether soft ground tools or rock discs. In case of ground with very abrasive materials, it could become necessary to intervene several times each week. A typical example is the Porto Metro, passing through variably altered and degraded granite, where the change of some discs and picks was at a *daily* frequency. The fact that it was actually an EPB Shield did not lighten the problem.

It follows that, in the plenum, the bentonite slurry has to be replaced by compressed air, which takes over the task of transferring the pressure to the face with the help of the filter cake previously established. The chamber will then be partially or totally emptied depending on the required type of intervention, by pumping-in more compressed air, adding to what is already present in the rear portion of the plenum.

Figure 6.10 shows the required value of the air pressure in the upper part of the rear compartment of the plenum, which will assure safe working conditions. The pressure value measured with the crown sensors must be higher than the pressure of the slurry at the same sensor position because the air pressure is not hydrostatic, but it is constant in the upper part of the rear compartment. In summary, the required air

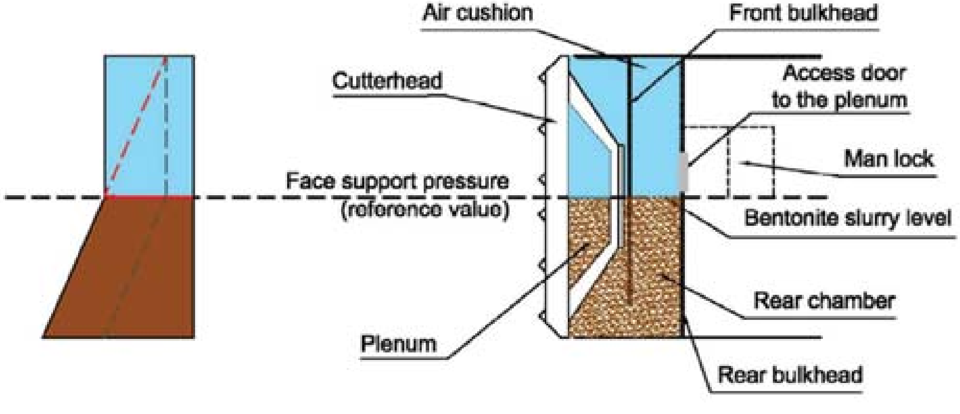


Figure 6.10 The face-support pressure when the chamber is partially empty.

pressure should be equal to, or higher than, the slurry pressure at the bentonite slurry level in the front compartment.

Workers' access is made possible by the presence of a "hyperbaric chamber" or "man lock", an air-tight double-bodied cylinder with air-tight bulkhead doors, permitting the workers' entry and the ambient pressurisation in the time and manner prescribed by law and the safety norms (including the compression and de-compression times). The man lock has to be double-bodied, to allow rescue teams to reach an injured worker at any time and phase of the operations (see Fig. 6.11). Usually there is also a second chamber, single-bodied, for transfer of materials.

For very big machines, as it has become usual during the last few years, all these facilities can be doubled (two chambers for workers and two for materials).

In the last few years, hyperbaric interventions at 6 bars have occurred (e.g. in Elbe tunnel and Westerschelde tunnel) for which recourse had to be made to true specialists, deep-sea divers. To avoid too frequent compression and decompression cycles, these workers lived in hyperbaric conditions in weekly shifts, and were available to be moved into the tunnel by a pressurised "shuttle" and from the shuttle to the pressurised chamber, without having to go through procedures (on each occasion) whose times could be totally incompatible with the tunnel advance.

6.2.1.4 The control of the quantity of excavation material

Excavating with machines which do not allow any visual inspection of the face (except in particular instances, during maintenance), has as a consequence one of the worst dangers is to provoke unwanted *over-excavation* of unknown dimensions. An example

Figure 6.11 Man lock installed on a Herrenknecht machine.

scenario could be an unstable or even collapsed ground (e.g. due to insufficient face pressure or due to the presence of loose lenses of incoherent material, below the water table) with a material ingress in the plenum, of a quantity higher than the theoretical (= excavation section × progress rate). To maintain equilibrium, the system will call for higher outtake, thus facilitating if not even increasing the over-excavation phenomenon, which could quickly lead to creation of a "chimney" to the surface.

It is, therefore, essential to control the quantity of the extracted material and confront it with the theoretical value, to be able to intervene *during* the excavation to avoid such occurrences. As this is a hydraulic system, the tools at the disposal of the TBM operator are the measurement of the in-flow and out-flow quantities, to be able to know the mix quantity taken out in excess, which will correspond to the quantity of the over-excavated material mixed with water. To calculate the amount of the dry quantity, the densities of the inflow and outflow material need to be known.

Thus, it is necessary to install in the relevant pipelines two measuring systems for density and flows to be elaborated by further calculations (Eq. 6.1).

$$Q_{ba} + Q_{ex} = Q_{be} + Q_f \tag{6.1}$$

Where (see Fig. 6.12)

Q_{ba} = slurry inflow; γ_{ba} = inflow density
Q_{be} = extraction outflow; γ_{be} = outflow density
Q_{ex} = excavated material inflow;
Q_f = slurry-loss outflow;
γ_w = water density

This method, first used on the Hydroshield Voest Alpine used in Naples for line LTR was later utilized, after further studies and tests, on the EOLE project in Paris (see Section 8.1), and it is now commonly installed on the modern slurry machines.

The quantity of dry material extracted, (P_{ms}) according to Bochon, Rescamp *et al.* 1997, is calculated with the formula (Eq. 6.2):

$$P_{ms} = \Sigma_t\,[(\gamma_{be} - \gamma_w)/(1 - \gamma_w/\gamma_s) \cdot Q_{be}] - \Sigma_t\,[(\gamma_{ba} - \gamma_w)/(1 - \gamma_w/\gamma_s) \cdot Q_{ba}] \tag{6.2}$$

Or, after a few passages, as a function of extraction outflow (Eq. 6.3):

$$P_{ms} = \Sigma_t\,[(\gamma_s \cdot (\gamma_{ba} - \gamma_{be})/(\gamma_{ba} - \gamma_s) \cdot Q_{be}] \tag{6.3}$$

and constitutes a fundamental tool for controlling the general stability of the excavation.

Actually, the phenomenon is much more complex and the calculation needs to be corrected each time and for each situation, taking into account the existing water table (to include the water quantity in the in-situ material) and possible *losses* of slurry through high permeability zones or through fractures in the ground.

Fig. 6.12 and the related formula below show the equilibrium of the flows of slurry and muck in the plenum. The quantity of slurry lost through the ground can be calculated as

$$Q_f = Q_{ba} + Q_{ex} - Q_{be} \tag{6.4}$$

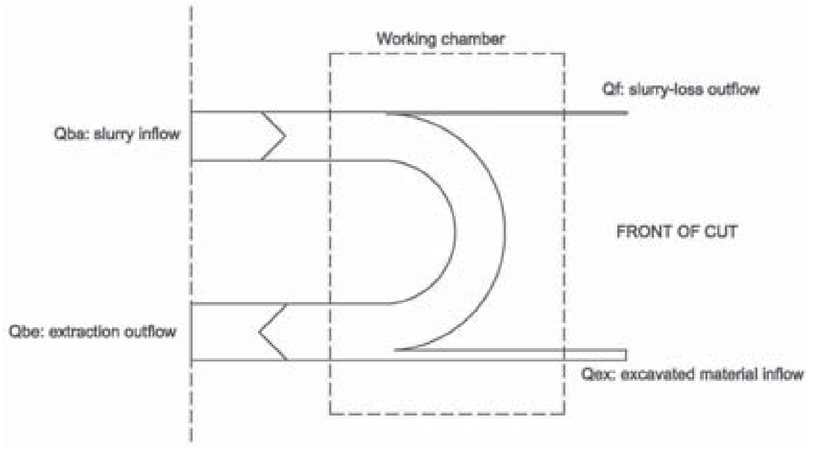


Figure 6.12 Slurry flows equilibrium into the plenum (after Bochon and Rescamps, 1997).

where Q_{ba} and Q_{be} are measured on the flowmeters (Eq. 6.4), and Q_{ex} is the theoretical excavated material. For an effective control and management of the process, the comparison between the actual and theoretical results needs to be done several times during the excavation and the operator needs to intervene to correct the trend of the parameter (see Section 6.3 for similar control of the extracted quantity also in the case of EPB excavation).

In controlling the excavation parameters and the monitoring data, it is more useful and effective to keep under control the "trends" of the data over time or in relation to the excavation length, more than just the absolute single values. This observation is particularly important for the "quantity of excavated material". If the "excavated dry weight", according to the formula of Bochon and Rescamp (1997), is graphed in relation to excavated length, the trend of such a graph or curve gives precious indications. Figure 6.13 shows a significant change of inclination of the

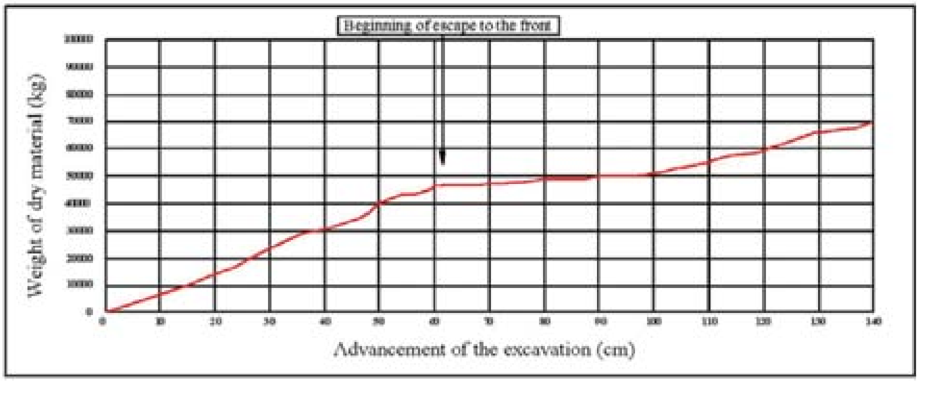


Figure 6.13 Graphical representation of recorded data of weight of excavated material.

curve, which can signify two equally dangerous things: (a) the beginning of a *loss* of slurry out of the face into the surrounding ground, with consequent loss of extracted material, or (b) the start of a blockage on the out-pumping system due, for instance, to boulders at the aspiration point, or the formation of blockages along the first stretch of the pipeline.

Conversely, a sudden increase of slope of the curve, which is less frequent but even more worrying, would signify a sudden unexpected "ingress" of solid material in the chamber, i.e. the possible beginning of face instability with the danger of collapse, or at least the beginning of an over-excavation.

6.2.1.5 Cross-control of principal excavation parameters

Figure 6.14 shows some excavation-parameter values measured on the Hydroshield (of Voest Alpine) which was utilised on the St. Petersburg Project.

In the upper part:

- Front pressure = slurry pressure measured in the plenum
- Air pressure = air pressure measured in the back chamber (reference value)
- Dry material = calculated value of cumulative dry material quantity, extracted by the pumping system
- Leaking flow = calculated value of potential loss of bentonite slurry (in the example shown by the figure the algoritm was probably in error, so the result is wrong)

Using the above parameters, it is possible to control the face-support pressure.

In the lower part:

- Flow in = quantity of slurry pumped into the plenum
- Density in = density of slurry pumped in
- Flow out = quantity of mixture pumped out from the plenum (slurry plus muck)
- Density out = density of the mixture pumped out (including muck)

Using these parameters, it is possible to calculate the Dry Material quantity.

As pointed out in the preceding sections, the control and management of the excavation process is complex. The following scenario will illustrate the complexity of the process. When working in urban areas, the specifications require that the "volume loss" (which causes the surface subsidence – see Section 5.1) should not exceed the range of 0.5 to 0.7%. However, the precision of the available instruments to measure the fundamental parameters is much lower. The parameter selected for this scenario is directly connected with the concept of volume loss, such as the measurement of the extracted material discussed above. The measurement precision of this parameter, which is derived from processing the results of four measurements, is influenced by the precision of the source. Moreover, the precision is affected by the possible system errors, such as the presence of water table and/or losses toward the exterior. Under these circumstances, the measurement precision of volume loss could hardly reach 5%. Therefore, it is not possible to measure and/or guarantee a volume loss <0.5% with instruments whose precision is lower by one order of magnitude. Another example of difficulty in realizing precision measurements is

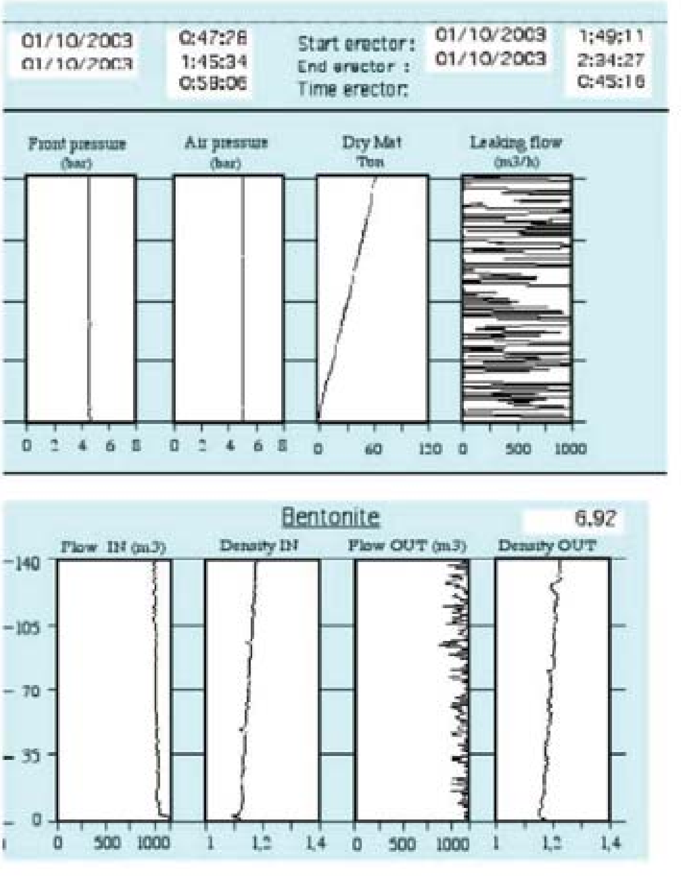


Figure 6.14 Parameters measured in S. Petersburg job site, during excavation with Hydroshield Voest Alpine.

the requirement to measure air pressure with a precision of 2 kPa when the slurry level oscillation (nearly uncontrollable) causes pressure variations of the order of 20 kPa.

A fundamental objective of this book is to demonstrate, with the help of numerous practical examples (from the cited case histories) that through the cross-control of the parameters essential for "control and management" of the procedure, with a correct and expert interpretation of their *values* and especially their *trends*, the desired precision can be obtained and the process can be managed with efficient tools and methods, of which the PAT is the final synthesis.

6.2.2 Typical procedures for managing the excavation process

The following are the typical procedures that will be applied in case the threshold figures for the parameters under control are exceeded. It is assumed that for all equipment related to the control of the parameters in Table 6.1, adequate amount of spare parts, or even interchange spare equipment, shall be available on site throughout the excavation; all of them shall be checked before the start of excavation. In addition, the personnel assigned to using, monitoring, and maintaining the equipment shall be correctly and continuously trained.

The pressure in the chamber decreases This implies a loss of bentonite slurry. The regulating system increases inflows of compressed air. The level of bentonite has to be corrected to the previous value, operating by means of the feeding pump and/or the return pump. If these actions are not able to resolve the problem (i.e. reinstall the correct pressure), then *alarm* conditions are present.

The pressure in the chamber suddenly increases This means that there is a possible inflow of excavated ground into the plenum. The system reacts by discharging compressed air. The bentonite level must be immediately recovered and kept under control. The volume of mucked material shall be checked: it must be the same or less than the theoretical value. In case this phenomenon continues, *alarm* procedures have to be applied. Actually, high pressure could damage both the tail sealing system (with the future, uncontrolled losses of pressure) and the bearing seals, with mechanical risks, especially if the machine is working at its pressure limits. High pressure could also be dangerous for the face stability, due to the possibility of increasing the pore pressure in the ground.

The slurry level in the chamber decreases Loss of slurry – and consequently loss of the relevant pressure – could be caused by ground fissures, ancient wells, open piezometers, and unsealed boreholes. In this case, the slurry level could drop below the bottom of the bulkhead separating the front from the rear part of the chamber, and the air flow into the upper front section, just against the excavation face and the support of slurry cake, could be eliminated. This condition could be manageable when the compressed air pressure is or is kept constant (no escapes) due to the low permeability of the encountered soils. Excavation can go ahead by taking special precautions: reducing muck flows and feeding the chamber with denser bentonite slurries. On the other hand, when sandy soils or soils with higher permeability are encountered, air escape can start and the face becomes unstable; therefore, *alarm* will apply.

Additional air is pushed into the chamber by the automatic compressed-air regulating system. The slurry level must be increased to the correct figure; otherwise, the

Table 6.1 Parameters under control and relevant control systems

Pressure at the face	
Compressed air pressure	TBM automatic system
Slurry level in the chamber	Automatic pumps system for feeding and extraction
Quality of the slurry	
Viscosity, Yield value, density, cake thickness	Site laboratory
Quantity and quality of mucked material	Double measurement system (density and flow) as well observations at the treatment plant
Segment mortar grouting	Injection pumps, manometers and automatic control system

confinement pressure will give an insufficient value (see Fig. 6.10). Therefore, the corrective measures shall be implemented: increasing the inflow through the feeding pump and, if necessary, decreasing the flow of the return pump. Again, if the slurry level is not recovered, *alarm* conditions will arise.

The slurry level in the chamber increases The automatic regulating system discharges air. The slurry level must return to the correct value, otherwise the pressure will increase (see the second procedure above and eventual relevant *alarm* conditions).

The extracted soil quantities are smaller than the theoretical values It is necessary to check whether less dense in-situ soil is encountered or obstructions (like boulders) appear in the chamber outlet or along the return circuit. In the latter case, the obstruction must be localised and the relevant pipeline section cleaned (and the TBM temporarily stopped). Otherwise, high-pressure water jets, installed close to the chamber outlet, could be used for cleaning the obstruction. In some cases, the slurry flow could be inverted as well, using the circuit by-pass. But the crusher (which may or may not be inside the chamber) could be an obstacle to this jetting operation and, furthermore, specific combination of the size of boulders and the soil consistency could make this measure inefficient. Therefore, access to the chamber could be required. The most adequate control consists in verifying the successive mortar grouting parameters in the affected area.

The extracted soil quantities are higher than the theoretical values A cross-check between the pressure and the slurry levels in the chamber is mandatory to verify the face stability. In fact, such an event could provide a warning about the existence of a chimney. It could even create or increase the effect of chimney. Advance speed must be immediately reduced and the return pump-flow reduced, eventually increasing the feeding flow as well. If this condition persists, *alarm* procedures must be applied.

Loss of bentonite slurry through the tail brushes Bentonite could escape into the tunnel under high pressure: this flow must be immediately interrupted and the grout should be applied around the segment in order to balance the bentonite pressure close to the shield brushes (Fig. 6.15). In general, grout pressure should be kept higher than the slurry pressure and TBM-advance speed controlled in relation with injected grout quantities, so that an accurate backfilling may be guaranteed during the TBM advance.

ALARM

As indicated above, alarm conditions are always recognized when the pressure in the chamber, the slurry level, or the quantities of mucked soil exceed the predefined threshold values and escape from the operator's control. As a maximum hazard, a collapse at the face could happen, creating a "cavity" that will be filled by the bentonite present in the chamber, replacing the collapsed material.

The countermeasures to avoid the dangerous potential migration of the cavity to the surface are related to actions carried out from the surface or from inside the tunnel itself. Generally speaking, these phenomena are slow enough to materialize. Therefore, their development could be arrested before they turn into disasters, provided that they are detected very soon.

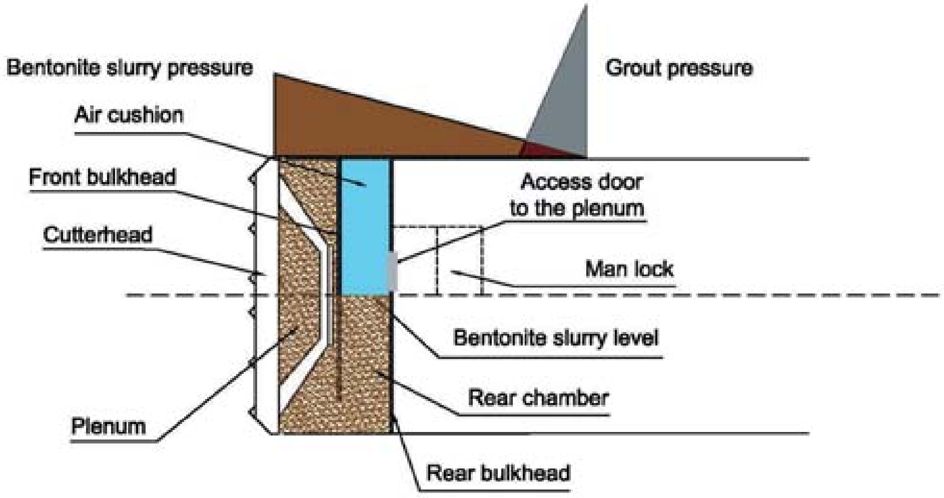


Figure 6.15 Comparison of grout pressure in the tail void and bentonite slurry pressure at the face.

When it is not possible, or is too difficult, to perform the mitigation measures from the surface (i.e. improvement of ground conditions through soil treatment), the only possibility consists in filling the cavity by injection of expansible materials (such as polyurethane and resins) from the tunnel. The correct material characteristics must be investigated, provided that the injections shall be performed under pressure. In addition to the above, the actions suggested in Subsection 5.1.8 should be carried out.

A typical example of the application of "Risk Management Plan" in a job using a Slurry Shield is provided in Appendix 5.

6.3 EXCAVATION CONTROL: THE CASE OF EARTH PRESSURE BALANCE SHIELD (EPBS)

6.3.1 Functional principles

The excavation method called EPB Shield is based on the principle that face support is provided by the (appropriately conditioned) excavated muck itself. A bulkhead separates the tunnel from the front part of the shield, where the cutter-head operates, thus creating the so called "excavation chamber" or "plenum".

The principle consists in creating an "accumulation" of material in the plenum by controlling the extraction and measuring the resulting "earth pressure", thus ensuring that it is kept at the level required by the stability calculations (see Figs 6.16 and 6.17). The muck is extracted from the chamber by using a screw conveyor, which is the tool for controlling and regulating the quantity of extracted material. The muck conditioning is performed by injections at the face, in front of the cutter head. The injected materials are usually foam agents, bentonite, and/or polymers. The purpose is to create a kind of "dough", which is as homogeneous as possible,

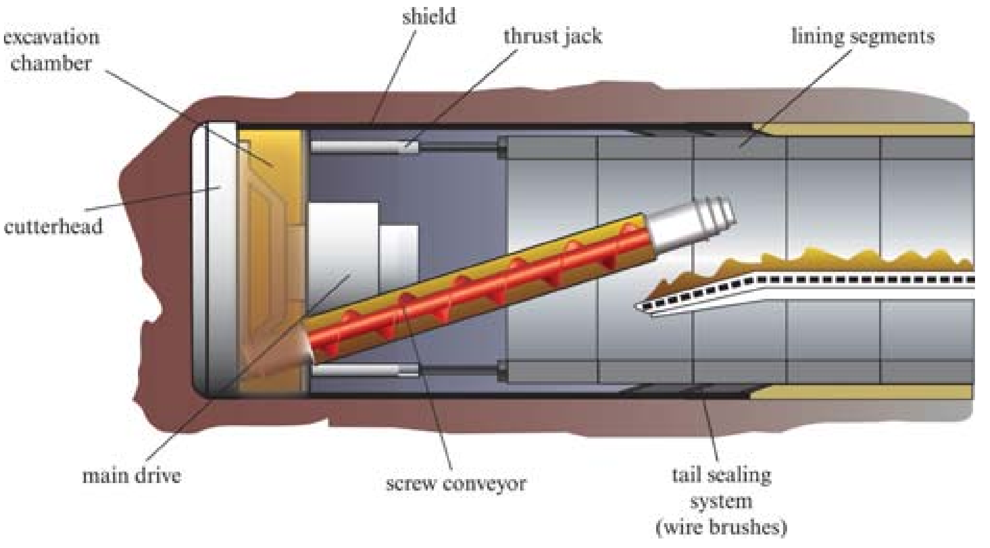


Figure 6.16 Conceptual scheme of an EPBS with principal components intervening during the excavation.

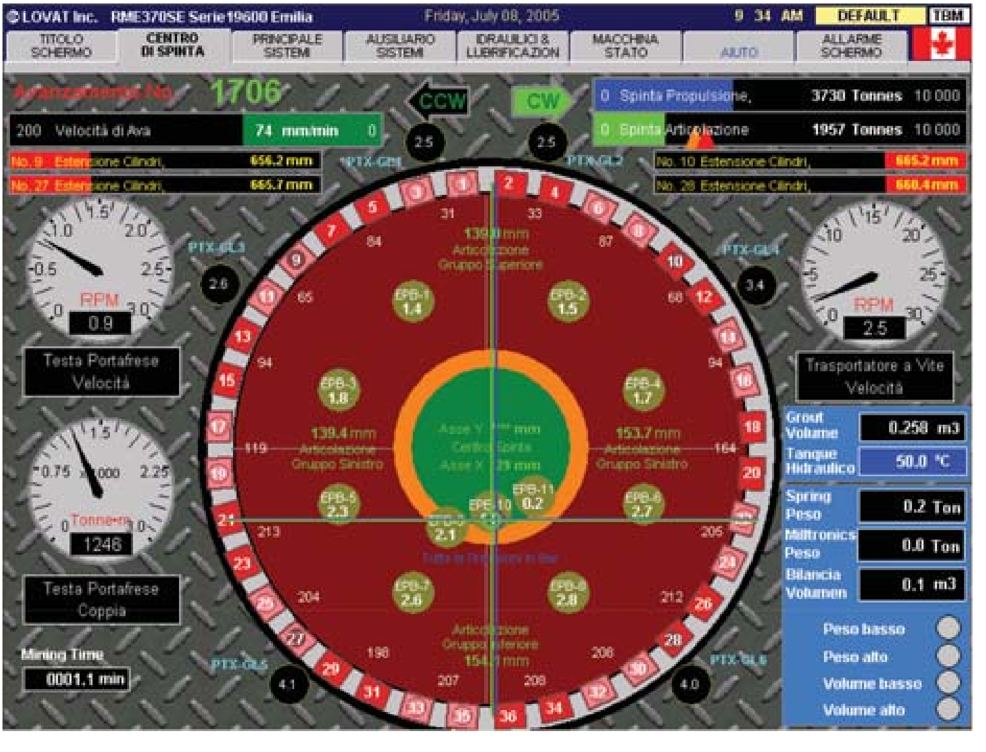


Figure 6.17 Screen display of the control parameters during excavation by EPBS.

to be able to manage the operating pressure inside the plenum and to better utilize the screw conveyor during extraction. A pre-cast ring is erected in the rear section of the shield, and the contact-grout backfill of the "annulus" between the ground and lining ring (the longitudinal grouting) is carried out (see Section 5.4 for details).

As further detailed later, the excavation process is influenced by different aspects connected with both the environmental conditions and the design, giving due consideration to the peculiarities of the excavation system.

Appendix 6 provides the typical procedures for excavating with an EPB Shield, based on the example of the Nodo di Bologna project (see Section 8.6).

6.3.2 Managing the excavation process

6.3.2.1 Excavation control in the learning phase

Before starting excavation, and during the initial phase of the "learning curve", the testing of the whole system is very important, in particular:

- It is necessary to perform general tests before the EPBS is started, as regards mechanical, electrical, hydraulic and safety systems (according to the specific procedures prepared during the design phase).
- In the initial phase of excavation, it is important to plan the progress in the first "experimental" zone of the tunnel (called the "learning curve", typically of a length between 50 and 200 m), which should allow the EPBS team members to fine-tune the following aspects:
  - correct method of ground conditioning in relation to the various additives and the local variability of the ground;
  - type of interaction and in-situ ground response to the EPBS excavation, with emphasis on the cutters' arrangement (adequacy of the cutters to the type of ground, net excavation times, penetration rate, torque necessary to excavate, thrust used for advancing), and
  - fine-tuning of the control systems of the principal excavation parameters (pressure sensors in the plenum, conveyor belt scales, volume measuring devices, backfill-grout-pressures meters, meters for ground- and plenum-injected water).

6.3.2.2 Excavation control in the regular excavation phase

Control of the excavation activity in EPB mode, in relation to ground stability, consists of the analysis and control of the main parameters connected with the excavation progress: face support pressure, weight (and volume) of the extracted material, apparent density of the material in the plenum, and the volume and pressure of the backfill grout behind the lining. These parameters are related to the parameters defined first in the basic design and successively updated/adjusted during the PAT preparation. The following subsections provide details of the use of these parameters in excavation control.

6.3.2.2.1 Face support pressure

This parameter is used to ensure that face stability is maintained during the EPBS advance and during the stoppages. It is applied, within the excavation chamber (plenum), via the excavated and appropriately conditioned muck. During the design stage, the reference value, at the crown level or at tunnel centreline (to be measured through the relevant sensors in the excavation chamber), and the relevant operational range are defined (see Section 5.2). Six to eight sensors are actually installed in the bulkhead, for controlling the stabilising pressure (see Fig. 6.16). If the pressure diminishes below the threshold value, the operator reduces the screw conveyor rotation speed (i.e. reduces the outgoing volume), thus favouring the material accumulation into the chamber, and, therefore, the increase of the pressure until the safe value is re-established. The final purpose is to maintain the "earth pressure" value within a 'safe operating' range (see Fig. 6.18).

It is necessary to implement the following controls before the excavation starts:

1. Calibration of pressure sensors with compressed air and/or bentonite slurry and/or a known-density liquid.

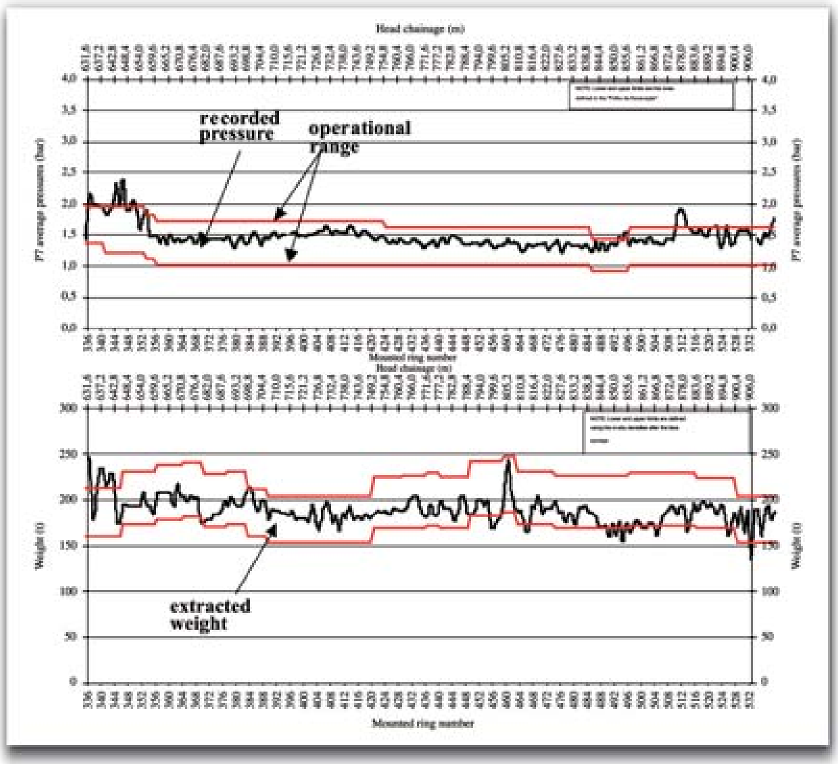


Figure 6.18 Average values of the face pressure and of the extracted weight.

2. Verification of pressure gradient at the respective levels of pressure sensors.
3. Calibration of the other parameter sensors: rotation speed of the screw conveyor, instantaneous rate of advance of EPBS, average total thrust of the hydraulic jacks, weight of the extracted material measured by the scale purposely fitted.

During excavation, other important controls need to be implemented:

4. Verification of pressure variations in relation to the varying operating conditions: rotation speed of the screw conveyor at the corresponding rate of advance and vice-versa; pressure variation as a function of the total thrust, under a steady rotation speed of the screw conveyor; and verification of the torque values as a function of the thrust and plenum pressure.
5. Control that the pressure in the excavation chamber (e.g. average value measured at the crown sensors) is always within the range established in the design.
6. *During stoppage (for ring erection or maintenance)*, the muck in the chamber will tend to consolidate due to the effect of gravity, thus separating the solids from the gaseous and lighter components (e.g. the air contained in the foams). In this case, the pressure should be controlled so that it does not fall excessively and, if needed, bentonite injections should be used to re-establish the design levels. For this purpose, the Secondary Face Support System (SFSS) has been demonstrated to be useful. If possible, the air present in the chamber could also be eliminated via an exhaust valve (Fig. 6.19), located close to the crown. To facilitate maintenance of the pressure during stoppages, it is also possible to reduce the muck extraction rate via the screw conveyor at the end of the excavation cycle (thus letting the

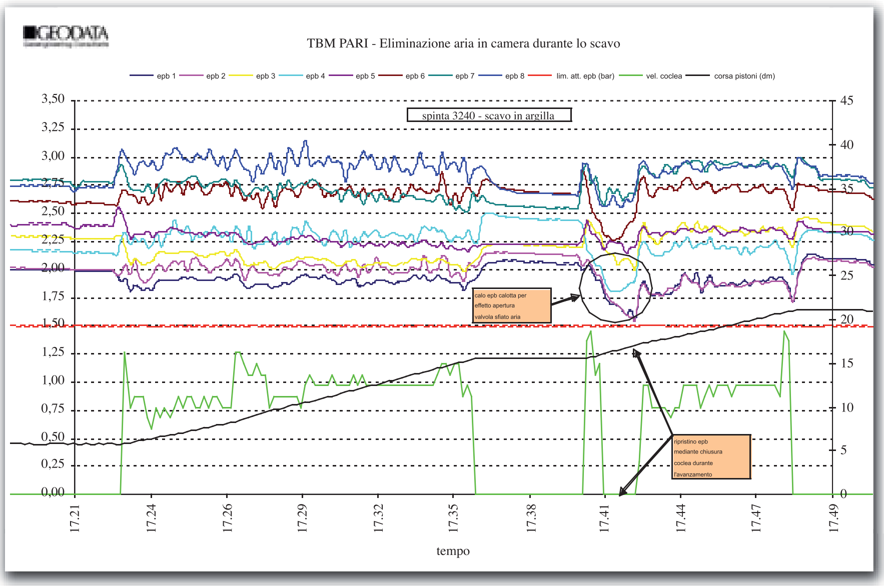


Figure 6.19 Example of the effect of air elimination at the crown on the EPB pressure.

pressure rise slightly). To achieve the same objective, it is also possible to realise a special stroke of a few centimetres, during the stoppage, thus exerting a sort of passive thrust on the face.

7. Again *during stoppage,* in some cases it could happen that the pressure would increase (instead of diminishing): this could indicate a possible "collapse" from the face or a water inflow into the chamber (with inferred possibility that the chamber is not full). A pressure increase in the chamber after the excavation stoppage could also signify that the foreseen support pressure is insufficient due to: local problems, unforeseen and unknown variations of the ground parameters, or increased hydrostatic pressure. All these potential causes will require the operator to implement a *necessary* increase in the reference pressure and to take this into account when updating the PAT. In such cases, at the re-start of the excavation, it is mandatory to perform cross-controls with other parameters (piezometer levels, surface subsidence, "mechanical" parameters of excavation such as torque and thrust, which often provide useful information about the ground characteristics), but at the same time it is appropriate to let the support pressure increase to the value *required* by the system.

It is evident that the control of the trend of the parameter values is more effective than the control of a "single value" due to the necessity to "anticipate", as much as possible, the events.

An active and automatic system to maintain pressure in the chamber is represented by injection of bentonite slurry into the plenum, through the SFSS mentioned above (Babendererde *et al*., RETC, 2005).

The systems so far analysed to control and guarantee the face stability can be considered "passive", in that the operator reacts to a variation of the control parameters and applies countermeasures, which "indirectly" bring the parameters back to the required levels. For increased safety, at least the parameter "face-support pressure" can be managed in an "active" manner (or pro-actively) by injecting an amount of bentonite slurry into the chamber, to raise the pressure to the desired levels, when it falls below the minimum threshold. This operation, although capable of being carried out manually, is easily adaptable to be automated, as it has been done in the Porto Metro (Fig. 6.20) and in substantial part in the High Speed Rail Underpass of the "Nodo di Bologna" (Fig. 6.21).

The adopted system has demonstrated its efficacy in contrasting the pressure drop at the end of the excavation, during ring erection stoppage, or during maintenance.

It should be noted that the volume of bentonite slurry consumed, for each intervention, amounts to only a few tens of litres under the proviso that the chamber is kept full using the procedures previously described. In this manner, the additional investment for this plant installation (which now comes on some EPBs as a normal fitting) vis-à-vis the standard of these machines is really modest, but very efficient and highly effective.

An example (1) of the correct evaluation and (2) of the correct management of the face support pressure, is shown clearly by the graph in Figure 6.22. The graph is a small sample of the continuous monitoring and pro-active management experience incurred in the High Speed Rail Under-crossing in the "Nodo di Bologna". The face-support pressure was closely related to that part of surface subsidence, which

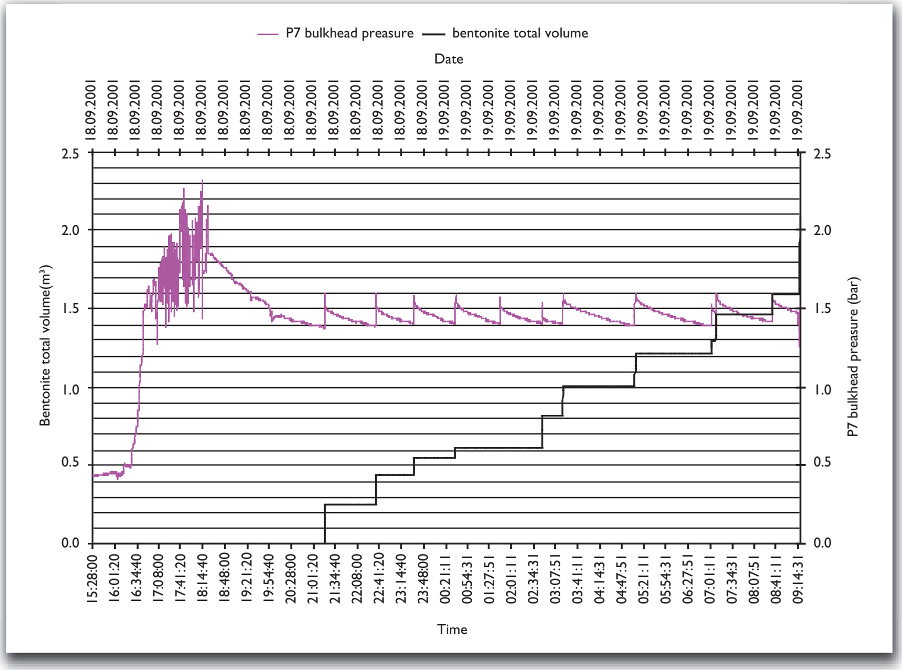


Figure 6.20 Pressure trend as related to small quantity of bentonite slurry injection (Porto Metro).

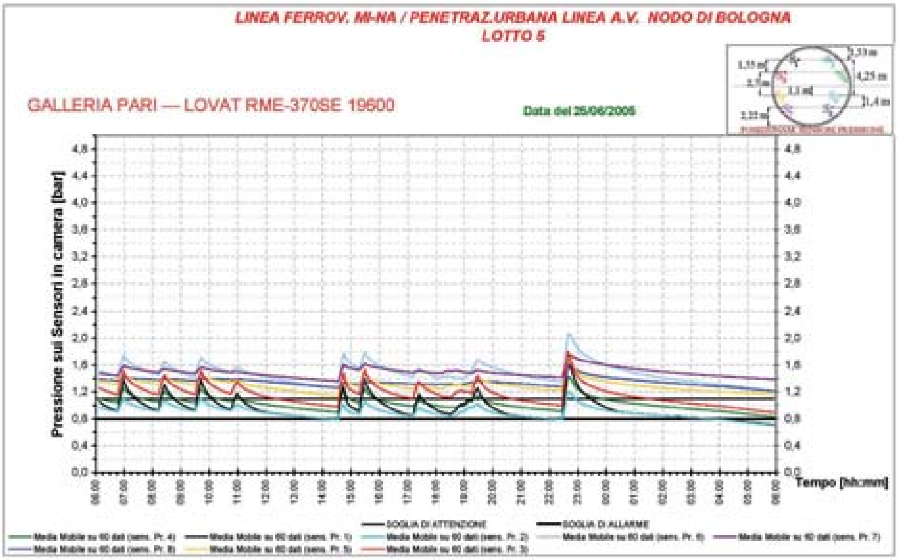


Figure 6.21 Effects of bentonite slurry injection on chamber pressure (H.S. Rail Under-crossing of Bologna).

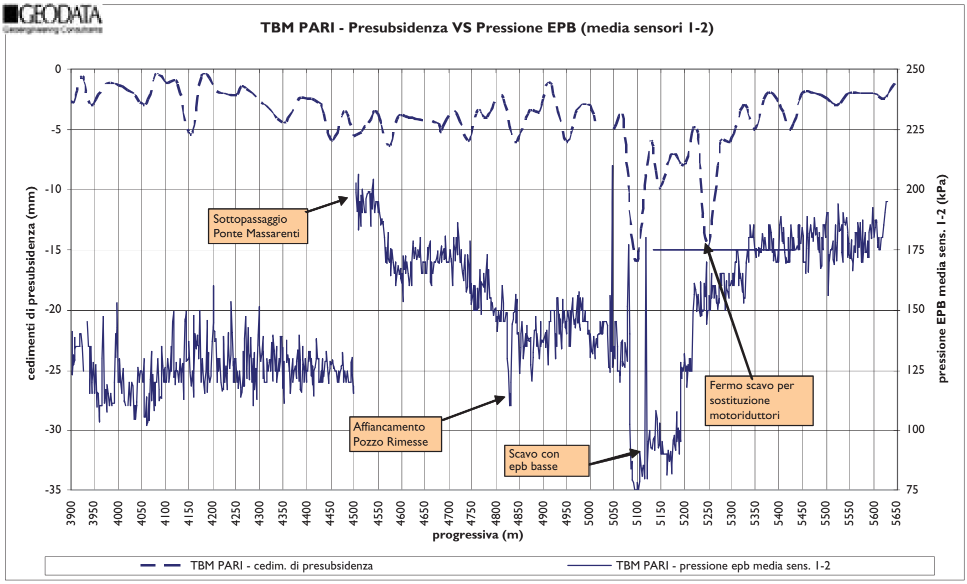


Figure 6.22 Effects of the chamber pressure in controlling the "pre-subsidence" settlement (High Speed Rail Under-crossing of the Node of Bologna).

was previously called "pre-subsidence", i.e. the settlement occurring ahead of the face in the phase in which the TBM approaches the monitored section. The magnitude of the pre-subsidence allows an estimate of the final subsidence value, which occurs at some distance behind the shield. According to the experience in the above project, the pre-subsidence settlement constitutes a more or less defined percentage of the final stabilised settlement. Such percentage or ratio is, however, conditioned by the following factors:

- Type of ground: whether cohesive or loose, in relation to the self-supporting capacity in the transitional phase corresponding to the shield length until the grout backfill point is reached ("physiological" volume loss due to the cutterhead profile, shield geometry, and conical shape).
- Excavation rate: This influences the possibility to effect backfill grouting as soon as possible, thus minimising the settlement "transfer" to the surface of the ground (picked up by the monitoring instruments). The production delays and the stoppages always cause increased settlement in the zone between the face and the shield tail.

6.3.2.2.2 *Extracted weight*

Measuring the extracted volume needs some special calibrated equipment. The most efficient method so far tested is to install scales on the EPBS belt, which transfers the muck from the screw conveyor to the tunnel mucking system, be it a continuous belt or muck wagons. In this manner, it is possible to measure the *extracted material weight* and to have a continuous measurement. By time-integrating the instantaneous extracted weight, the cumulative value of the extracted weight is obtained (Fig. 6.23). Through an evaluation of the in-situ density of the material, its volume can be derived and compared with the theoretical value, which represents the quantity of material be effectively excavated. This type of control avoids excavating beyond the theoretical volume (over-excavation).

The controls to be implemented on the extracted weight, during the excavation phase, are:

1. Calibration of the scales, both statically and dynamically using a known weight; verify whether the functioning of the scales is linked to activation of other mechanical parts (such as the rotation of the cutterhead and/or the thrust cylinders) with the connected risk of not being able to register the weight when excavation is stopped (e.g. during chamber emptying).
2. Verification of the relationship between the weight of the extracted material and rotation speed of the screw conveyor for the same rate of progress (during excavation).
3. Verification of the relationship between the weight of the extracted material and rate of progress for the same rotation speed of the screw conveyor (during excavation).
4. In situ material density evaluation with regular observation and controls of the face conditioning and controlling the muck.
5. Water-meter calibration for the water supplemented by the conditioning additives; instrument calibration of the cumulative water pumped at the face during the excavation cycle.

Figures 6.23 a,b,c show three examples of management of the "extracted weight" by the TBM operators:

The colours of the lines portray the following:

- Light blue: theoretical trend.
- Red and orange: higher and lower attention limits, respectively.
- Blue: actual (measured) value of weight.
- Green: actual (measured) value of volume..

Figure 6.23a shows a regular trend of the extracted quantity, whose magnitude lies between the two attention limits.

Figure 6.23b shows how, at a certain point, the extracted quantity becomes lower than the theoretical one ("under-extraction"). The required action (by the operator) is to try to adjust the result, probably increasing the rotation speed of the screw.

Figure 6.23c shows a case of "over-extraction" in the initial phase of the excavation cycle. The intervention of the operator allows to recover the situation, by returning between the attention limits before the end of the cycle.

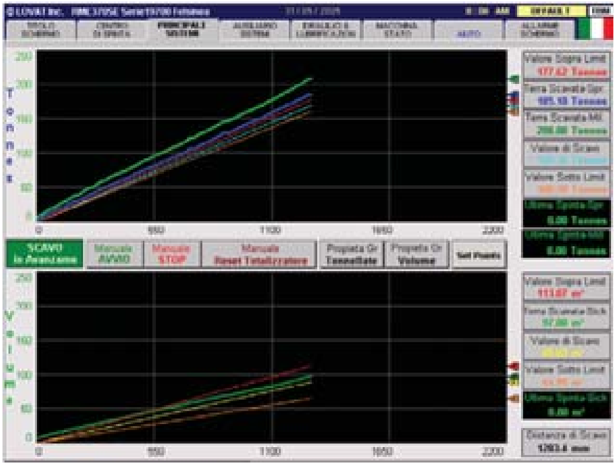


Figure 6.23a Regular trend of the cumulative extracted weight.

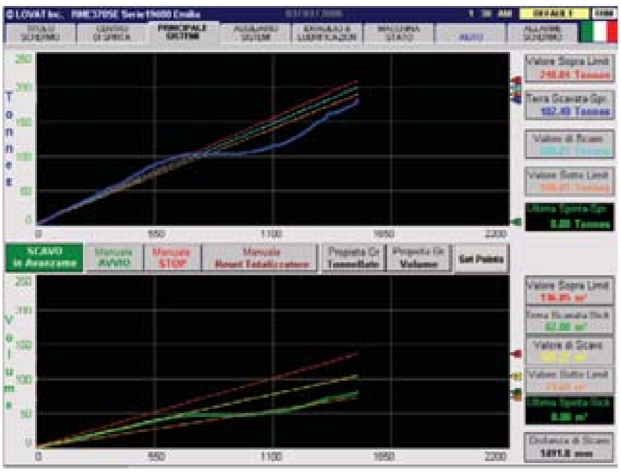


Figure 6.23b Anomalous trend of the cumulative extracted weight: "under-extraction" case.

6.3.2.2.3 Apparent density of the material in the excavation chamber

This parameter provides an indication of the consistency of the material in the excavation chamber (in relation to the conditioning effect characterised by liquid and aerial components), as well as its capacity to supply adequate face support pressure. It also gives an effective indication of the *filling rate* of the plenum.

The joint control of the earth pressure in the chamber and the material extracted through the screw makes it possible to secure the face stability. However, the possibility

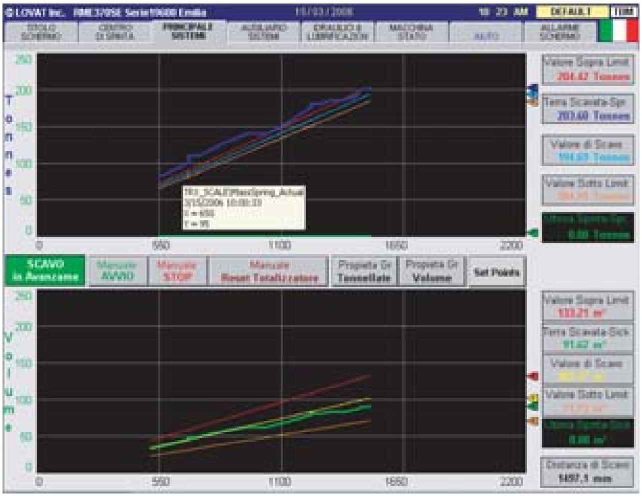


Figure 6.23c Example of the use of screw conveyor for controlling the material extraction.

of not incurring the "collapse" problems is even more effective when there is the certainty that the chamber is effectively full. In fact, if the chamber is full of material with a density level of the same order of magnitude as the material to be excavated, even a face instability phenomenon can not cause substantial damages, for there is no possibility of movement of material at the face.

The above concern is not minor. The use of conditioning products involves injecting vast quantities of air in the excavation chamber. For instance, air could separate from the foaming agent and introduce air "bubbles" in the upper part of the chamber. The pressure measured at the crown will give a "false", or at least incomplete, information: there is pressure, but it cannot be guaranteed that the chamber is full. Only by controlling that the "apparent density" of the material in the chamber is always higher than the minimum threshold value, there can be the certainty of an effective filling of the chamber. This minimum density level corresponds to the density of a material that is capable of transmitting an "effective pressure" to the face. Further, if the chamber is not completely full, some material could enter the plenum, even if the extracted quantity is equal to the theoretical one, which implies that the extracted volume control is illusory; this could lead to false information that could generate very dangerous consequences in urban environment excavation.

Utilizing the pressure sensors on the EPBS bulkhead, located at least at three different levels, a "pressure gradient" or "apparent density" can be calculated, from the ratio of the pressure values measured by the sensors at different levels and the vertical distance between them (Fig. 6.24).

The topic of this "pressure gradient" into the plenum is complex and is affected by many parameters like yield stress of the air-sand-water mixture (Bezuijen *et al.*, 2005), but it is used here only as the "filling index" of the plenum itself.

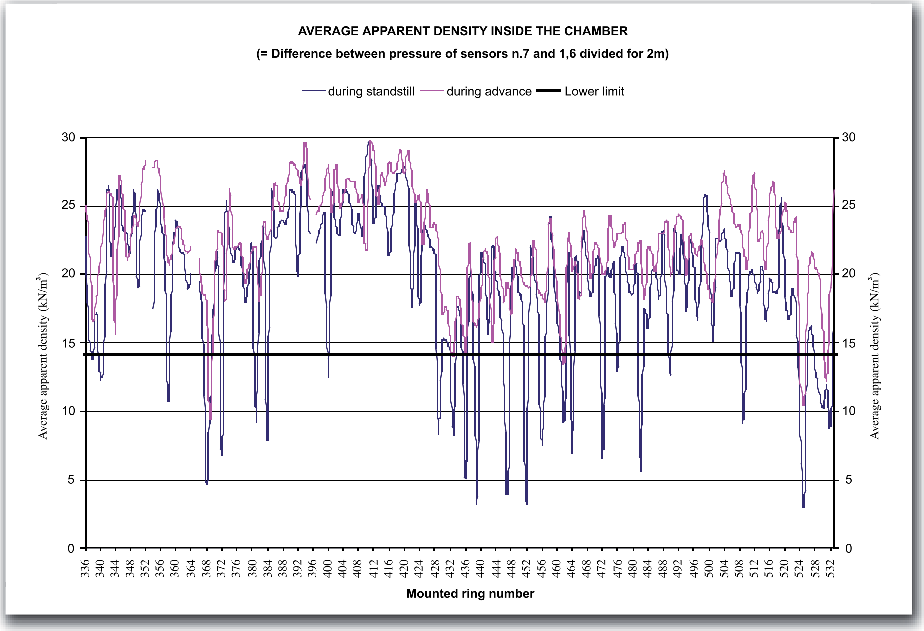


Figure 6.24 Calculated average values of the "apparent density" inside the chamber.

This method for controlling the density of the material was first used in the Porto Metro and has proven to be a simple and effective method. This method supplies, without complicating the life of the operator (already preoccupied with pressure- and volume/weight- controls), an indication of the correct functioning of the system. In addition, the system responds promptly and safely to the operator's intervention: for example, a reduction of the screw conveyor rotation speed quickly brings the values of apparent density above the threshold limits; if this intervention is not successful, an alternative would be to purge the air excess from the upper part of the plenum, through an apposite valve.

6.3.2.2.4 Pre-cast lining backfill

The backfill of the annular void between the pre-cast ring and excavation profile provides support to the surrounding ground and further prevents the deformation of the tunnel lining and, therefore, additional lost ground and potential surface subsidence (see section 5.4). For each cycle, the grout-backfill injection is made on a ring located away from the face at a distance corresponding to the shield length, i.e. the ring which was just excavated in the previous cycle (see Fig. 4.13 in Section 4). The time interval between the excavation phase and the backfill phase of the same ring (strictly connected to the rate of progress of the EPBS) can be critical for the short-term stability of the ground over-lying the shield, especially where over-excavation exists (the higher the progress rate, the quicker is the void stabilisation by lining ring backfill).

Before the excavation starts the following controls are necessary:

1. Calibration of the volume meters (i.e. strokes counter of the pump) and pressure meters (also calculating the expected head loss between the manometer and the injection point) for the backfill mix.
2. Pressure-trend verification in relation to the effective backfill (by bringing pressure to the highest value and measuring the backfill volume in a defined advance rate);

After the excavation has started, the following controls need to be implemented:

3. Control that the grouting operation is carried out concurrently with the excavation and according to the foreseen procedures (in particular, the progress rate has to be adjusted to the concurrently occurring backfill, so that the backfill is carried out at the prescribed pressures).
4. Control that the final injected-grout volumes correspond to the design range.
5. Execute the boring-sample tests (both in the crown and in the invert, where possible) to check the effectiveness of the filling.
6. Make slump and/or 'spreading' tests on site on the backfill grout to evaluate its consistency and deformability.
7. Verify the adequate greasing of the shield's tail brushes (and of the gaps between the brush rings) to avoid grout inflow during the backfill operations.

6.4 INTEGRATED REAL-TIME MONITORING SYSTEM

6.4.1 The need for real-time monitoring

The residual risks in an urban tunnelling project may be controlled during construction following the PAT methodology (Section 2.6), i.e. the implementation of PAT can theoretically assure all the Parties that *the construction shall proceed as a controlled process*. However, as discussed in Section 6.1, each PAT needs to be reviewed and updated constantly during construction following the advancement of each planned stretch of the tunnel. It was also pointed out that one of the important sources of construction feedback for updating a PAT is the data generated by the comprehensive monitoring system, including the TBM performance parameters recorded on-board.

One of the main objectives of the monitoring programme is to check the design hypotheses and detect any anomalous trend in the monitored parameters. Two threshold values are normally defined for each parameter, namely the 'alarm' and the 'attention' thresholds (see Sections 5.1.9 and 5.1.10). The attention threshold value, once reached, should help to draw the necessary attention of the Parties involved to the need for a more careful control of the construction process in order to stay below the alarm threshold. The alarm threshold value, once exceeded, will require that a decision be made on whether or not to activate immediately the corresponding predefined counter measures. Clearly, monitoring during construction plays an important role in controlling the residual risks associated with tunnelling operations in urban areas.

The subject of what and how to monitor has been dealt with previously in the various sections; the following is a summary list of references to those sections and the monitoring aspects:

- Section 5.1.9: monitoring of the actual extent of the tunnelling-induced ground movements and the associated impacts, according to the results of detailed BCS and BRA.
- Section 5.2.3.2: monitoring and adjustment of the face-support pressure, considering that the potential instability of the excavation face is the major source of risk or severe damage to properties and/or infrastructures on surface.
- Section 5.3.7: monitoring of the tunnel structure including its stress-strain behaviour and interactions with the TBM and the surrounding ground.
- Section 5.4.4: monitoring and control of the backfilling process, taking into account that an inefficient and untimely, or an ineffective, backfilling of the tail void is another major source of risk of instabilities and damages.
- Section 6.2.2: monitoring and control of the excavation by a Slurry Shield.
- Section 6.3.2: monitoring and control of the excavation by an EPB Shield.

The amount of data generated by a comprehensive monitoring plan for an urban tunnelling project can become so voluminous as to be unmanageable unless a strategy has been carefully conceived and timely implemented. In addition, in order to respond to the needs of risk management, the data obtained from monitoring must be processed quickly, ideally on site, and presented in an easily intelligible form, to be of immediate value to the decision-making process.

Therefore, there is a clear demand for real-time monitoring and availability of results to effectively manage the residual risks and any unforeseen events during the construction of a tunnel in a city environment.

6.4.2 The need for integration of real-time monitoring with other project information

The correct interpretation of the monitoring data in an "alarm" situation, in order to judge what is really going on "behind the scenes", requires immediate availability of the other project information: the PAT design for the specific tunnel section in question, all the site investigations done in the zone of concern, the performance records of the TBM in the same zone, etc. The need to immediately access all potentially relevant data may disorient the whole project organization, if the information is not organized properly.

Furthermore, in comparison with other construction projects, an urban tunnelling project inevitably generates larger and more complex sets of information, apart from the monitoring data. Effectively managing this bulk of information to ensure its availability and accuracy is an important managerial task, both for the ordinary management and the risk management of the project. Poor or missing information can readily lead to project delays; uneconomical, faulty, or risky decisions; or even the failure of a tunnel section or the collapse of a building. There are instances where the contractor and the engineer have suddenly discovered, after the TBM excavation front has already passed the position of a building, that the building had started to tilt or settle at an accelerated speed. There was an imminent risk of collapse, but no counter measure could be immediately activated to avoid it. With better and timely information, the problem could have been identified and understood earlier, so that remedial measures might have been adopted to stop the tilting or settlement in time, thus avoiding major damages. Both the project design and control are heavily dependent on accurate and timely information, as well as the ability to use this information effectively. At the same time, too much unorganized information presented to managers can result in confusion and paralysis for decision-making.

In general, as a project proceeds, the types and extent of the information used by the various organisations involved will change. A listing of the most important sets of information would include:

- site investigation data, both before and during construction,
- BCS data,
- results of BRA,
- design documents produced at different stages, including drawings and specifications,
- intermediate analysis results during planning and design,
- construction schedules and cost estimates,
- TBM performance data recorded on board,
- other construction records like construction field activity and inspection logs,
- data from various monitoring operations,
- quality control and assurance records,
- health and safety plans and records,
- chronological files of project correspondence and memoranda,
- cash flow and procurement accounts for each organisation, and
- legal contracts and regulatory documents.

Some of these sets of information evolve as the project proceeds. The accumulation of monitoring data over the entire course of the project is a typical example of overall growth of information. The progress of the excavation face with the passage of time results in steady additions in the monitoring-readings, whereas the activation of a new monitoring section leads to a sudden increase in the number of instruments to be managed. Some information sets are important at one stage of the process, but may be ignored in later stages. A frequent example is the monitoring data regarding all instruments positioned on a critical building according to the results of BRA, but not even one alarm threshold value was reached when the TBM excavated under and moved away from that building. Other examples include the planning or structural analysis databases, which are not ordinarily used during construction or operation. However, it may be necessary at later stages in the project to do additional analyses to account for the desired changes. In this case, archival information storage and retrieval become important. Even after the completion of construction, a historical record may be important for use during operation, to assess responsibilities in case of facility failures or for planning similar projects elsewhere.

To efficiently manage the huge amount of data generated by a tunnelling project in urban environment, it is useful to understand also the nature of these data. Many parameters characterizing and/or documenting the tunnel advancement are one-dimensional and can be simply related to the station number. However, the parameters showing the influence zone of the tunnel are generally three-dimensional, for example, the settlement basin around the excavation front. Most construction records and project correspondence have to be related to the position of the tunnel section and the chronological development of the project. Thus, practically all data sets are four-dimensional: 3D for space and 1D for time.

Another important aspect concerns the accuracy and use of the information. Numerous sources of error may be expected in the project information. While numerical values are often reported to the nearest decimal point or to values of equivalent precision, it is rare that the actual values are so accurately known. Living with some uncertainty is unavoidable: a prudent decision-maker should have an understanding of the uncertainty in the different types of information and the possibility of drawing misleading conclusions.

Furthermore, inaccuracy can also come from transcription errors of various sorts. Typographical errors, incorrect measurements from reading instruments, or other recording and calculation errors may creep into the sets of information which are used for project control. Despite intensive efforts to check and eliminate such errors manually, their complete eradication is virtually impossible. However, such errors can be minimized by implementing a computerized validation process.

The transfer and flow of four-dimensional data among the Parties involved in the project is of vital importance to a collaborative work environment, because, in this environment, many professionals are working on different aspects of the project, and sharing information simultaneously, and because data are fundamental to taking decisions. Hence, additional risks can occur if the availability and integrity of information in real time is not assured.

Clearly, success in Risk Management for an urban tunnelling project requires data transparency and a collaborative-work environment, which are often difficult to realize. A collaborative-work environment should provide facilities for sharing data files or databases, tracing decisions, and communicating the information by efficient means. Integration of the real-time monitoring with the other important sets of project information described above, through a properly-designed computerized system, can facilitate the creation of these conditions, constituting an indispensable tool for the efficient and effective management of risks.

If agreed by all Parties concerned, the implementation of a RMP geared with the PAT methodology can reduce, or even virtually eliminate, the possibility for unwarranted claims by the Contractor. Thus, it is necessary that a collaborative work environment be created for the project. The best contractual form to facilitate this creation is discussed separately in the Annex to this book.

Finally, it should be pointed out that while there may be substantial costs due to inaccurate or missing information, there are also significant costs associated with the generation, storage, transfer, retrieval, and other manipulation of information. In addition to the costs of secretarial work and provision of tools such as computers, the organization and review of information often require an inordinate amount of the attention of project managers, which may be the scarcest resource on any construction project. Thus, it is useful to understand the scope and always look for the best alternative for organizing project information.

6.4.3 The current common practice and its shortcomings

Given the huge amount of information associated with construction projects, formal organization of the information is essential in order to avoid chaos. When microcomputers were first introduced to the construction industry in the early 1980s, attempts were made to organize project information into a series of special purpose data-files. A data-file consists of a set of records arranged and defined for a single application system. The use of such data-files was not easy; it was uncommon at that time due to availability and access problems.

In the late 1990s, with the introduction of personal computers, great efforts were made to develop database systems to facilitate the organisation of project information. Equivalent organization of information for manual manipulation is now possible, but tedious. Computer based information systems also have the significant advantage of rapid retrieval of data/files? for immediate use and, in most instances, lower overall costs.

Formally, a database is a collection of stored operational information used by a project team, plus a series of application programs (or user interfaces). This stored information has explicit associations or relationships depending on the content and definition of the stored data; and these associations may themselves be considered as parts of the database.

From a functional point of view, a database has three essential components: the Database Manager Program (DBM), a number of predefined user-interfaces (or application programs), and the Database Manager.

A user need not be concerned about the details of data storage since this internal representation and manipulation is regulated by the DBM, which is the software program that directs the storage, maintenance, manipulation, and retrieval of data. Users retrieve or store data by issuing specific requests to the DBM. The objective of introducing a DBM is to free the user from being concerned with known details of how data are stored and manipulated. At the same time, many different users with a wide variety of needs can use the same database by calling on the DBM. Usually, the DBM will be available to a user by means of a special query language. For example, a user might ask a DBM to report on all readings of all monitoring instruments located in a given zone along the tunnel at a particular date. The desirable properties of a DBM include the ability to provide the user with ready access to the stored data and to maintain the integrity and security of the data.

Predefined user interfaces are the means by which the users view the database. Of all the information in the database, one particular user's view may be just a subset of the total. A particular view may also require specific translation or manipulation of the information in the database. For example, the user interface (or subroutine) to view the settlement trough at a selected location, and its evolution with time, might consist solely of a list of settlement survey points positioned on the ground surface at that location, even if the underlying database would include all the subsidence survey points along the entire tunnel route. As far as that subroutine is concerned, no other data exist in the database. The DBM provides a means of translating particular external models or views into the overall data model. Different users can view the data in quite distinct fashions, yet the data can be centrally stored and need not be copied separately for each user. User interfaces provide the format by which any specific information needed is retrieved. Database "users" can be human operators or other application programs such as the program for displaying the settlement trough mentioned above.

Finally, the Database Manager is an individual or group charged with the maintenance and design of the database, including approving access to the stored information. The assignment of the database administrator should not be taken lightly. Especially in large organizations with many users, the Database Manager is vital to the success of the database system.

Nowadays, one can choose from a series of commercial database management systems for a project and it is always necessary to customize the user-interfaces to suit the particular project needs, and sometimes even to develop additional new interfaces (if the database system selected is an open system).

The importance of relating each piece of project information with its position in the three-dimensional physical space was recognized a long time ago. However, it was only during the last few years of the last millennium that it became possible to maintain this spatial relationship in a database using a Geographical Information System (GIS) as the generalized user interface. Today (2007) the general picture of managing project information in the field of urban tunnel construction can be summarized as follows:

- Computerized databases are adopted, but often not centralized, i.e. the different databases adopted for the same project do not talk to each other, or talk only to some extent and with great difficulty.
- The use of databases to manage the monitoring-data during construction is still rare, but the use of spreadsheets is a common practice. Only in the last few years, a few engineering companies and associations have started to develop integrated monitoring systems on a GIS platform, allowing access to the system via internet. However, no off-the-shelf commercial packages are available.
- Document management systems like Projectwise from Bentley and PowerDocs from Hummingbird have only started to be used by some contractors and owners.
- Various other software are used for particular applications, such as Primavera Project Planner (3P).
- TBM manufactures do provide, for each TBM, an on-board system with database support to record and archive all the relevant TBM-operation and performance parameters, but they are all closed systems and are not developed following a set of common specifications.
- Project communication is done through email and closed computer networks in addition to the traditional means of talking, written reports, and specifications and drawings.

From the point of view of risk management, the major shortcoming of the current practice of data management for urban tunnelling projects is the lack of integration of the real-time monitoring data with the other important sets of project information.

Furthermore, for mechanized excavation in an urban area using a "city machine", the "black box" on-board the TBM is only a partial "flight recorder", because it does not include/record the other relevant pieces of information, typically those reflecting the influence of the tunnel excavation on the surrounding environment, which may, in turn, influence the behaviour of the TBM and/or its operations.

Clearly, there is a strong need for integrated systems to achieve more economic savings and to facilitate greatly the task of risk management of urban underground construction projects, especially those excavated by a mechanized method.

6.4.4 The innovative approach

Knowing the needs of, and the new possibilities offered by, the modern information technologies, it is natural to look for innovations in order to overcome the shortcomings of the current common practice discussed in the previous subsection.

The innovative approach, which emerged out of necessity, is the effective integration of real-time monitoring with the management of all the other relevant project information, to assure an efficient and effective management of the residual risks during construction of a well-designed urban tunnel, excavated by a traditional method or by using a "city machine".

The four pillars for the implementation of this innovative approach are:

1. Relational and centralised database(s).
2. Geographical Information System (GIS) to permit linking each piece of information not only with time, but also with the location of the source of the information through geo-referencing.
3. Internet and/or intranet to permit remote access to the data stored in the centralized database(s) via different types of predefined user interfaces.
4. Commercially available, powerful, document management systems that have open architecture to permit communication with the three technologies mentioned above.

It is useful for the reader to succintly illustrate the relational model of databases. In this conceptual model, the data in the database are viewed as being organized into a series of "relations" or "tables" of data, which are associated in ways defined in a data dictionary. A relation consists of rows of data with columns containing particular attributes. The term "relational" derives from the mathematical theory of relations, which provides a theoretical framework for this type of data model. Here, the terms "relation" and data "table" are interchangeable. An advantage of the relational database model is that the number of attributes and rows in each relation can be expanded as desired. As additional items are defined or needed, their associated data can be entered in the database simply as another row. Also, new relations can be defined as the need arises. Hence, the relational model of database organisation can be quite flexible in application. Application systems can be expected to change radically over time. Thus a flexible system is highly desirable; this flexibility can be readily achieved with the relational database model. The model has many other advantages; the interested reader can consult, for example, the many free publications on the subject available on the world wide web.

There are some other important aspects, which may determine the success or failure of the above-mentioned innovative approach, including:

- All the design requirements for monitoring are transformed into relevant Technical Specifications for Construction, which respond to the questions:
  - what to monitor? (which is both a design and a construction issue)
  - how to monitor?
  - how the data generated by the monitoring should be managed and utilized?
- The responsibility for monitoring is determined by the Risk Management Policy established specifically for each project; it is usually assigned to the Party in the best position to absorb and/or manage the residual risks, specifically:
  - For the conventional contract (where the Contractor is responsible for the construction), it is usual that the Owner/Employer is directly responsible for implementing the monitoring plan; however, he may designate a specialist subcontractor to act on his behalf and sometimes the Designer may be charged to act as this subcontractor. In this case, the Engineer appointed by the Owner shall be responsible for managing the interpretation of the monitoring data and the utilization of the results obtained.
  - For the Design and Build contract, it is common that the Main Contractor is responsible for implementing a monitoring plan in accordance with the contract specifications, as well as, sometimes, for the interpretation and use of the monitoring data. However, the Owner may in any case have a small team (in his project organisation) dedicated to supervise the monitoring execution done by the Contractor and to assist the Contractor to manage the critical situations.

The usefulness of a database system is particularly evident in integrated design environment or integrated management environment. In these systems, numerous application-programs can share a common store of information. Data are drawn from the central database as needed by individual programs. Information requests are typically made by including pre-defined function calls to the database management system within an application program. Results from one program are stored in the database and can be used by subsequent programs without specialised translation routines. Additionally, there is usually a series of predefined user interfaces, by which a user can directly make queries to the database.

However, it should be pointed out that, in an overabundance of enthusiasm derived from the advantages of database systems, it might be tempting to conclude that all information pertaining to a project might be stored in a single database. This has never been achieved, neither is it likely to occur nor desirable in itself, for both technical and non technical reasons which include the following:

- Continuous changes in information needs. As the development of a project proceeds, the types of information and the level of detail required will change greatly. For example, the basic data required for the definition of a new PAT may be quite different from that required for the previous PAT, depending on the presence, or absence, and the type of the infrastructure on the surface.
- Database "diseconomies" of scale. Even with constant increase in the computing power and storage capacity of the data-storage medium, it can become less and

less efficient (and uneconomical) to find the desired information as a database gets bigger and bigger.

- Unsuitable data for computing. There are always some untidy pieces of information, which cannot be easily defined or formalized to the extent necessary for storage in a database.
- Disadvantages of over-centralized data processing. Having a huge database on a supercomputer to do the management of all the information for a project is both costly and not reliable. The current computer technology suggests that using a number of servers, even deployed at the various points where the work is performed, is more cost effective than using a single, centralised super server (computer). For example, the server which stores the TBM operation and performance data on board the TBM itself is a database system, independent from the other database(s) used at the project site. The distribution of data in a limited number of servers not only has cost and access advantages, but it also provides a degree of desired redundancy and increased reliability, even though for the purpose of data integration it may be necessary to write specific protocols to permit communication among the servers.
- Many organizations are involved in a project. Often, each organization needs to retain its own records of activities, whether or not the other information is centralized. Geographic dispersion of work, even within the same firm, can also be advantageous. For example, for the construction of Torino Metro – Line 1, it was necessary to organize different teams to supervize the construction of the various TBM excavation sections (in total, there were three lots), see Section 8.4.
- Incompatible user perspectives. Considering the many organizations involved in a project, it is unavoidable to make trade-offs between different groups of users and application systems, when defining a single data management solution. A good organization for one group may be useless for another. A typical example in this case is the definition of search profile for using a document management system.

Finally, it should be emphasized that, for the implementation of an RMP itself, an integrated monitoring system should be activated as early as possible, especially considering that for many parameters a long-time background monitoring prior to construction is important to understand the true influence of the tunnel construction on the surrounding urban environment.

6.4.5 The pioneer system GDMS

Following the innovative approach outlined above, an Integrated System, code-named "GDMS" for Geodata Data Management System, has been developed and constantly improved in recent years. The prototype of this system was developed and applied to the management of the construction of Lines C, S and J of the Porto Metro Project (1999) and to that of the Torino metro Line 1 (2000, see Fig. 6.25). For both projects, the running tunnels were excavated by EPB shields. The program was subsequently adopted in 2003 by Santiago Metro (Chile) to facilitate the construction of Line 4, which was excavated using NATM. GDMS was restructured with more functionality and made user-friendlier in 2004 to serve the construction

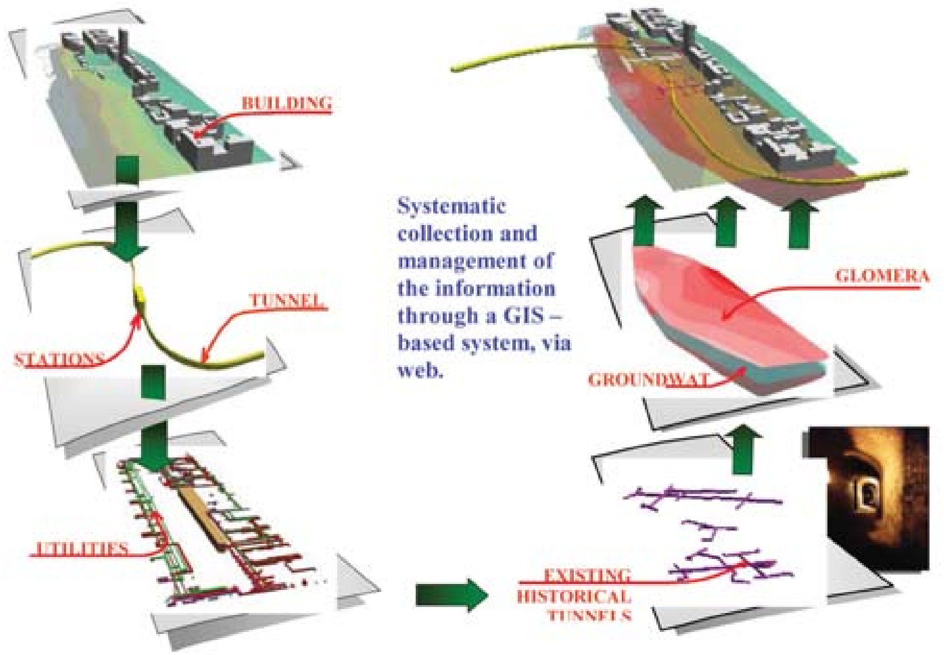


Figure 6.25 Sketch of data management using the prototype GDMS for Turin Metro, during 2000–2006.

of the SMART project in Malaysia (see Section 8.5).

The evolution of GDMS has shown that good organization of information will typically lead to desire to store new types of data and to provide new views of this information as standard managerial tools.

Implementing an integrated information system such as GDMS requires considerable care to insure that the resulting program is capable of accomplishing the desired task. In general, a variety of details are required to make the computerized system an acceptable alternative to a long-standing manual and record-keeping procedure or data interpretation procedure. It was realized right from the beginning of this endeavour that a multidisciplinary approach is obligatory to the successful development of GDMS. In addition, it was also necessary to consult experts, even from other industries, with extensive experience in implementing similar systems. Furthermore, it was also quickly realized that coping with the details makes a big difference in the system's usefulness.

In addition to following the innovative approach described in the previous subsection, the following criteria/principles were established for the development of GDMS:

Simplicity of design for users Simplicity of use was considered a critical factor affecting the successful implementation of the system. Therefore, the Principle of Least

Astonishment was followed in designing the system, and the many associated user interfaces, by making the communication with the users as consistent and predictable as possible.

Open system architecture and modular design In GDMS, comprehensive engineering and "business" databases are foreseen to support different functions throughout the lifetime of a project.

Flexibility of the system for adaptation to future changes This has been an important design and implementation concern, because the construction phase itself includes overlapping design and construction functions, especially when the PAT methodology is employed. During the construction phase, monitoring data of all kinds must be made available to the people involved in the project, and even to the concerned public, in real time. In the example of Santiago Metro, new reports or views of the data constituted a common requirement as the system was used. In fact, initial views of the settlement trough (a series of 2D graphs), along both the transverse and longitudinal sections, were implemented. But the feedback from the early experience of applying the system to Santiago Metro indicated that often it is more useful to show a settlement contour map and its evolution with time for a critical tunnel section. Another example is the application of the system for a new project, for which the Client may already have his favourite document management system or GIS program. In this case, it is necessary to substitute the corresponding module in GDMS with that of the Client. Another important aspect of flexibility is to give the possibility for the users to personalize the views according to their specific needs.

Another important step to be followed for the development of GDMS is the clear definition of the basic functional requirements. In fact, the system is required to perform at least the following tasks:

- Collect in a database, geo-reference, and organise all the *ante operam* information of a given project area (network of utilities, traffic, buildings, etc.) and the basic data about the project itself (e.g. alignment, stations, shafts, worksites, geological model, piezometric levels at different times, etc.), including additional investigations during construction, in a well-structured and unique reference system.
- Collect and geo-reference all the information related to the Building Condition Survey and Building Risk Assessment.
- Collect, geo-reference, and organise all the information produced during construction (e.g. additional investigations, geotechnical and structural monitoring data, progress of the works as documented in the shift reports, etc.).
- Manage the manual and/or real-time acquisition of the geotechnical and structural monitoring data, doing standard graphical representations, comparing readings with thresholds, identifying adverse trends, and automatically sending warning messages to pre-selected responsible people in the organisation by SMS, e-mail, etc.
- Communicate directly with database on-board the TBM, which contains the operation and performance records of the TBM.
- Communicate directly with the database of a document management system.
- Make provisions for validating all the collected information.

- Analyze, extract, and compare data as a function of the geographical locations, so that the information can be correlated and cross-checked (e.g. TBM parameters vs. geology, building monitoring vs. TBM parameters, monitoring readings vs. attention and alarm thresholds, oscillation of piezometric levels vs. progress of excavation, etc.).
- Produce factual reports, semi-automatically, following the requirements of the user.
- Become able to record the "as-built" drawings and the subsequent history. This can be particularly useful during the operation and maintenance life-cycle phase of the constructed facility because, in this manner, plans for the facility can be accessed from the database when changes or repairs are needed.

The modular, functional structure of GDMS is illustrated in Figure 6.26.

6.4.6 Example applications of GDMS and development trends of the innovative approach

As mentioned previously, GDMS was initially developed for application to the Porto metro and the Torino metro project and it has been further developed and applied to other challenging urban tunnelling projects, the most recent being the SMART project (see Section 8.5 for more details).

The benefits of integrated real-time monitoring with efficient management of the other sets of project information have been demonstrated, for example, for the Turin

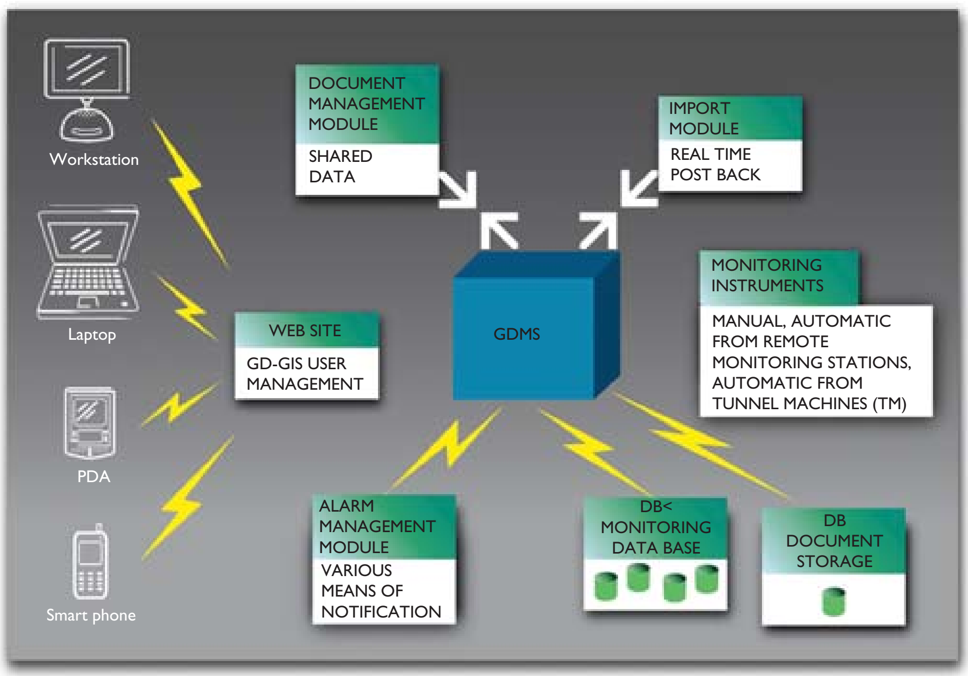


Figure 6.26 Sketch showing the modular design of GDMS (see www.geodata.it for more details).

Metro project (Fornari *et al.*, 2005) and for the Bologna high-speed railway junction project (Marchionni *et al.,* 2007). The trend is that this type of system is becoming indispensable for the successful control of tunnel construction in urban areas, carried out by a modern "city machine" or by the conventional method.

In fact, the GDMS system is a computer-aided, risk-management tool, envisioned for the knowledge and information-intensive construction of future urban-tunnel projects with ever increasing challenges. If all the engineering firms that have embarked on the development of a similar system join forces, there is a good chance that a corresponding industrial standard will be established in the near future.

7

Health and safety

7.1 GENERAL CONSIDERATIONS

7.1.1 TBM Safety requirements

The safety standards for European Community, related to TBM (Tunnel Boring Machine), were developed by the CEN TC 151 Committee (Comité Européen de Normalisation, Technical Commitee), Working Group WG4. Four EN (European Norms) standards related to different tunnelling machines are currently in force. These standards are listed in Table 7.1.

TBM manufacturers have to provide a technical manual, which includes a complete risk analysis that can refer to information extracted from the EN standard. Preexistent national laws are still an important reference for specific situations that are not explicitly reported in the European standard.

In order to effectively choose the tunnelling machine and its auxiliary installations, technical information regarding the construction-design, together with the site constraints, is provided by the contractor. A good construction-design is fundamental for the correct choice of the tunnelling machine and thus for the entire safety of the construction site.

Tunnelling machines currently (2007) employed in urban environments have diameters ranging from 4 to 15+ m. They can be compared to a complex industrial installation, but they are different in installation flexibility, difficulties of materials, personnel access, and work environment. All of these characteristics can increase the level of risks in case of an emergency. As a consequence, it is not sufficient to strictly comply with the current regulations for safety. The TBM risk analysis should be referred to the specific operative context – in case of an urban environment – with the objective of setting up the appropriate countermeasures. EN 12336 standard highlights that concept: "a relationship of mutual information between constructor and user have to be guaranteed with reference to particular condition and place of use (for example, the type of ground and local safety conditions)".

7.1.2 TBM selection and adjustment

Risk analysis has to consider all the hazards related to the use of a TBM, which are in any case fewer than what are normally occurring in the traditional, mining type of environment (see Fig. 7.1).

Table 7.1 EN standards for tunnel boring machine

Machine type	*Standard*	*Date*
Unshielded tunnel boring machines and rod-less* shaft boring machines for hard rock	EN 815	May 97
Tunnelling machines – boring machines, continuous miners and impact rippers – safety requirements	EN 12111	July 2004
Tunnelling machines – pressure zone access – safety requirements	EN 12110	July 2004
Tunnelling machines – shield machines, thrust boring machines, auger boring machines, lining erection equipment – safety requirements	EN 12336	May 2005

* These are TBMs for shaft and not "Raise Borers", which are pulled with rods.

Information sources for the "background" operative conditions have to be the design documents. The designer should include all the necessary information and, in a European context, the "safety design coordinator" will make a preliminary risk analysis linked to the environmental conditions of the site.

The manufacturers should put the CE label on the machines when the E.S.R. procedures are being followed. All these steps need to be followed for pre-1996 machines also. If old machines are renovated, they are considered as "new machines" and, as a consequence, are subject to the relevant Directives (89/392/CEE, 91/368/CEE, 93/44/CEE and 93/68/CEE).

The assembled machine needs to be revised in its totality as a unique machine and, afterwards, an authorized bureau will certify and validate the "system TBM+Back up" for CE marking. The same procedure can be applied to unmodified, used machines.

Figure 7.1 R. Guttuso, Sulphur mine, 1953; Contemporary and Modern Art Museum "Mario Rimoldi" of Ampezzo Rules, Cortina d' Ampezzo.

7.2 TBM WORKING ENVIRONMENT

7.2.1 Design and safety

Excavating with EPBS or Slurry Shield in an urban environment is an activity that is strongly influenced by machine management procedures and back up operations carried out after the assembly of lining of prefabricated segments. Therefore, it is necessary to assume the correct machine management, including maintenance operations. Briefly, safety in TBM workplace is a function of:

- a complete and detailed tunnel design;
- a correct choice of TBM for the tunnel construction;
- a correct choice of excavation tools and equipments;
- carrying out the excavation by complying with design parameters;
- monitoring of design parameters and ground effects, and
- dynamic calibration of excavation parameters with reference to the monitoring results.

If the excavation does not comply with the design specifications, the safety manager has the responsibility to stop the excavation activities, and to ask the designer for clarification.

7.2.2 Risk analysis by working phases

The working phases (of a cycle) of a TBM are illustrated in Figure 7.2. Every phase in the production cycle is complex and involves a great number of persons, equipments and actions. The more complex the cycle, the greater the potential hazards.

However, due to the cyclic recurrence of the routine activities, the hazards can be well defined. Hazards and relevant risks are related to single or combined activities.

The main phases of the working cycle are contemporary even when located in different zones of the machine. Assembly, disassembly, and extraordinary maintenance are non cyclic and less repetitive activities, because they depend on the working site, ground conditions, and TBM progress. As a consequence, they need to be monitored with particular care.

As an example, the activities and the related hazards of two working phases are reported in Table 7.2. Hazards can repeat for different phases, so the same countermeasures for reducing the risks for workers can be applied. The contractor and machine manufacturer should first do a risk analysis of the environment, where the TBM to be manufactured will be applied, and then should analyze the design requirements for the user's need in order to "tailor-make" the machine, from safety point of view.

7.3 CRITICAL PROCESSES

7.3.1 Assembling and disassembling

Before transporting heavy parts of a TBM through the city streets, it is necessary to make a thorough study and a direct check of the transportation route, for its capacity

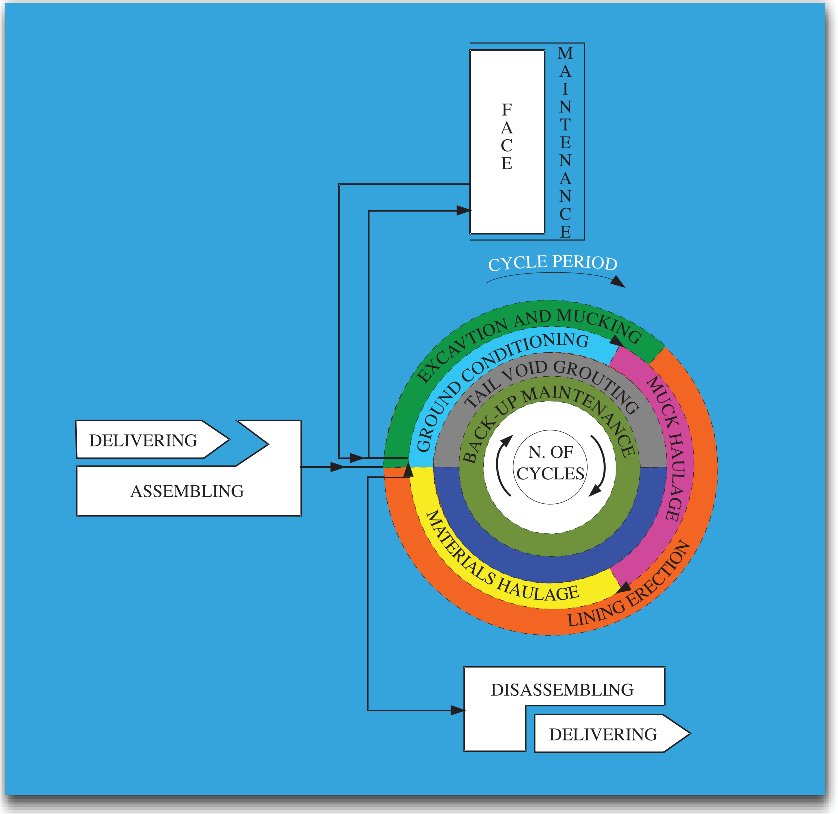


Figure 7.2 Working cycle of a TBM.

to accommodate the dimensions of the parts. The assembling activities also need a large space, which is not always available in an urban context. Sometimes it is necessary to use distant areas for pre-assembly operations. The planning of phases and spaces is critical both for the correct work organization and for the safety of workers. The availability of a very large space does not necessarily mean safer conditions; often smaller spaces require rigorous planning and a better-organized layout, to respect the rule: "organization is safety".

The operations for assembly of heavy and large metallic components, the lowering in shafts, and the surface handling can expose the workers to the risks related to these activities. Assembly planning requires a perfect knowledge of the machine parts (bulk and weight) and the assembly instructions (see Table 7.3). The assembly design includes a correct choice of equipment and auxiliary tools in order to reach all the working places. Indeed, there are a lot of places to be reached by the workers for welding, bolting, positioning, and fixing hanging loads or metallic parts. The TBM shape impedes an upper-level access, mostly in curved parts or parts inside the shield, where the predominant risk for the workers is represented by a potential fall

Table 7.2 Activities and hazards related to mechanized tunnelling

Excavation: danger source	*Emerging dangers*
Excavation (air lock closed)	Noise, vibrations, moving mechanics, pressure pipes breakdown
Mucking (train stopped and conveyor on service)	Moving mechanics, falling objects, tape breakdown, noise
Ground conditioning with polymers, foams, etc.	Pressurized pipes breakdown, allergens
Tail void grouting	Pressure pipes breakdown, allergens
Back up maintenance	Pressure pipes breakdown, allergens, electrical parts on tension, hand tools use
Operational context	Tight and slippery passages, uncomfortable positions of job, unevenness, openings on the transit plans, elevated temperature, poorly ventilated zones, zones too much ventilated, obstacles to ground and on the shapes of passage, insufficient lighting system, fire
Environmental context	Flooding, methane, soils polluted by hydrocarbons or leakages, noise, dust, radon exposure
Lining erection: Danger source	*Emerging dangers*
Segments delivering to erection machine	Movements of heavy materials, raising and transport breach, lack of train braking system
Erector movement	Movements of heavy materials, raising and transport breach
Segment positioning and implantation	Movements of heavy materials, breach of the coupling system, pressure pipes breakdown, uncomfortable positions of job, high working positions, tight and slippery passages, noise
Back up maintenance	Pressure pipes breakdown, allergens, electrical parts on tension, hand tools use, welding smoke
Train arriving/leaving	Convoy in narrow spaces, insufficient lighting system, wrong maneuvers, lacked braking, poor visibility from the guide place
Operational context	Tight and slippery passages, uncomfortable positions of job, unevenness, openings on the transit plans, elevated temperature, poorly ventilated zones, zones too much ventilated, obstacles to ground and on the shapes of passage, insufficient lighting system, fire
Environmental context	Noise, dust, radon exposure

of metallic parts. Therefore, it is important to provide: small-size elevators in order to step into the back-up wagons; scaffolds for reaching the points, where elevators cannot go; railing for pedestrian-path protection along the gangway; and slings and cables for preventing falls, if other protective devices cannot be realized.

For lifting heavy loads, hooking points need to be planned and crane stability needs to be checked. The personnel need to operate in protected areas. Procedures and monitoring instruments need to be provided to the crane operators, because they cannot see what happens to the load when it reaches the bottom of the assembly shaft (see Fig. 7.3 for example).

Table 7.3 Key hazards and actions during assemblage of a TBM

Key hazards	*Key actions*
Obstacles and spaces in building sites	Verification of routes Planning of areas
Lifting of heavy loads	Crane stability check Analysis of workers' positions Good communication
Narrow passages	Planning and organization
Working at high levels	Correct choice of auxiliary tools and special Personal Protective Devices

Figure 7.3 Winter assembly of a TBM at the bottom of a shaft.

7.3.2 Excavation and mucking

If a cavity at the extrados of the shield should be created, it has to be identified and rapidly filled. A rapid filling of the voids can be achieved with injections of expansive materials, usually polyurethane bi-component resins. The management of these underground injections has to be carried out carefully, because some of their components are noxious when coming in contact with or inhaled and, in case of fire, they can produce toxic gasses. Specific tools and procedures for handling these hazards should be chosen carefully. The attentive excavation control and monitoring (see Section 6) reduces or even eliminates the need for these injections, and, as a consequence, could also avoid the relevant risk of fire and chemical contamination for workers.

In the case of an EPB Shield, the muck extracted from the work chamber, or plenum, is transported first out of the plenum by the screw conveyor and then into wagons by a conveyor belt system. In some cases the muck is transported directly to the tunnel portal by a conveyor belt. All the belts should be protected against accidental contact and against entrapment into the rollers, preferably with a protective barrier. Emergency stop-switches are necessary but not sufficient. The key hazards and actions related to conveyor belt transport are given in Table 7.4. The conveyor belt protection arrangements are illustrated in Figure 7.4.

If necessary, the extracted material could be transported to a temporary storage place on the surface, where it will be treated if contaminated by the additives such as foams, polymers, and bentonite. These additives need to be selected by evaluating their harmful effect on underground workers with reference to the allergenic characteristics in a confined environment for contact and inhalation.

7.3.3 Haulage

Transportation of muck (if not done by a belt conveyor), segments, grouts, spare parts, and workers is carried out using trains, which implies the following risks:

- derailment in tunnel, in TBM, or in manoeuvre square;
- runover people and materials;
- fire and smoke, and
- working place pollution.

The reduction or obstruction of driver-visibility in the train cockpit, with reference to the direction of tunnel entry towards the TBM, represents a peculiarity of this transport system that needs to be compensated by the installation of video cameras and a monitor in the locomotive cabin. However, the operators visibility still remains uncertain despite the use of these tools; therefore, the driver needs to become familiar with the working points along the route which have to be well illuminated. An acoustic signal has to be provided during the train transit.

The entry of the TBM back up is controlled by a traffic light and, in some situations, it is recommended to install traffic lights along the route so that the driver can

Table 7.4 Key hazards and actions related to conveyor belt transport

Key hazards	*Key actions*
Chemical risk from filling materials	Excavation monitoring Choice of materials Suitable equipments and procedures Avoid storage in TBM
Chemical risk from excavation additives	Knowledge of allergenic characteristics Medical consultation
Conveyor belt	Belt protection with grids

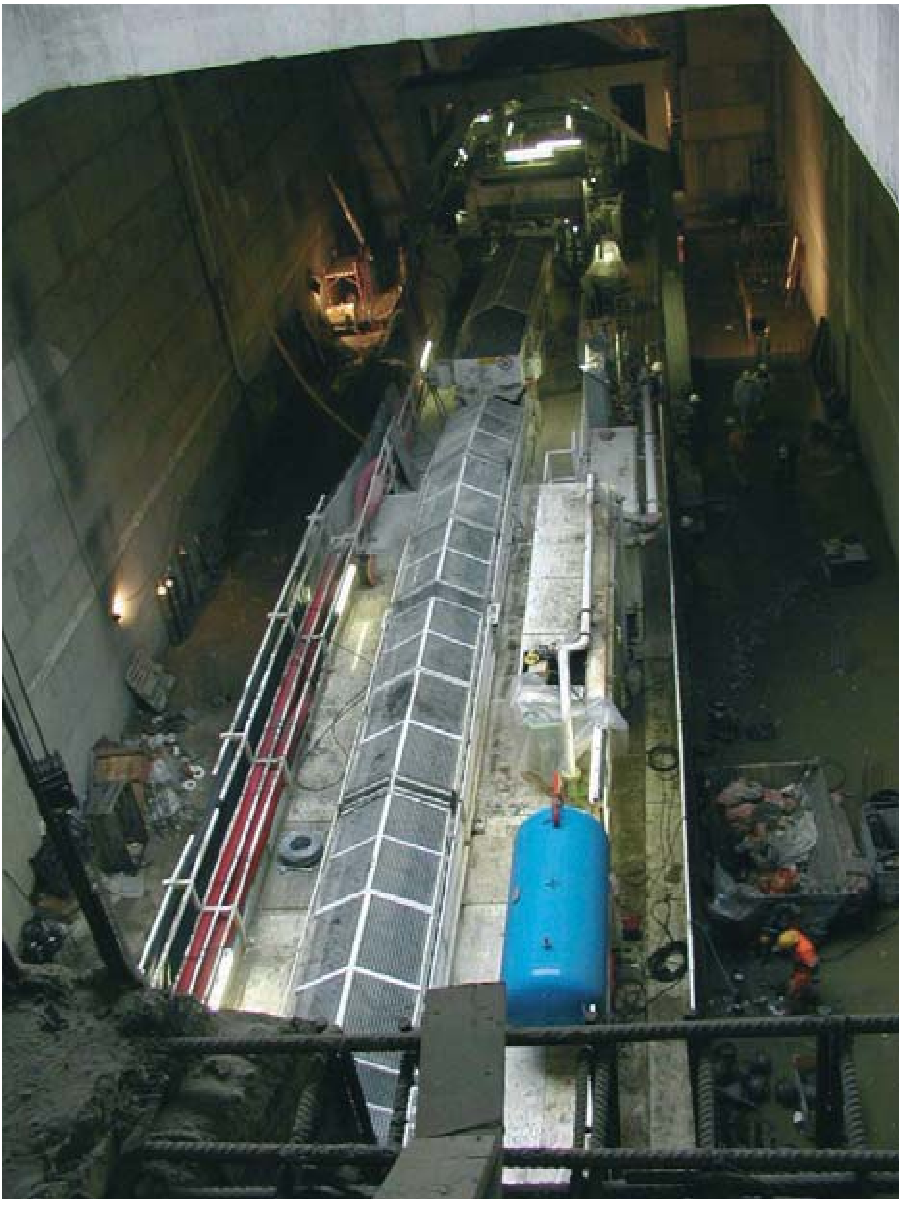


Figure 7.4 Conveyor belt protection arrangements.

advance securely. Train velocity is an important parameter that needs to be controlled; it depends on the quality and inclination of the tracks, and the loads to be carried. Finally, it is important to have correct braking space, especially in proximity of the face. An effective communication system among the train, the TBM, and the surface station is essential.

The transport of workers is generally done by means of special wagons. Tunnels in urban context are characterized by a series of access points, so that the distance to reach the TBM is often less than 1000 m. For this access, it is recommended to carry out a protected pedestrian path, along a banked metallic gangway, so that the risk of being run over is reduced (see Table 7.5 and Fig. 7.5). Where this is not possible, it is essential to guarantee a space in a range of 50–70 cm, which should be sufficient to avoid wagons-men collisions.

Figure 7.5 Pedestrian gangway.

Table 7.5 Key hazards and actions related to railway transport

Key hazards	*Key actions*
Short locomotors visibility towards TBM	Video camera on the back train Tunnelling work positions indicated Traffic light Dead man system
Path slope	Automatic braking wagons Stockyard and machine buffer
Worker run over	Pedestrian gangway High-visibility protective personal devices
Load overall dimension	Template passing

7.3.4 Lining

The hazards arising from handling of the segments and their release from the erector can generate high risk if the workers are not in a protected area and if the procedures are not strictly applied. A concrete segment has a weight of several tons; and the effects of a release from the erector at a considerable height inside the shield can be limited to substantial material damages only if the working positions and the safety procedures have been effectively organized. It is recommended to test the mechanical and pneumatic hooking efficiency by lifting and holding the load a few centimetres from the ground just for a few seconds. The erector worker, operating in a protected area, and checking the position of the other workers in the vicinity, will be able to lift and set the segment. At that moment, by staying at a higher position, the responsible worker will install the segment by bolting or driving the connectors.

A lot of accidents have been caused by collision with the segment during the phases of approach to the erector. The separation of the transit passage from the working zones and pedestrian paths avoids the contact and collision.

The choice of the method for connection between segments and successive rings has a big impact on the work organization and on the TBM structure itself. The adoption of automatic connectors, for example, avoids the bolting operations which have to be carried out at high elevations, with working balconies and gangways for approaching the cap.

A list of key hazards, and the key actions for managing the potential risks, is provided in Table 7.6.

7.3.5 Maintenance

The maintenance operations can be divided into the following three types:

- installations along the tunnel and on the surface site;
- electrical/mechanical maintenance on the TBM, and
- substitution of tools and repair of the excavating head.

The first two operations are routine activities that need special surveillance when carried out during transit of trains, or concurrently with other operations such as tail void grouting.

Access to the plenum and its preparation need a careful analysis. The plenum, where the tool replacement and cutterhead maintenance operations are carried out,

Table 7.6 Key hazards and actions related to lining installation

Key hazards	*Key actions*
Segment falling	Efficiency of the catching system Segment engagement control Remote control
Segment transit	Separating passages and working platforms
Working operation at high altitude	Choice of connectors Ergonomics of working places

is a narrow and slippery place, which can expose the workers to the risk of fall on metallic obstacles. The hazard related to a local face collapse maintains a high-risk level, in spite of support procedures, through air or slurry pressure. Workers are in a pressurized environment that limits their stay and their fatigue endurance. Their work consists also in assembling heavy metallic elements and in the use of manual tools (see Table 7.7 and Fig. 7.6). All access and work tasks in the plenum are restricted to skilled personnel only. The workers are subject to medical examination because of the work in hyperbaric conditions. In a few special situations, the access to the plenum of a Slurry Shield, if in mud pressure, is restricted to personnel with underwater equipments who have been trained to work in conditions of absence of visibility. The case is

Figure 7.6 Assembling air locks.

Table 7.7 Key hazards and actions related maintenance

Key hazards	*Key actions*
Work in pressure	Skilled personnel First aid transport and medical treatment
Work in heights	Platforms and slings Half-full chamber maintained
Head movements	Rotation blockage Maintenance team charge

very uncommon in an urban environment, where the working pressures rarely exceed 2–3 bars, because of the small overburden.

For the maintenance of a "city machine" with an air pressure of 1 to 3 times the atmospheric pressure, the following conditions have to be guaranteed:

- skilled personnel with hyperbaric qualification;
- guaranteed excavation-head rotation blockage;
- presence of a moving scaffold in the excavating chamber;
- arrangement of hooking points for safety sling;
- do not empty 100% of the chamber, while work is being carried on in the upper part to reduce the height of fall of people or materials;
- closed sliding doors (if provided);
- rescue sling for recovering the injured from the bottom of the chamber, lifting apparatus, and transfer into the hyperbaric chamber;
- presence of locomotives with stretcher in the machine, and
- activation of emergency hyperbaric chamber at the entrance and provision of a health worker.

In some cases, the maintenance operations require also to reach external parts on the front of the cutterhead (see Fig.7.7). In this case, previous ground treatment of consolidated zones around the face should be done in order to allow the cutterhead

Figure 7.7 View from the air lock.

retraction, without any collapse. Special procedures have to be issued and comprehensive monitoring of ground movements are required.

7.4 ADDITIONAL CRITICAL ELEMENTS

7.4.1 Noise

TBM working place is characterized by noise accumulated from, and generated by, various correlated sources: equipment placed on the back up, excavation activity, and material transport. The confinement of the working spaces aggravates the problem. It is recommended to adopt all the effective measures for reduction of emissions, and for minimizing the harmful effects on the workers. The TBM manufacturer has to declare the maximum generated level of the sound in the working areas and, subsequently, put the CE mark on the TBM in conformance with the 2000/14/EC rule.

Reduction of the generation of noise at the source is the responsibility of the manufacturer; it needs to be achieved by means of structural actions such as positioning and layout of the TBM along with the back up (electric and hydraulic equipments, secondary fans, conveyor belt's engine, hoppers), and correct design of mechanical, and hydraulic drive system.

The reduction of noise effects has to be reached by:

- Soundproofing the machines, or using machinery with sound absorbent devices.
- Soundproofing the rooms, where the personnel are working or resting during break.

While the manufacturers evaluate the sound emission level in the working environment, contractors have to evaluate the Sound Pressure Level (SPL), expressed in dBA, in the different tasks and decide which actions need to be adopted to reduce the risks. For instance:

- use of a protective personal device for specific operative tasks;
- staff turnover (for providing relief) in locations where noise intensity is higher, and
- personal monitoring of intensity and frequency of noise.

Table 7.8 reports the workers' typical noise exposure (estimated value of SPL) that can be measured around a TBM with reference to different working phases and positions.

Table 7.8 Estimated values of SPL (expressed in dBA) in the TBM areas

Operation	*Erector*	*Back up lower platforms*	*Back up higher platforms*
Excavation	>85.0	<85.0	<85.0
Excavation and Mucking	>85.0	<85.0	<85.0
Lining	<85.0	<80.0	<80.0
Grouting	<85.0	<80.0	<80.0

Source: Metro Torino measurements, 2002–2005.

7.4.2 Ventilation

The design and realization of an underground ventilation system has to be carried out with reference to the surface operative context. The urban environment is characterized by a series of air pollutants, such as fine dusts, fumes, and combustible gases which may result in exceeding the alarm levels for environmental pollution, most of all during winter months. If the tunnel takes air from a polluted area and brings it into a confined environment, the pollutant concentration will increase in those areas where the constructor is trying to extract or dissolve pollutants produced by the working operations. For this reason, the air intakes need to be located far from traffic, traffic lights, or the parking area for heavy vehicles. The dust and fumes are usually concentrated on the ground; therefore, the air intake needs to be placed at least 4–5 m above the ground.

The configuration of a TBM air distribution system is quite critical. Even if the main installation is correctly dimensioned, an air distribution system improvized by the workers does not guarantee safe working conditions. The air distribution system needs to be correctly designed by the manufacturer. Mobile deviations may feed other zones. Control rooms and other fixed positions require a separate and conditioned ventilation system. Rapid airflows should be avoided on belts because they dry the material and can raise dust. During the installation of a TBM, the following aspects should be taken into account: the minimum personnel needs (3–4 m^3/minute each); the quantity of air for dissolving or extracting fumes and dusts produced by diesel engines (locomotives generally); and the air required to control the heat resulting from all personnel and equipment and especially by the electrical installations. It is important to note that the ventilation pipes require periodic inspection in order to avoid ripping and loss of air along the tunnel.

7.4.3 Dust

As a group, dusts are extremely dangerous because of their dimensions related to the particular activity at pulmonary level. Fine dusts have a diameter lower than 100 micron. Of particular interest to urban tunnelling are the dusts having diameters between 0.1 micron and 100 micron and, according to their size, on their capability of hanging at an aerial level. The tunnel face represents the main source of dust generation. The propagation of dust from the face is limited. The ground is extracted from the excavation chamber where water, polymers, and foams (having the function of transmitting supporting pressure to the face) capture dust particles and stick them to the extracted material (EPBS). In an SS, the mucking process is hydraulic, so the dust particles remain confined in the transport system.

Some minor dust sources arise along the machine back up: the wet material falls from belts, it dries because of its exposure to the airflow and the dusts are released into the atmosphere. A portion of the dusts falls on the back up wagons, and the rest flies along the tunnel, precipitating on walls and on the train transit routes. At every train passing, the dust is lifted. It is essential to control the minor dust sources on the machine and along the tunnel. Washing platforms and working gangways has to be a routine practice.

Dusts are dangerous for health and safety in different ways:

- clouds of dust lifted by transports may reduce the visibility, most of all in an emergency situation;
- dust carries other pollutants such as radon and hydrocarbons, and
- dust is harmful for mechanical action inside the respiratory system, but also for biochemical actions related to the permanence and accumulation of noxious dusts.

A sequence of interventions is required for noxious dusts:

- avoid generation of pollutants;
- capture pollutants that cannot be stopped at their source;
- adopt closed cabins for workers;
- provide personal protective devices in particular situations, and
- make frequent or continuous monitoring.

It is not sufficient to design a monitoring and dust-suppression system to guarantee the complete capture of noxious dusts. It is essential to take into account the entire excavation system, starting from the machine choice. For example, a Slurry Shield is more efficient compared to an EPBS in terms of the smaller amount of dust generated and in terms of transportation of dust out of the tunnel. The ventilation system can employ pressure and/or suction, for responding to requirements and the operative context.

7.4.4 Smoke and PAHs

PAHs (Polycyclic Aromatic Hydrocarbons) results from an incomplete combustion of organic materials containing carbon. PAHs adhere to fine dust, that is mechanically generated by thermal engine functioning mainly at low temperature. Those particles can be inhaled due to their dimensions, and PAHs pass into lungs through respiration. Gas and particulates emitted by diesel engine vehicles also contain some derived compounds containing the nitrite group NO_2; these compounds are more actively carcinogenic compared to PAHs. The serious danger of PAHs and particulates, with particular reference to confined areas as a tunnel, forces the adoption of the procedure about noxious dusts already described in section 7.4.3. In a mechanized excavation, the employment of diesel locomotives for personnel and material transport represents the only source for the production of fine particulates and incombustible waste. If the TBM working place is maintained clean by means of direct ventilation, the driver of the service train is the only person exposed during his transit in the tunnel. The driving cabin has to be closed and requires an air filtering system, mostly if the underground route is very long.

7.4.5 Other hazards

In urban areas, the risk of encountering hazardous gas is usually restricted to the discovery of a gas pocket or gas coming from tank leakage or pipe failure. It is essential to install on the TBM head a series of sensors for gas such as CH_4, CO, O_2, and Sox (see Fig. 7.8). However, the likelihood of the presence of natural gas should always be investigated in geological studies and appropriately considered for the choice of excavating machine and auxiliary equipments.

Radon, Rn-222, is an inert gas produced from the decay of uranium radioisotopes in the rocks. Radon can be found far from the source because it is water-soluble; and when dissolved in underground water, it moves through rock fractures and can be found concentrated in alluvial formation and in karst areas. The inhalation of radon and its decay elements for a long period increases considerably the risk of cancer for the population exposed.

The presence of radon is frequent in underground excavations. It is mandatory to monitor radon levels by installing dosimeters on the TBM and along the tunnel. The main control measure for radon is provided by the ventilation of the tunnel. The fresh uncontaminated air supply dissolves and carries out the gas.

Hazardous waste or polluted soil could, with very low probability, be encountered when tunnelling in urban areas. Depending on the depth of the tunnel, the waste could include: organic materials, heavy metals, used batteries, and oil spills. A pre-design survey of the surface area along the selected alignment should include the investigation of the possible presence of hazardous waste and ensure a very low probability of its occurrence. The Project Manager and Contractor should be prepared to deal with the hazard, should it materialize.

Figure 7.8 Methane detectors.

7.5 EMERGENCIES

The organization of emergency procedures in an urban construction site is in a more favourable condition, because of the interface with the first-aid stations and fire fighting services, which protect the city. Fire-prevention and first-aid procedures can be organized together with the fire brigade so that skilled and trained personnel can attend the containment and extinguishing activities. In addition, it will take only a few minutes for the rescue teams to reach the site. It is required to organize informative meetings in order to reduce the intervention time by adopting the correct emergency equipments, precisely flagged in the site. The site must have its own equipment and skilled personnel for acting immediately on fire and for bringing first aid.

7.5.1 Fire

A fire in a tunnel has to be always considered a "top event". First of all, fires must be avoided, but in case of the spread of fire, it is important to put it out immediately through monitoring systems and fast extinguishing aids.

It is compulsory to evaluate the following issues:

- sources of fire: electric motors, inflammable materials, and equipment;
- number, location and duration of the workers' presence in TBM and along the tunnel during different working cycles;
- number, location, fire load reduction, and duration of transport presence in the tunnel;
- estimate of personnel transfer velocity in case of emergency;
- estimate of the response time of first-aid, and
- other aspects to be considered, such as ventilation shutdown, rising of temperature, etc.

In a mechanized-excavation site, the more frequent source of fire is represented by the train locomotive where the fire load due to the fuel tank is concentrated. In order to immediately put the fire out, a careful maintenance has to be carried out on the locomotives, and the drivers need to be properly trained on fire-fighting aspects. They have to be equipped with auto-protective devices and fire extinguishers on board. It is recommended to have driver-operated foam extinguishers.

A fire in a tunnel is very dangerous for the TBM personnel because of the release of smoke. For this reason, a careful risk evaluation should be done. A safe place where to wait for the rescue squad has to be provided. In an urban excavation site with close stations/shafts (entry distance <500 m), the risk of a tunnel fire is limited to the TBM exit way (station or shaft). The fire fighter can be extremely quick and the personnel, provided with self-rescuers, can exit the tunnel without waiting the arrival of the fire fighter. Thermal sensors will be installed also on electric TBM machines because, due to the hydraulic oil temperature and belt surveillance, they can alert the crew. All these systems have to be integrated.

7.5.2 Rescue

The first aid must be guaranteed in every area and for every working phase. The TBM should have a service train, provided with devices for stretcher transportation. If a double railroad cannot be realized, mobile exchanges have to be provided along the tunnel.

Particular attention must be given to organize the rescue for works in hyperbaric conditions as pointed out in section 7.3.5. Rescue equipment such as breathing apparatus, fireproofing overall, manual trolley, and stretcher must be stored at the nearest entry of the TBM for a quick availability. TBM personnel should be trained for self-rescue and should be provided with anti-smoke masks for reaching, by foot, the closest exit. Service shafts should be provided with equipment for vertical lifting of stretchers. Real training sessions for following the full rescue procedures, from recovery of the injured in excavating chamber to the ambulance on surface, are necessary (see Fig. 7.9).

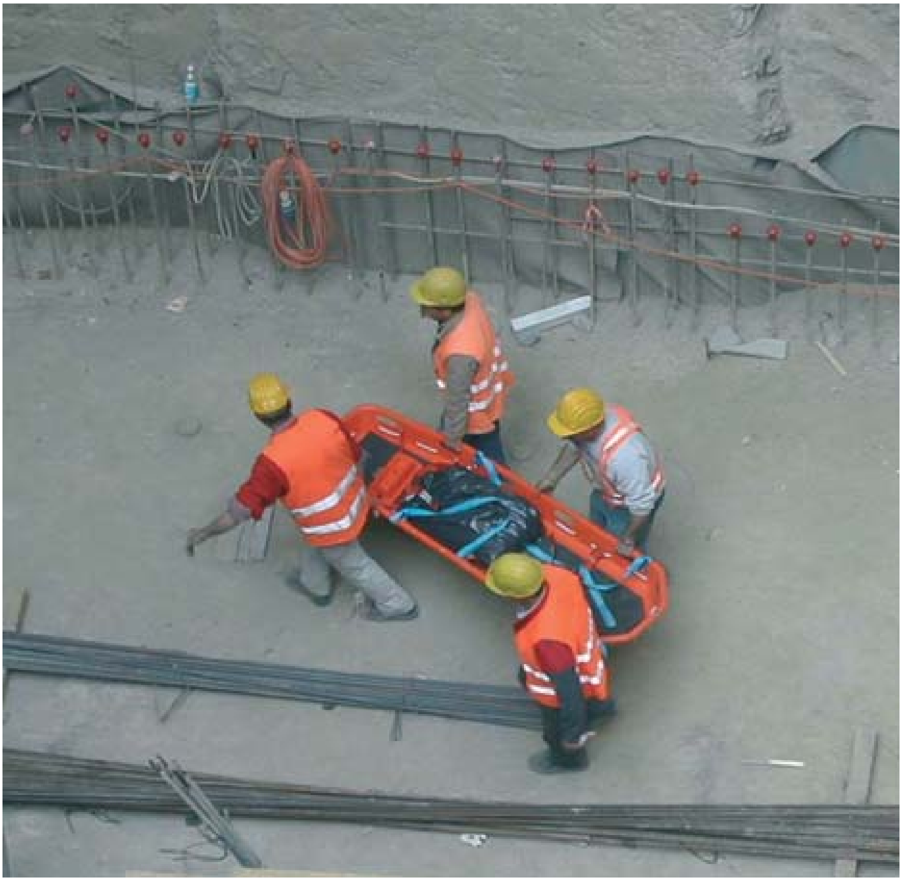


Figure 7.9 Rescue training session.

8

Case histories

This section includes the case histories of six tunnelling projects (partially carried out by authors of this book) with the objective of providing informative details of some of the subjects addressed in the previous sections of the book.

The following table illustrates a quick reference to the source of information included in the various case histories.

Subsection	*Case History*	*References*
8.1	EOLE Project, Paris, France	Bochon *et al.* 1997
8.2	St. Petersburg Metro, Russia	Grasso *et al.* 2004 T & T International, 2002
8.3	Porto Light Metro, Portugal	Gaj *et al.* 2003
8.4	Turin Metro Line 1, Italy	Carrieri *et al.* 2004, 2006 Grandori *et al.* 2005
8.5	Smart Solution, Kuala Lumpur, Malaysia	
8.6	High Speed Railway Line, Bologna, Italy	Minguez *et al.* 2005 Marchionni *et al.* 2007

8.1 EOLE PROJECT - PARIS

8.1.1 Project particulars (Table 8.1)

Table 8.1 Project particulars

Location	Paris, France
Name	EOLE—Est Ouest Liaison Express Gare Nord Est—Gare St.Lazare Condorcet
Construction period	1994–1997
Owner	SNCF—Societé Nationale des Chemins de Fer Français
Contractor	J.V. Lodigiani S.p.A.—DCG—Desquenne et Giral Co.
Consultant and construction follow-up	Geodata S.p.A.

8.1.2 General description

The EOLE Project was a part of the Line E of the "Réseau Express Régional (R.E.R.)", a new communication axis between the East and the West side of the region around Paris (Fig. 8.1).

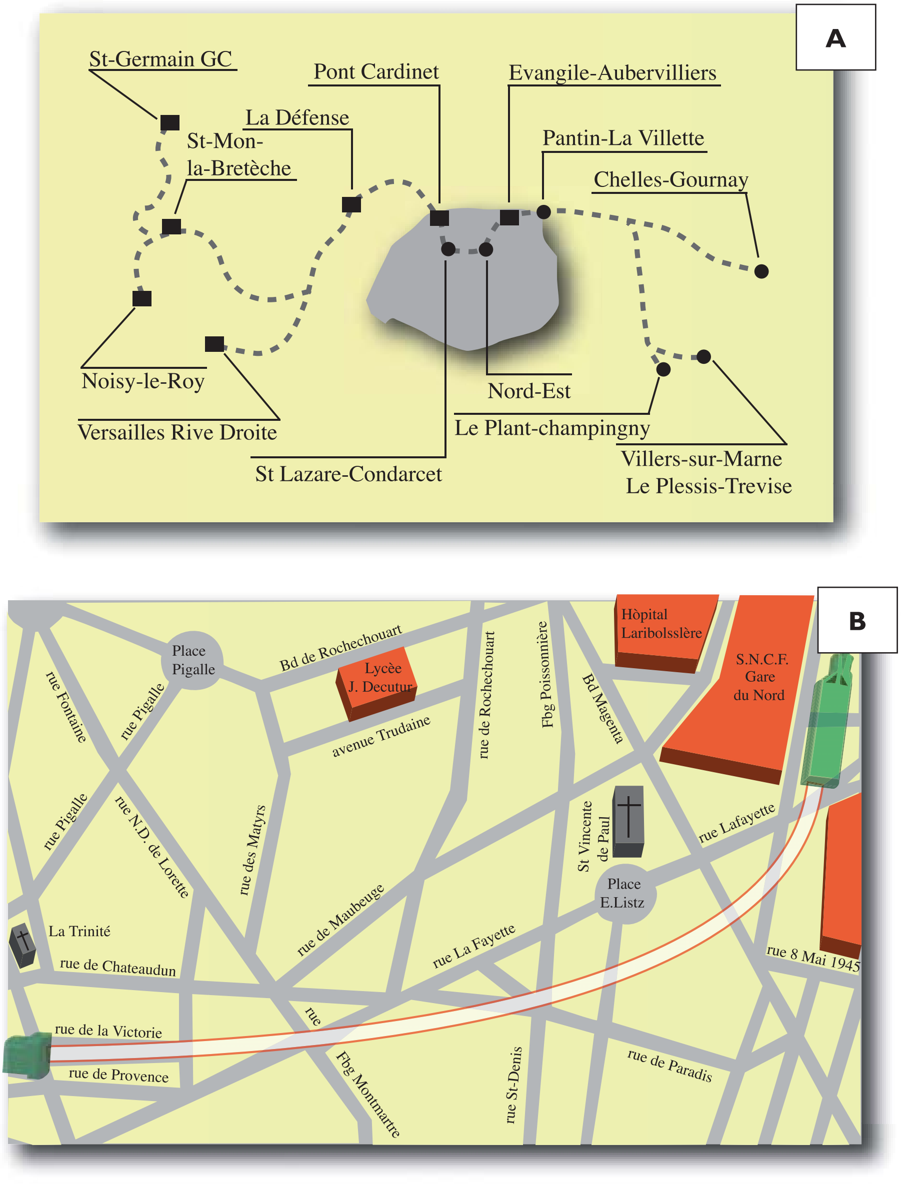


Figure 8.1 Plan view of the St. Lazare and Nord Est stations and the connected tunnel (A: general layout, B: detailed plan view).

The alignment foresaw a twin-tunnel (Fig. 8.2), 1700 m in length and about 30 m below the street level, and two underground stations, the “Magenta station” and the “Condorcet station”. The Tunnel excavation, using a Hydroshield (Fig. 8.3), began in 1994 and finished three years later, in 1997.

The construction of the first tunnel went smoothly without any major problems. However, some difficulties were encountered when excavating through the “sables de

Beauchamps" formation due to the presence of sticky materials which led to clogging of the cutterhead and the plenum, especially in the muck-sucking zone. During excavation of the second tunnel, a collapse occurred till the ground surface, causing cracking of a building under restructuring. The restoration works lasted for over 6 months. The Contractor was not judged to be responsible for the damages.

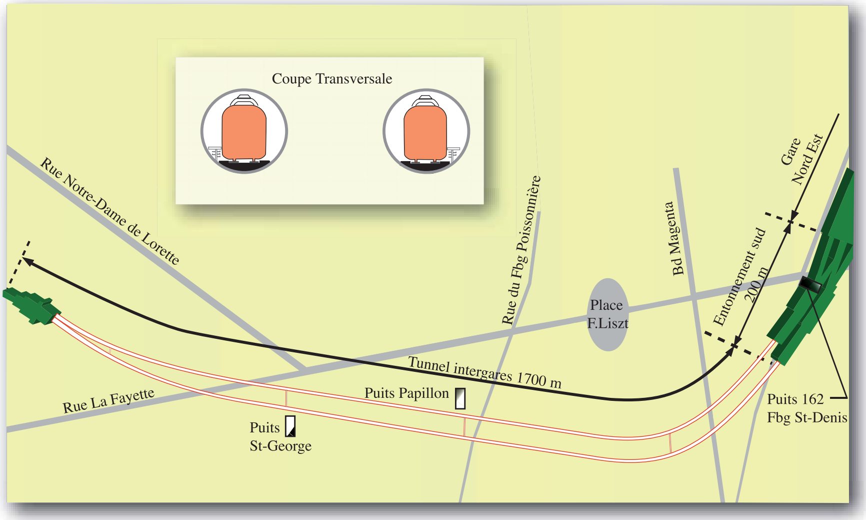


Figure 8.2 A plan view and a section of the EOLE twin-tunnel.

Figure 8.3 A prospective of the Voest Alpine Hydroshields machine.

8.1.3 Tunnel characteristics (Table 8.2)

Table 8.2 Tunnel characteristics

Type	Twin tunnel
Length	2 × 1,7 Km
Overburden	20 – 30 m
Diameter	7.4 m
Lining type	Segmental
Ring Type	Universal
Thickness	35 cm
No. of segments	5 + 1
Ring length	1.4 m
Connections	CONEX

8.1.4 Environmental and geological conditions

The geological context of the ground was characterised by a sequence of Tertiary formations, weakly disturbed by tectonic actions, in terms of global deformation. The alignment crossed a heterogeneous sequence of different types of ground (Fig. 8.4).

From the hydrogeological point of view, two different water tables were intersected by the alignment: the first was contained in the Beauchamp sand, above the clayey

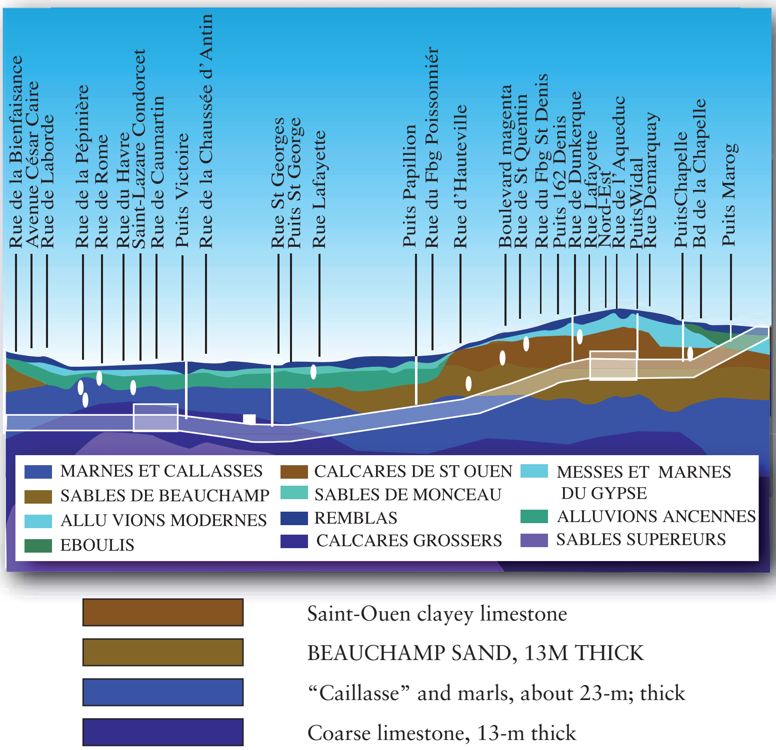


Figure 8.4 A longitudinal, geological section along the EOLE tunnel.

sequence, while the second was related to the water reservoir contained in the coarse limestone.

8.1.5 Method of excavation

The nature of the materials, the presence of water tables, and the reduced space available for installation of plants, led designers to choose the twin-tunnel solution rather than just one tunnel of a larger diameter. Consequently, the potential for risks in excavating the tunnel in a complex urban environment was considerably reduced.

In such an urban context, the most delicate aspect was face-stability control during excavation.

Due to the following reasons, a hydroshield machine was chosen:

- maintaining the counter-pressure applied at the face at prescribed levels;
- mixed face conditions;
- presence of the water table;
- possibility to operate directly in the excavation chamber for maintenance activities.

A critical aspect of using bentonite slurry is represented by difficulties to position and manage the treatment/separation plant for the slurry incoming from the excavation chamber (bentonite mixed with the excavated material).

In this case, the separation plant was located about 1 km far from the tunnel portal, and linked to it by a complex piping system, 300 mm in diameter (Fig. 8.5).

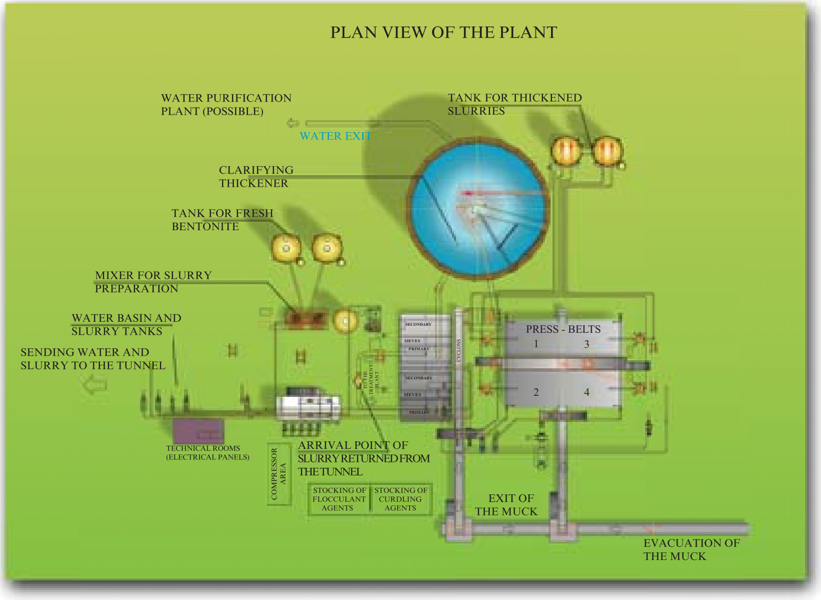


Figure 8.5 Plan view of the separation plant.

8.1.6 TBM data (Table 8.3)

Table 8.3 TBM data

Manufacturer	Voest Alpine
Model	PDS 740-OS/RM
TBM Type	Hydroshield
Cutterhead diameter	7.385 m
Power	3200 kW
Max. Thrust	60,000 kN
Max. Torque	5500 kNm
TBM Shield length	7.39 m
Back-up length	90 m

The configuration of the cutterhead of this hydroshield machine is illustrated by the photo in Fig. 8.6.

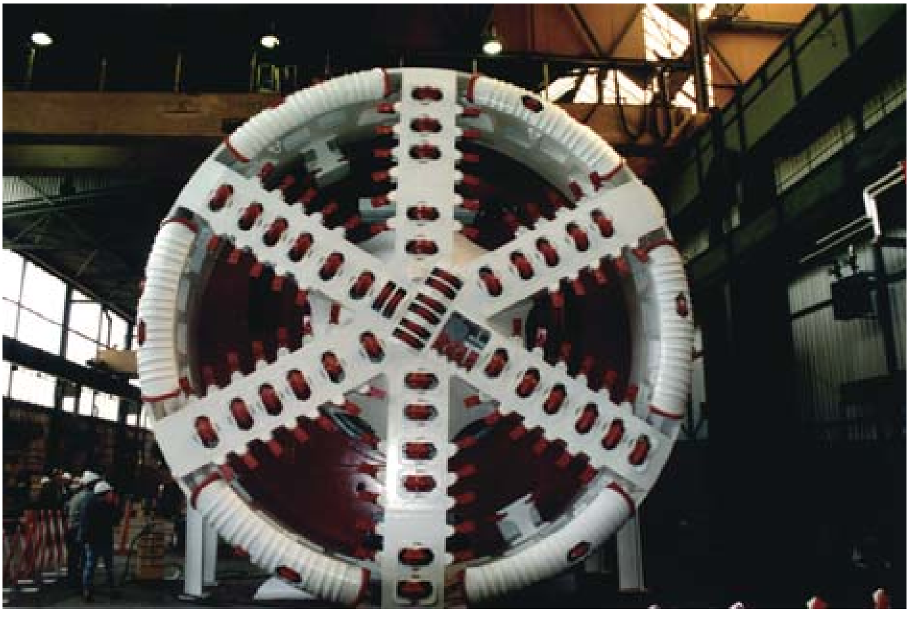


Figure 8.6 Cutterhead of the Hydroshield TBM.

8.1.7 Key factors

An important technical aspect of this project was the systematic use of probe holes during machine advancement, drilled directly through the cutterhead (Fig. 8.7).

Probe holes were executed systematically for a length of 55m while recording all data related to the drilling operation, which allowed the best calibration of advancement parameters of the machine (counter pressure values, bentonite viscosity and density, etc.).

A particular georadar was inserted in the probe hole, which permitted exploration of the ground within a radius of about 5 m from the probe hole. Operating in this way, it was possible to recognize the presence of any cavity and/or very poor material zones ahead of the excavation face.

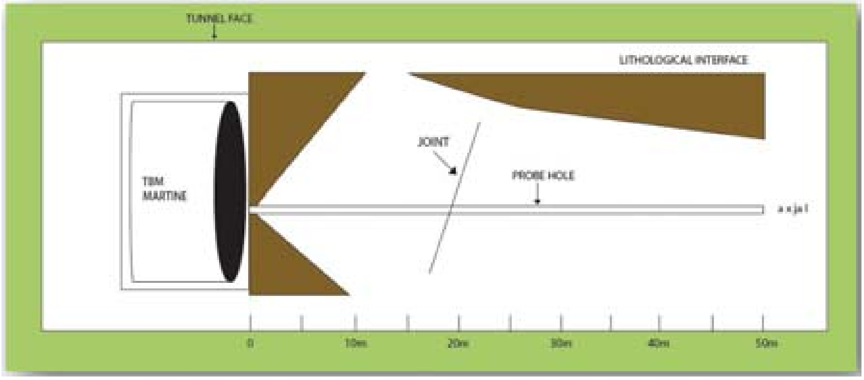


Figure 8.7 A schematic view of a probe hole and the ground conditions identified by the georadar.

The particular difficulty of this tunnelling project was related to excavating through extremely heterogeneous terrains, representing limiting conditions for the application of a hydroshield TBM, including: (1) Sticky clays; (2) Blocks "fluctuating" in marly matrix; (3) Decompression zones; and (4) Voids produced by decomposition of chalks.

However, the principal problem encountered was linked to a fortuitous event: the excavation of an experimental shaft for the realization of a connection with the planned permanent ventilation shaft.

The extremely heterogeneous formations were the true challenge of this project. During the excavation of the first bore, it was necessary to face the problems like "stickiness" of the clays in the "sables de Beauchamps" formation. The sticky-clay problem was overcome through modification of those parts of the shield machine that govern the flow of the materials, aimed at facilitating their easy passage and avoiding the accumulation of materials in particular dead-angle zones. Excavation of the other sections like the crossing of the ground known as the "marnes et caillasse" (a sort of mixture of marl, limestone, and boulders) did not confront any problems, and even the excavation of the "calcaires grossiers" (limestone up to 300 kg/cm^3) was done smoothly, thanks to the particular design of the cutterhead of the machine.

During the excavation of the second bore, when everything seemed to be known and under control, a chimney type of collapse took place, causing damages to a building under restructuring in rue Papillon. About six months were necessary to make the ground treatment and injection in order to create the necessary conditions to resume the tunnel excavation. In fact, the tunnel was subsequently excavated and completed without any further problems.

The installation on-board the TBM of a modern system to measure and record the excavation parameters allowed trace back of responsibility of the above collapse event. The measurement of the flow rate and the density of the slurry used, both inflow to and outflow from the plenum, allowed the back-calculation of the "dry quantity" of materials extracted from the plenum and compare it with the "theoretical dry quantity" of the materials excavated following the first successful applications of such a procedure in the excavation of the running tunnels of Naples metro LTR. The constructor JV together with their consultant were able to demonstrate that not even a cubic meter more of solid materials was extracted from the plenum than what was theoretically supposed to be excavated and, on the contrary, the extracted volume was less than that resulting from the excavation. Indeed, the particular area crossed by the TBM showed a strong under-compaction (thus an abnormally low density), with sands in suspension and unstable under the groundwater level. The approaching of the TBM in this zone with its vibrations caused the activation of the "natural compaction" of the sands, sucking materials from the overlying layers. The contemporary presence of an antique well, abandoned for centuries and poorly backfilled, helped to create the rest of the necessary conditions to warrant the collapse.

The excavation resumed in the same manner and with the same procedures as those used before the collapse and proceeded till the end without any further problems.

The experience gained from the EOLE project convinced the Design Consultant, Geodata, that the control of the excavation parameters could potentially give even more meaningful and important results than those obtainable from the "search of the guilty": the preliminary bases for the PAT were casted.

8.2 St. PETERSBURG METRO

8.2.1 Project particulars (Table 8.4)

Table 8.4 Project particulars

Location	St. Petersburg, Russia
Name	Metro Line 1
Construction period	2002–2003
Owner	Peterburgskij Metropoliten
General Designer	Institute Lenmetrogiprotrans (LMGT)
Contractor	Metrostroi
Sub-contractor	JV: Impregilo S.p.A. & NCC
Consultant and designer for sub-contractor	Geodata S.p.A.

8.2.2 General description

Under a design-build type subcontract, Impregilo (Italy)-NCC (Sweden) JV assembled and launched a 7.4-m diameter Voest Alpine Polyshield in 2002 to rebuild an 800 m stretch of the section between Lesnaya and Muzhestva stations of Line 1 (Fig. 8.8), which opened initially in the early 1970s and closed in 1995 when persistent water and soil inflows through the progressively damaged lining became too great to be controlled. The modified TBM, built originally for the EOLE project in Paris (see Section 8.1), had to withstand hydrostatic pressures up to 5.5 bar and pass through very complex soft ground conditions along the twin tunnels.

The original metro tunnels were excavated at about 65 m below the surface in the early 1970s using ground-freezing technique over a length of abut 500 m and with the two tubes aligned in a stacked configuration to limit the extent of the frozen zone.

After a lengthy and comprehensive study, the decision to rebuild the section of metro by excavating two new tunnels by mechanized tunnelling was taken. The depth and alignment of the new tunnels were predetermined by the fixed locations of the existing underground stations and the limits by which the alignment could be moved within the tolerances of the line and level of the metro-tunnel operations (see Fig. 8.10). Within these constraints, the alignment was moved first, as far as possible, to one side to bypass the region of ground disturbed by the original tunnel excavation and second, it was raised, as high as possible, to help reduce the potential hydrostatic pressure (at 60 m depth). Nevertheless, the invert of the new tunnels, in a side-by-side configuration, lies at a depth of about 55 m, imposing a potential hydrostatic head of 5.5 bar.

The TBM and the reinforced, bolted, and gasketed precast-concrete segmental lining were designed to meet the challenges related to the complex ground conditions. The lining was designed by Geodata for Impregilo/NCC. The cutterhead, seals, and pressurized components of the TBM were upgraded by Voest Alpine to the highest specifications.

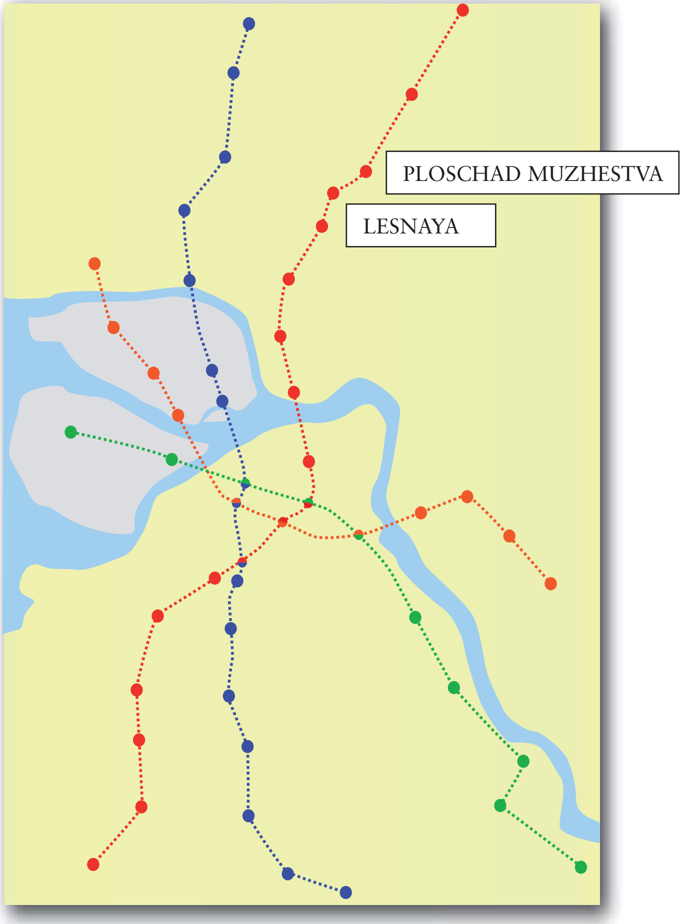


Figure 8.8 Map of St. Petersburg Metro, with Line 1 (the red line from North to South).

8.2.3 Tunnel characteristics (Table 8.5)

Table 8.5 Tunnel characteristics

Type	Two single-track tunnels
Length	2 × 790 m
Section	69.5 m^2
Diameter	7.4 m
Lining type	Segmental
Ring Type	Universal
Thickness	40 cm
No. of segments	5 + 1
Ring length	1.4 m

8.2.4 Environmental and geological conditions

Most lines of St. Petersburg Metro run below the Neva River delta in ground, largely comprised of good quality, stiff, over-consolidated, and laminated clays.

However, the section along the Red Line between Lesnaya and Ploshchad Muzhestva stations (Fig. 1) cuts across a deep and ancient glacial channel, filled with saturated glacial deposits including extremely fine loam and sands. Despite the fact that the geological and geotechnical conditions were generally well known, there remained considerable uncertainties regarding the application of a TBM. Therefore, a detailed site investigation program was commissioned to explore the potential cause of the earlier failures of the old tunnels and to provide detailed information for design and construction of the new tunnels.

A detailed geological model (see Fig. 8.9) was constructed representing clearly the stratigraphy, the variation in geotechnical properties and hydrogeology, both for the design of the tunnel lining and for an accurate prediction of the required tunnel-face-support pressures, the control of which would be critical to the success of the project.

The investigations revealed that the buried valley was at least 122 m deep and the proposed tunnel alignment would encounter low plasticity clays, silts, and fine sands, often in lenses or thin beds within the paleovalley.

8.2.5 Method of excavation

It was recognised that the safest and most economical way to reconstruct the new tunnels was to use a pressurized tunnel boring machine. Consequently, tunnel construction was carried out jointly by the Russian main contractor Metrostroi and Impregilo-NCC JV, the latter was responsible for the difficult sections of tunnel through the buried paleovalley. For this stretch the JV proposed a 7.4-m diameter Polyshield from Voest Alpine.

The machine was launched via a 70 m deep access shaft constructed close to the Lesnaya Station; it was driven to the reception chamber at the Ploshchad Muzhestva Station where the TBM was turned around to complete the second drive.

Only the central part of the new tunnel was realized by mechanized tunnelling (Fig. 8.10, the lower part, which shows also the relationship between the old and the

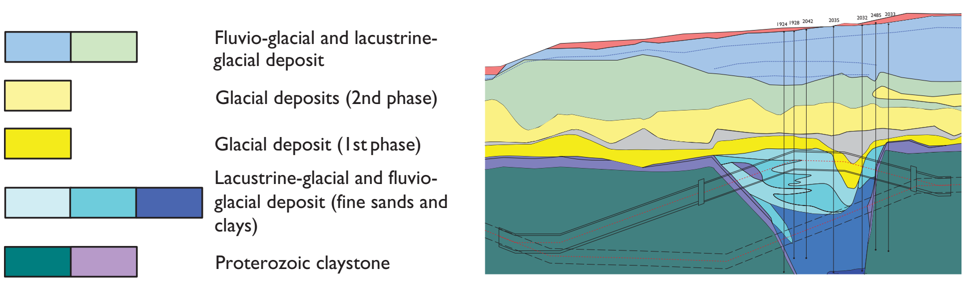


Figure 8.9 A longitudinal, geological section along the section between the Lesnaya and the Ploshchad Muzhestva station

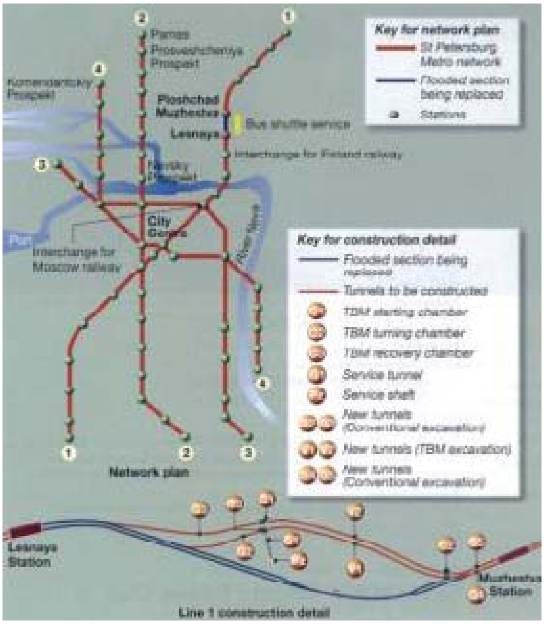


Figure 8.10 Plan view of the alignment.

new alignment). The link with the existing Line 1 was made through two connecting tunnels, excavated by the conventional method.

8.2.6 TBM data

The machine was a 7.4-m diameter Voest Alpine Slurry shield (see Table 8.6 and Fig. 8.11), built originally for the EOLE underground railway project in Paris, and refurbished to face the critical conditions of the metro tunnel. Modifications were made to the TBM for the greater hydrostatic pressures, which included a fourth set of brushes in the tail seal, an additional lip seal on the main drive bearing, a new man-lock for hyperbaric interventions, and new heavy-duty disc cutters.

The universal lining ring is comprised of five segments and a key, each with a width of 1.4 m and a thickness of 350 mm. The segments were connected using only plastic, CONEX type of dowels and featured twin gaskets with an integrated hydrophilic seal.

TBM tunnelling commenced in early February 2002 and, following early trials of the slurry system, completed the first drive to Ploschad Muzhestva somewhat later than anticipated in early May 2003. The return drive benefited greatly from the experience of the first tunnel and, by contrast, took just three months, breaking through on the 27th of November 2003.

The TBM performed extremely well in the onerous operating conditions, but there were no problems with either the slurry pressure system or the tail-void-grouting system. The tail seal and main bearing seals all performed satisfactorily.

Since the tunnel lining technology was new to Russia, and there remained some understandable unease because of the performance of the earlier tunnel lining within

Table 8.6 TBM characteristics

Manufacturer	Voest Alpine
Model	PDS 740-OS/RM
TBM type	Polyshield
Cutterhead diameter	7.385 m
Power	320 kW
Thrust (max)	60,000 kN
Torque (max)	5500 kNm
TBM Shield length	7.395 m

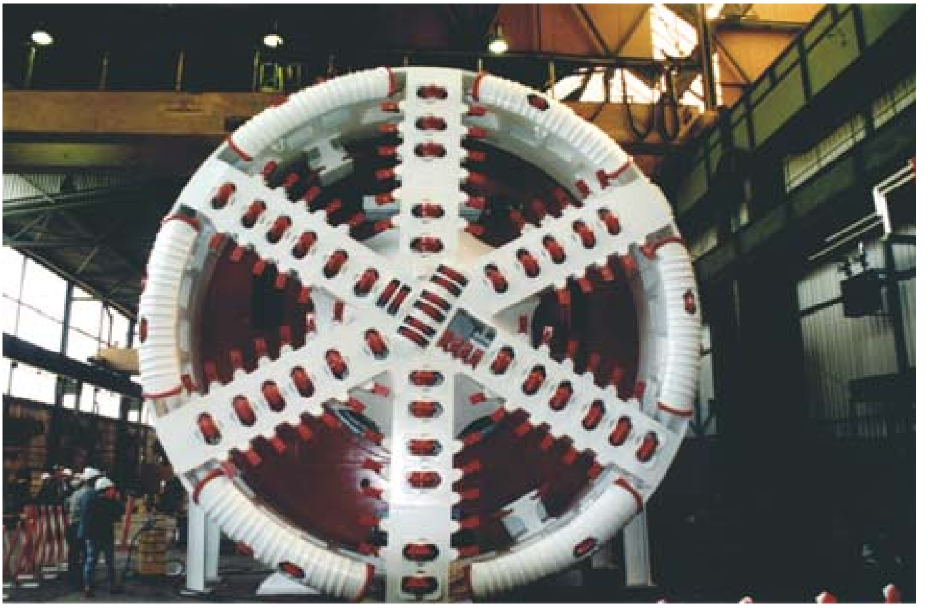


Figure 8.11 Cutterhead of the Hydroshield TBM (the same of Eole Project).

the buried valley, the precast segmental lining was extensively instrumented. Ten rings in each of the two tunnels were equipped with vibrating-wire strain gauges installed during manufacturing of the segments. The joints between the segments were also precisely monitored to observe their behaviour and the effectiveness of the gaskets. Measurements indicated that the lining remained in compression and that the stresses developed were well within the design values. The joints remained closed and the tunnel lining stayed watertight. In addition, a total of eighteen piezometers were installed to monitor pore-pressure response to slurry pressure. Results indicated that the pore pressures increased and then fell back to equilibrium values as the TBM passed.

In order to provide relevant reference values for controlling the TBM excavation process through real-time monitoring, extensive and parametric numerical analyses were conducted to understand the interaction of the excavation with the surrounding ground including those at and ahead of the face (see Fig. 8.12, for example).

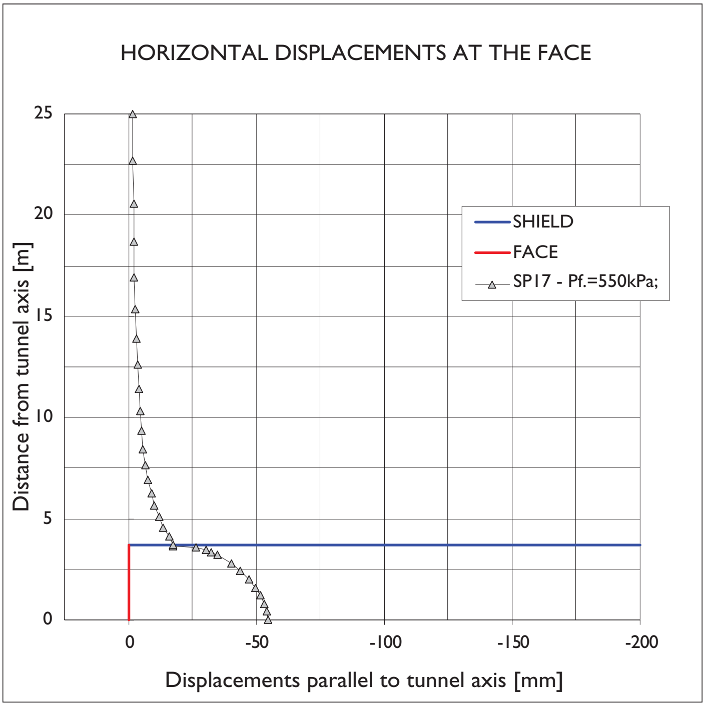


Figure 8.12 Horizontal displacements at the face.

8.2.7 Key factors

Design phase:

- Tunnel face stability.
- Very critical geological conditions.
- Project contraints.
- The segmental lining performance.
- Settlement prediction.

Construction phase:

- Tunnel construction and TBM monitoring.
- Monitoring system.
- Post-construction investigation.

8.2.8 Risk management

The Risk Management Plan (see Appendix 5).

Monitoring system with application of GDMS (see Section 6.4 and Figs. 8.13 and 8.14).

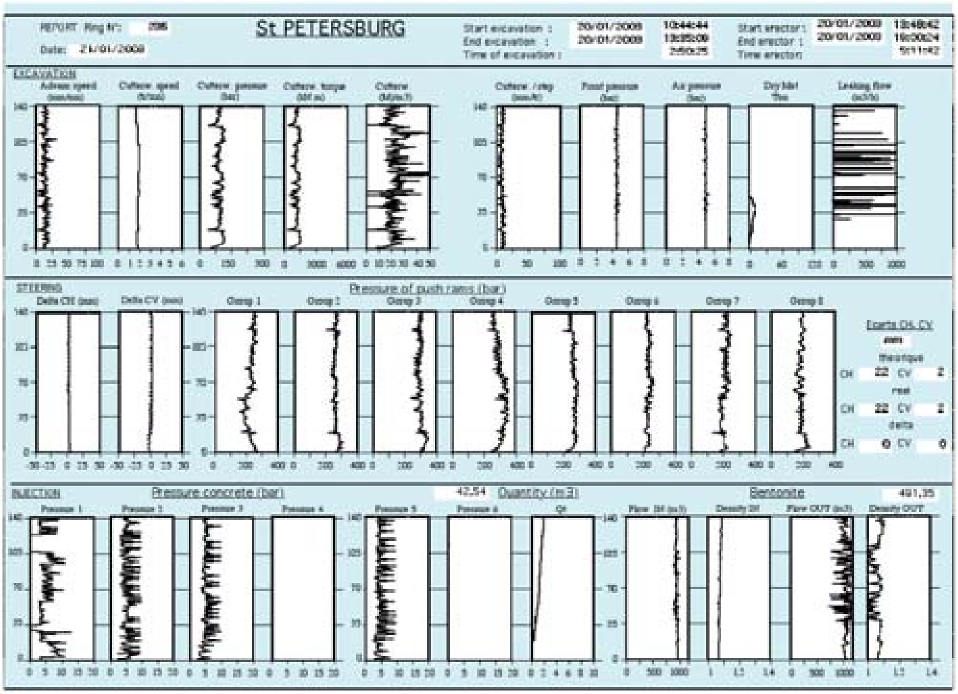


Figure 8.13 Example of a record excavation parameters of the Hydroshield in one stoke.

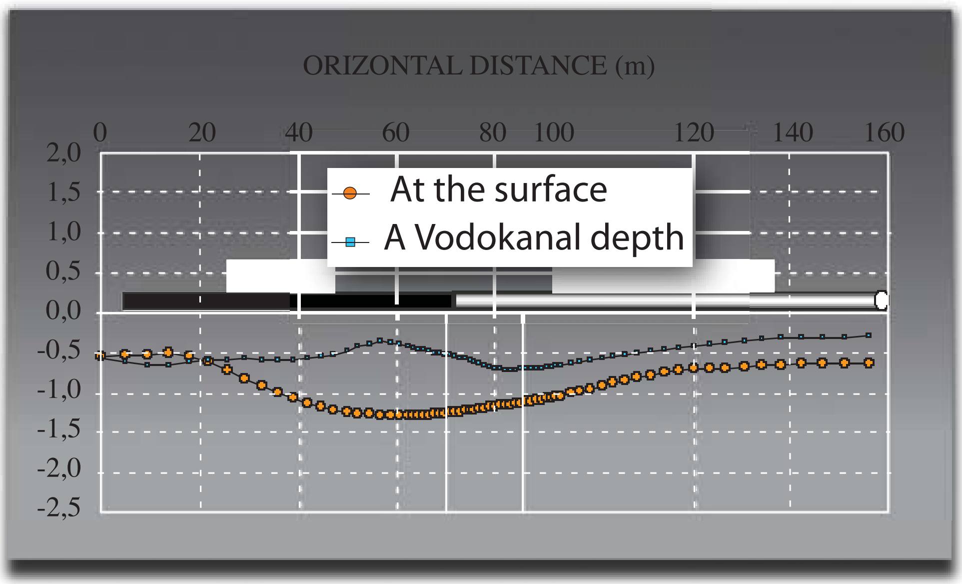


Figure 8.14 Example of the settlement measured at different depths.

8.3 PORTO LIGHT METRO

8.3.1 Project particulars (Table 8.7)

Table 8.7 Project particulars

Location	Oporto, Portugal
Project period	2000–2003
Name	Line C –Line S
Owner	Metro de Porto
General Contractor	NORMETRO J.V. Soares da Costa, Somague & Impregilo for the civil Works, and Transmetro
Geodata activity	Geological-geotechnical characterization, tunnel design, building risk assessment, resident Engineering Services

8.3.2 General description

The Porto Metro is a 70 km light rail system centred on Porto, composed of 20 km of new line and 50 km of existing line (see Fig. 8.15). The underground part of the metro consists of two tunnels (Line C, 2.5 km, and Line S, 4 km) driven with two Herrenknecht EPB Shields with diameters of 8.7 m and 8.9 m, respectively. The two tunnels serve 10 underground stations within the city center (see Fig. 8.16 and Fig. 8.17).

Figure 8.15 The general layout of the Metro of Porto.

Figure 8.16 The central part of the metro system of Porto.

The civil works were managed by the TRANSMETRO JV of Soares da Costa and Somague of Portugal and Impregilo of Italy, who chose to excavate the tunnels with the use of two EPB machines.

Geodata of Italy was contracted by the JV for the geological-geotechnical characterization, tunnel design, building risk assessment and, in JV with Mott McDonald of England, for Resident Engineering Services during construction.

8.3.3 Tunnel characteristics (Table 8.8)

Table 8.8 Tunnel characteristics

Total Tunnel Length	Line C: 2.5 km
	Line S: 4.0 km
Overburden (min-max)	3–30 m
Finished internal diameter	Line C: 7.8 m
	Line S: 8.0 m
Excavation section	Line C: 60 m^2
	Line S: 62.7 m^2
Lining type	Segmental lining
Ring Type	Universal
Thickness	30 cm
No. of segments	6 + 1
Ring length	1.4 m

Figure 8.17 View of the Aliados station before TBM breakthrough.

8.3.4 Environmental and geological conditions

The ground is composed of a course granite belonging to the formation of "Granito do Porto" (see Fig. 8.18). Alluvial material is often found above the weathered granite due to the presence of several water courses. The granite is weathered to different grades, from fresh rock to residual soil, showing highly variable geotechnical conditions (see Fig. 8.23). The weathered granite locally exhibits a metastable structure, which can accentuate the potential for collapse, depending on the high porosity and the reduced cohesive strength of the loosened/leached residual soil. Consequently, the ground tends to follow an elastic-brittle-plastic behaviour, leading to sudden failures at the surface with practically no warning signals if the tunnel face is not properly supported or if uncontrolled over-extraction is allowed.

The groundwater table follows roughly the shape of the surface topography. A large number of old wells and "minas" (old and small hand-excavated water tunnels) are present in the area. These wells and "minas" influenced the hydrogeological

Figure 8.18 The rock core of Porto granite—an example.

characteristics of the ground, so the groundwater moves not only in the porous medium and fractures but also along the preferential channels represented by the "minas".

The running tunnel and underground station construction interferes with a densely populated urban environment, with more than 2000 buildings in the influence zone, including many important and historical buildings such as the Town Hall. The minimum overburden of 3–4 m was prevalent in two sections of the tunnel, located in the historical centre of the city, where the TBM had to be driven underneath sensitive buildings. The water table was located at between 10 m and 25 m above the tunnel.

8.3.5 Method of excavation

Excavation of Line C Tunnel first began in June 2000, but was soon interrupted due to three major collapses in October 2000, December 2000, and January 2001. In order to overcome these initial difficulties, which led to 9 months of TBM stoppage, a new integrated approach was adopted in 2001 for both the Design and the Construction phases.

The Line C tunnel was completed in October 2002. Excavation of Line S tunnel began in June 2002 and was completed successfully at the end of October 2003.

Due to the extreme variability and unpredictability of geological conditions and due to surface constraints, the special requirement to operate the EPB always in closed mode was imposed, when the excavation was resumed in September 2001.

Most stations were constructed by the cut & cover method (see Fig. 8.19 and Fig. 8.22, for example).

Figure 8.19 An example of open excavation for station construction.

8.3.6 TBM data (Table 8.9)

Table 8.9 TBM data

Manufacturer	Herrenknecht
TBM type	EPB
Cutterhead diameter	8.740 m
EPB pressure (max)	3 bar
Thrust (max)	70613 kN
Torque (max)	12900 kNm
Penetration rate (max)	80 mm/min
No. of thrust jacks	36
No. of grouting lines	6
No. of injection lines	6

Both TBM were assembled and launched from an open trench (see Fig. 8.20 and Fig. 8.21, for example) which corresponds to the ramp approaching a station.

Figure 8.20 The Campanha Portal.

Figure 8.21 The cutterhead of the first Herrenknecht TBM in Porto.

Furthermore, after the accidents that occurred during the excavation of the first 500 m and due to the very strict safety requirements, the following special modifications were made to the original machine configuration, leading to a higher degree of safety and a better overall performance:

Set up of an active Secondary Face-Support System (SFSS), which always guarantees the appropriate face support, by means of bentonite slurry that is automatically pumped into the excavation chamber whenever the pressure drops below a certain level.

Installation of an Emergency Double-Piston Pump (EDDP) after the screw conveyor in order to deal with the liquid muck and the uncontrollable support pressure oscillations.

Installation of a second balance under the conveyor belt in order to cross-check the muck weight measured by the first balance.

8.3.7 Job site organization and references

Average performances:

average daily production:	5 rings = 7 m
best day (06.02.2002):	13 rings = 18.2 m
best week (7–13 October 2002):	56 rings = 78.4 m
best month (May 2002):	148 rings = 207.2 m

Working time:

The TBM was normally operated on a 24-hour basis in a 6-day week.
Almost one shift was used daily to execute the cutter-head maintenance under hyperbaric conditions.

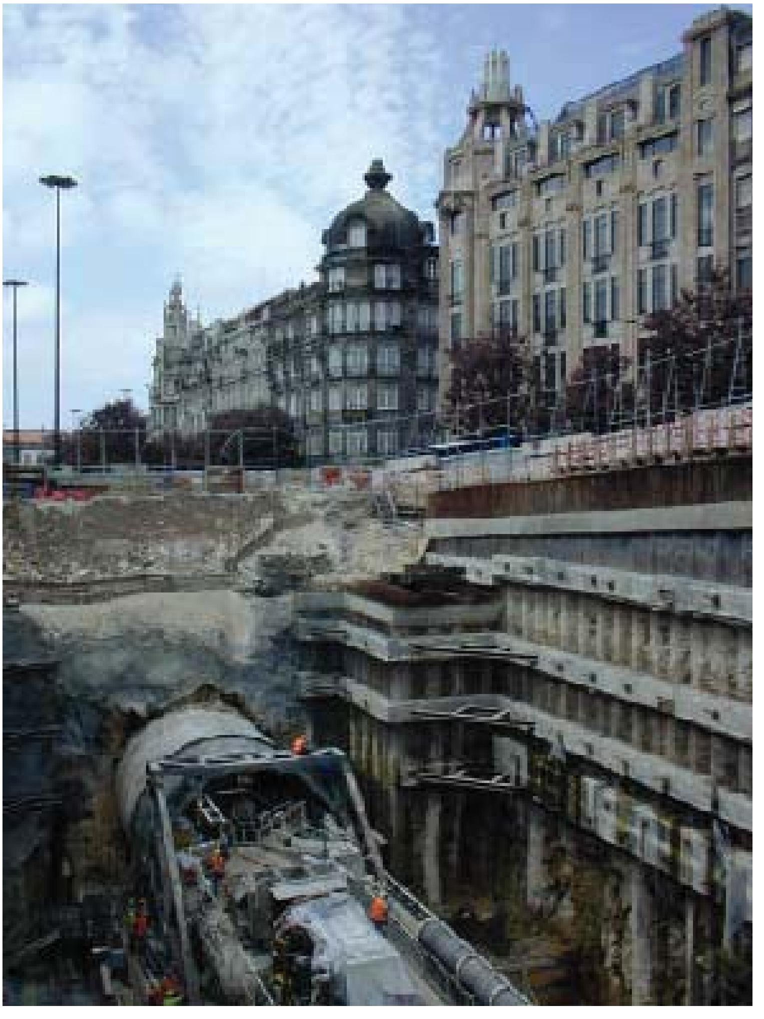


Figure 8.22 Alliados station.

8.3.8 Key factors

After the initial accidents, three key factors were identified as essential for the re-start of the tunnelling and the successful completion of the works:

Production of detailed Plan for Advance of Tunnel (PAT) for each stretch of the tunnel, so that all parameters and design issues related to tunnelling are effectively addressed prior to the actual excavation of that stretch.

Implement working procedures covering all of the tunnelling phases to ensure that the TBM operations are carried out in a controlled and safe manner.

Set up of an integrated follow-up team between the Contractor and the Designer to manage the design and construction process.

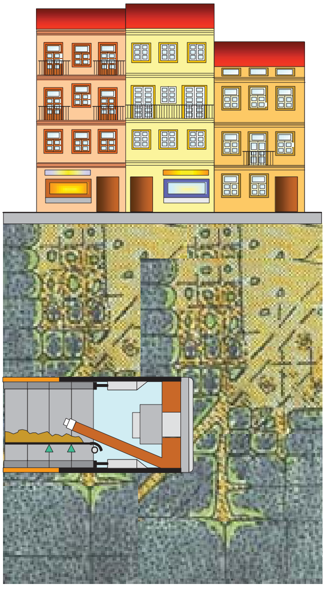


Figure 8.23 A conceptual view of the typical granite in Porto encountered during excavation.

The design approach—The Plan for Advance of Tunnel (PAT)
The design addressed the following main issues:

- definition of the correct working parameters of the TBM to minimize the face and volume loss;
- estimate of the extent and shape of the foreseeable settlement trough;
- evaluation of the acceptable deformation limits of the buildings, and
- definition of both the preventive and the remedial measures.

To deal in detail with all these subjects in Porto, the design method PAT, was related to tunnel stretches of short length—200 m to 1 km—and included the following separate documents:

- report on the geological investigation campaign and its interpretation;
- report on the building risk assessment;
- report and drawings about the monitoring of underground structures and surface buildings (see, for example, Fig. 8.27 and Fig. 8.28);
- report on the evaluation of the TBM's working parameters;
- geotechnical profile with indication of the TBM's working parameters, and
- summary report on the PAT.

Towards the end of each stretch, the experience gained was summarized in specific back-analysis documents that helped to optimise the subsequent stretch of the tunnel. Thus a process of continuous enhancement was implemented.

It is worthwhile to highlight the following points:

1. In addition to the "traditional" design information such as geology evaluation, structural calculation, etc. a specific set of TBM working parameters was determined for real-time monitoring, see Figs. 8.24–8.26. In particular, the "Report on the evaluation of the TBM working parameters" contained the definition of the reference value and the relevant operational range of the following parameters:

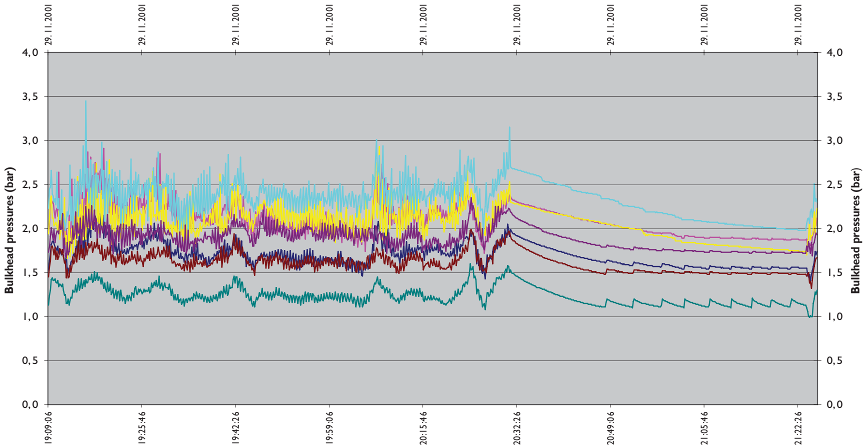


Figure 8.24 Graphs showing the face-support pressures measured at different levels of the plenum.

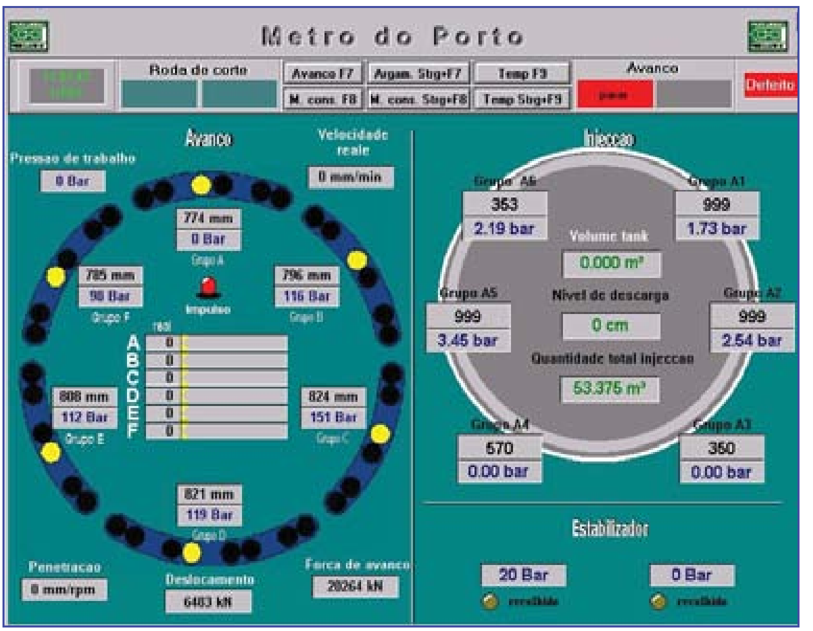


Figure 8.25 Frame 1 of machine operator's screen showing various control parameters.

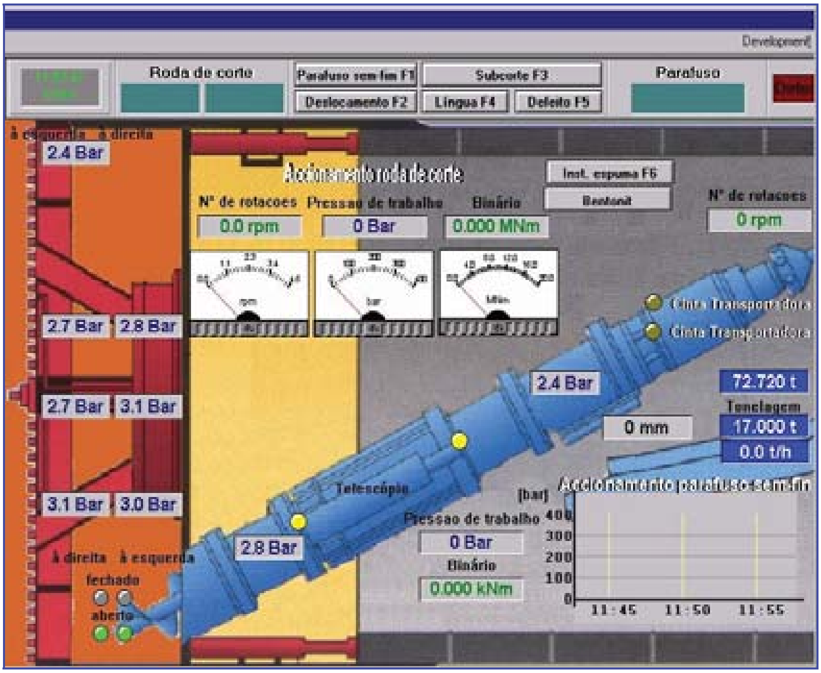


Figure 8.26 Frame 2 of machine operator's screen showing additional control parameters.

Face-support pressure.
Apparent density of the muck in the chamber.
Weight to be extracted at each ring.
Longitudinal-grouting pressure and volume.
Additional bentonite-slurry injection volume and pressure.

2. The TBM working parameters were summarized from the PAT and delivered to the TBM crew in a very simple form called "Excavation Sheet".
3. Following the real-time and back-analysis activities, the Excavation Sheets were continuously updated, based on the actually encountered conditions, so that the PAT could be regarded as a live document, always kept up-to-date.
4. The implementation of the PAT and its continuous updating has proved to be a very effective tool, as the geological conditions and the design parameters for the TBM are given in advance together with the instrumentation and the monitoring requirements.

8.3.9 TBM working procedures

In order to ensure that the TBM operations are carried out in a consistent and controlled manner, it has been necessary to give the TBM Crew specific information about the EPB operational criteria to ensure that the TBM Crew members fully understand the EPB working modes and the impact of the tunnelling on the surrounding environment (ground and buildings).

For this purpose a careful and detailed preparation of working procedures has been carried out—jointly by the Contractor and the Designer—covering the following critical activities:

- advance and face support (standard practice to always maintain the adequate face support;
- pressure as well as exceptional measures to deal with anomalous situations;
- primary longitudinal grouting;
- secondary radial grouting;
- lining ring erection;
- repair of damaged segments;
- probe holes in advance;
- cutterhead maintenance intervention (including hyperbaric works), and
- calibration of weight scales.

The procedures contain practical instructions about the actual manoeuvres to be made by the operators for a correct tunnelling operation; they also include the flow of information to be activated in case of problems.

Approved by the Tunnel Manager and the Resident Engineer, the procedures are introduced and explained to the Shift Engineers, TBM pilots, and Shift Foremen during regular introduction briefings. In many cases, these procedures have also been revised and updated to take into account the experience gained in the previous stretches of the tunnel.

After an initial period of reluctance, the working procedures were fully applied, proving to be very useful in reducing the occurrence of wrong operations due to "human errors".

PORTO METRO
BACK-ANALYSIS OF THE MEASURES
PAT: **3-4** **Section:** Pk 14 20 **Geological condition: A**

DATA				SUBSIDENCE BASIN			
Tennel section	A	[m]	59,4	Inteference of displ. basi	i	[m]	15,20
Overburden from tunnel axis	z	[m]	19,0	Limits inf. e sup. de	li	[m]	-38
Value K	**k**	[-]	**0,8**	Sagging zone	ls	[m]	38
Loss volume	**Vp**	[%]	**0,7**	Basin volume	Vs	[m³/m]	0,4158
Far from tunnels axises	D	[m]	-	Max displacement	S max	[m]	0,01091

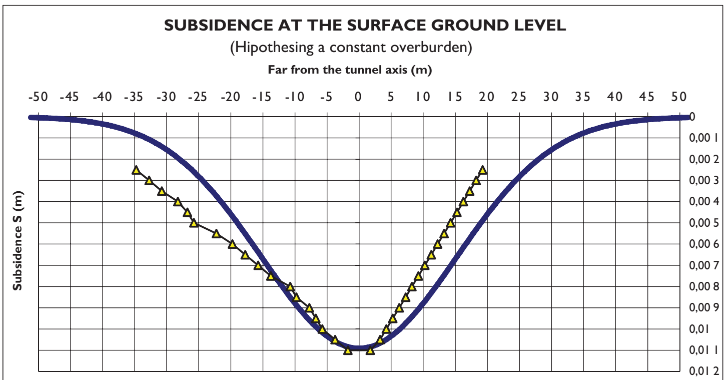


Figure 8.27 Theoretical vs. actual subsidence at the surface.

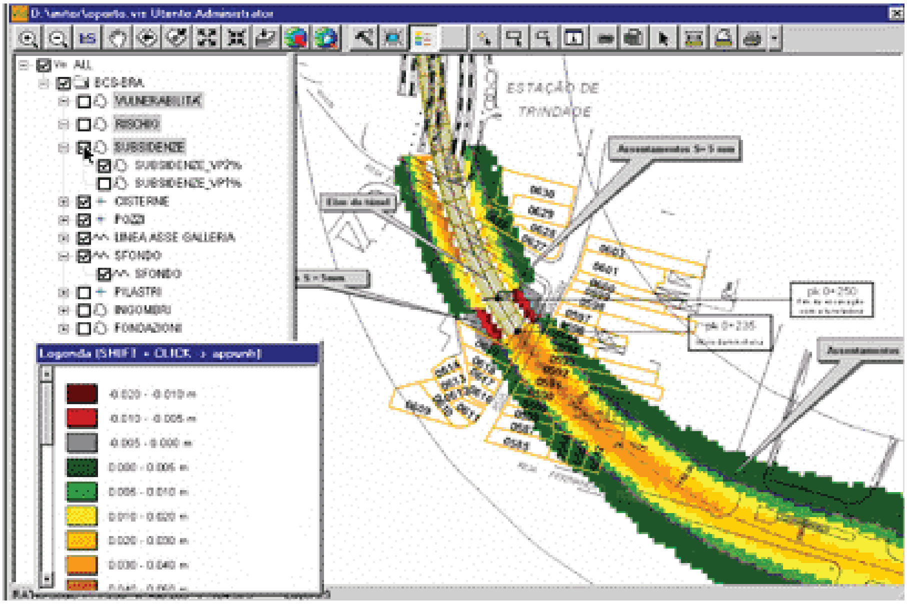


Figure 8.28 A stretch of the tunnel showing the zones of influence of excavation on surface subsidence.

8.3.10 Integrated TBM follow-up team and day-by-day supervision activity

The works were managed and supervised by a team composed of specialists, from both the Contractor and the Designer, who fully integrated their skills with the aim to achieve the highest TBM performance, conforming with the highest safety and quality standards, as well as the overall cost optimisation. The Designer's representatives acted as a part of the Resident Engineering staff, whose task was to provide the design, supervision of the works, and the specialist assistance on site. The Contractor's representatives focused on the production and site organization issues, whereas the Designer's representatives focused on the supervision of the works (i.e. mainly to guarantee the safety and the quality of the works with respect to the contractual and design obligations) and – what turned out to be very useful for the success of the project – to the continuous interpretation of the interaction between the TBM and the ground.

The task of the designer's representatives can be summarised as follows:

The geotechnical engineer acted as Design Manager, providing the Geotechnical Design and the Risk Assessment.

The TBM Engineer carried out the continuous analysis and interpretation of the data automatically recorded by the TBM data-logging system; in practice, he studied the TBM "behaviour", even recognising the encountered ground conditions and, therefore, allowing an effective forecast of the influence of tunnelling on the surrounding ground and buildings. After each stretch of tunnel, the engineer prepared a specific TBM performance back-analysis report, providing indications with regard to deficiencies of the system and the relevant foreseeable improvements.

The monitoring engineer managed the overall monitoring system including geotechnical deep and surface instrumentation, as well as building instrumentation; in parallel with the TBM engineer, he provided the indication of the ground and building response to the tunnelling activity. He reviewed the frequency of the readings according to the necessity and finally issued the relevant back-analysis.

The geologist provided the detailed geological characteristics of the rock mass, both at the design stage and during the construction. Through the daily face-mappings and the interpretation of the probe holes in advance, the geologist was able to adjust the forecasted geology with great accuracy, thus allowing the optimisation of the input data for defining the TBM-working parameters. Due to the necessity of work under hyperbaric conditions, two geologists (one from the Contractor and one from the Designer) were present on site, hence, guaranteeing daily face-mappings under any circumstances.

In addition to the follow-up team described in this case history, a design team of up to 10 individuals (including engineers, geologists, and draftsmen) was necessary to prepare and issue the actual design documents.

8.4 TURIN METRO LINE I

8.4.1 Project particulars (Table 8.10)

Table 8.10 Project particulars

Location	Turin, Italy
Construction period	2001–2006
Name	Turin Metro Line 1 (Tratta Deposito–Porta Nuova)
Owner	GTT S.p.A (Turin Transport Groups)
Designer(s)	Systra (France), GEODATA (Italy)
Contractor	Grandi Lavori Fincosit, Grassetto, Seli, Rodio, Co. Ge. Fa. and V.I.P
Engineer(s)	Systra & Geodata

Figure 8.29 The layout of the first section of Turin Metro Line 1.

8.4.2 General Description

Turin Metro Line 1 is part of the public transportation development plan of Turin and its suburban areas. It is an automated, driverless Metro based on the VAL system (Véhicule Automatique Léger). The VAL system has been operating successfully in some French cities (Paris, Lille, Toulouse and Rennes) for almost twenty years, but it is running for the first time in Italy. Line 1 is divided into 4 sections constructed in different phases:

The first section is 9.5-km long and is in operation. It extends entirely underground from the town of Collegno, west of Turin, developing along Corso Francia and ending at Porta Nuova railway station.

The second section is 3.7-km long. It stretches from the Porta Nuova station to Lingotto area including 6 stations. It is expected to be completed in 2009.

The third and forth sections will be the West and South extensions of the line to the suburb areas.

Figure 8.30 TBM-bored tunnels of Turin Metro supported by the segmental lining.

The construction of section 1 was started in November 2000 and finished in December 2005.

A single circular tunnel of 6.8-m diameter contains the double-track line and was bored by TBMs, except for the Deposito to Fermi stations interval, which was constructed using the cut and cover method. The TBM-bored tunnel was lined using pre-cast reinforced concrete segments (Fig. 8.30).

The 15 stations that are approximately 17-m wide and 56-m long, with lateral platforms were built by the cut and cover method. The average depth of the platforms level is about 17 m (see Figs. 8.31 and 8.32).

Figure 8.31 Excavation of a station from the surface.

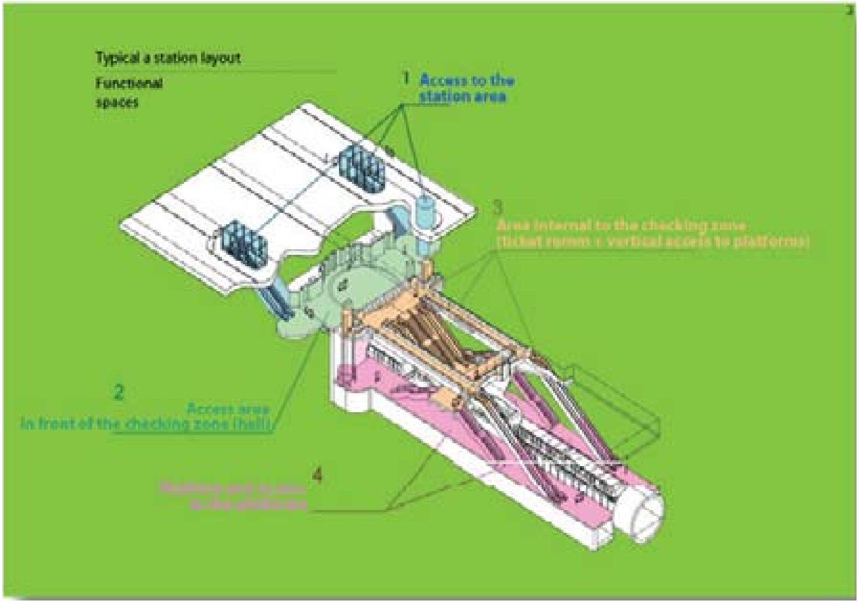


Figure 8.32 Typical layout of a station in Turin Metro.

8.4.3 Tunnel characteristics (Table 8.11)

Table 8.11 Tunnel characteristics

Total Tunnel Length	8 km
Boring diameter	7.8 m (Lots 3 & 4)
	8.0 m (Lot 5)
Overburden (min–max)	3–30 m
Lining type	Segmental lining
Ring type	Universal
Thickness	25 cm (Lots 3 & 4)
	30 cm (Lot 5)
No. of segments	6 + 1
Ring length	1.5 m
Connectors	Bolts

8.4.4 Environmental and geological conditions

Approximately 80% of Turin city lies in a semi-flat plain formed by successive alluvial fans at the end of the Alpine valleys, the Dora Riparia and the Stura di Lanzo rivers. These fans consist of fluvial-glacial deposits, remodelled, at least in the more superficial levels, by the waterflows that cross the area.

The first section of Line 1 was completely excavated in the upper part of the fluvial-glacial and fluvial deposits (Figs. 8.33 and 8.34). These deposits present horizontal and vertical discontinuous levels (lens) with different grain size distributions and varying degrees of cementation; the levels with greatest cementation (conglomerate levels) are typical of the Turin subsurface and are known as the "puddinga" formations.

From the hydrogeological point of view, the subsurface of Torino is composed of a system of overlapping strata in which the presence of the first superficial water table is evident. Problems of interference between the running tunnels and the flow of ground water arises in the central stretch of Section 1.

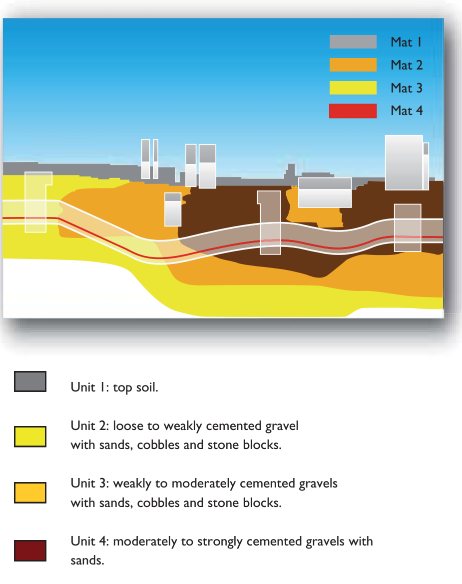


Figure 8.33 Four geotechnical units were recognized along the tunnel profile.

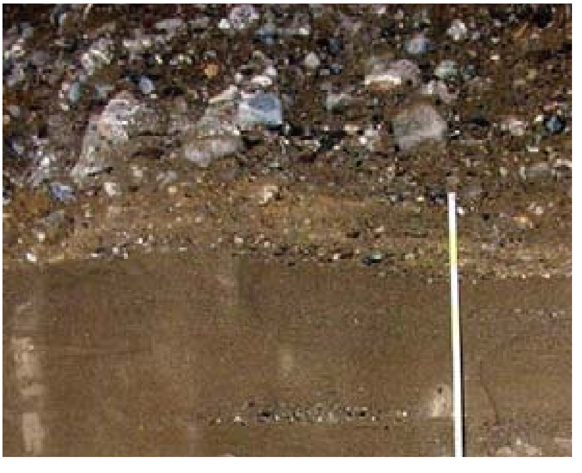


Figure 8.34 Soil stratum of the Turin Metro.

8.4.5 Method of excavation

The running tunnel construction works were divided in the following Lots (Fig. 8.35):

Lot 3: from Fermi station to Pozzo Strada station (approximately 3600 m)
Lot 4: from Pozzo Strada station to Principi di Acaja station (approximately 2650 m)
Lot 5: from Principi di Acaja station to Porta Nuova station (approximately 2900 m)

The tunnels of Lots 3, 4 and 5 were excavated by EPBS (Earth Pressure Balance Shield). A comparison between the feasible use of either EPBS or SS (Slurry Shield) was carried out based on the geological conditions and arbitrary responsibility of the contractor and ultimately the EPBS was chosen. The TBM's specifications are given in Table 8.12.

Figure 8.35 The division of Section 1 into Lots 3, 4, and 5.

Figure 8.36 EPB Shield manufactured by LOVAT.

The TBMs used for Lots 3 and 4 are new machines from LOVAT (see Fig. 8.36). The TBMs used for Lot 5 is a second-hand machine refurbished by NFM, which has been successfully used in the excavation of Milan Railway bypass and, subsequently, for the excavation of a railway tunnel near Calalzo, Italy (the Monte Zucco Tunnel).

8.4.6 TBM data (Table 8.12)

Table 8.12 TBM data

Manufacturer	LOVAT (Lots 3 & 4), NFM (Lot 5)
TBM Type	EPB RME 306 Series 20600 (Lots 3 & 4)
	EPB Mod. 1331056 (Lot 5)
Power	2100 kW (Lovat)–2000 kW (NFM)
Thrust (max)	76000 kW (Lovat)–2000 kW (NFM)
Torque (max)	20400 kNm (Lovat)–15000 kNm (NFM)
Shield length	10 m (Lovat)–9.1 m (NFM)
Back-up length	98 m (Lovat)–100 m (NFM)

8.4.7 Job site organization and TBM performances

Each TBM drive involved two shafts: one for lauching and one for receiving the TBM, see Fig. 8.38, for example; and each TBM had quite a few breakthroughs at the various stations along its path.

Average excavation rates (see Fig. 8.37 and Fig. 8.39).

Lot 3 – Lovat TBM – 7.4 m/d = 220 m/month
Lot 4 – Lovat TBM – 10.0 m/d = 300 m/month
Lot 5 – NFM TBM – 7.7 m/d = 230 m/month
Advancing speed (best day): 37.5 m

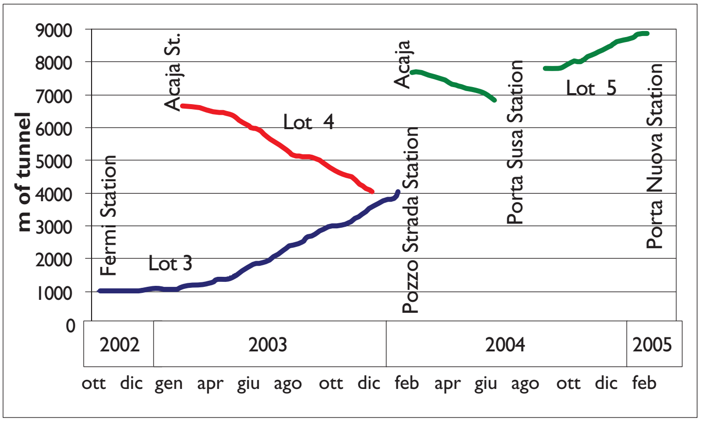


Figure 8.37 Completion period of the Lots 3, 4, and 5.

Figure 8.38 EPB reached a receiving shaft.

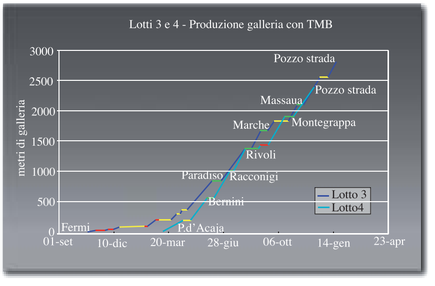


Figure 8.39 TBM production of Lots 3 and 4.

8.4.8 Key factors

The construction of the tunnels encountered a series of severe difficulties and risks, mainly due to the delicate urban context within the working area, which required particular attention to be paid to both the design and the construction aspects.

8.4.8.1 Risk assessment

Design phase

Underpassing important buildings upto 5–6 floors in height.

Passing of the TBM excavation below large infrastructures that are sensitive to settlements (underground railway line, important monuments like the 16th century Cittadelle defense tunnels, etc).

Construction phase

Particular attention to the public safety during construction along the main streets of the city.

Active support of the excavation face and cavity.

The particle size distribution of the ground is at the limit of the current TBM technology (see Fig. 8.42).

Presence and management of large rock boulders in the fine matrix.

Wear of the cutting tools (due to a high quartz content)

Time factor

The necessity to complete the construction of the metro line before the Turin Winter Olympic Games, held at the beginning of 2006, was a significant constraint.

The risks

Damage to properties.

Damage to surface infrastructures and subsurface utilities.

Potential delay in completion was identified, quantified, and managed following the principle of ALARP (As Low As Reasonably Practicable).

8.4.8.2 Risk management

The management of the identified risks was facilitated by the implementation of a GIS-WEB based monitoring system, both for the design and the construction phase (see Fig. 8.40).

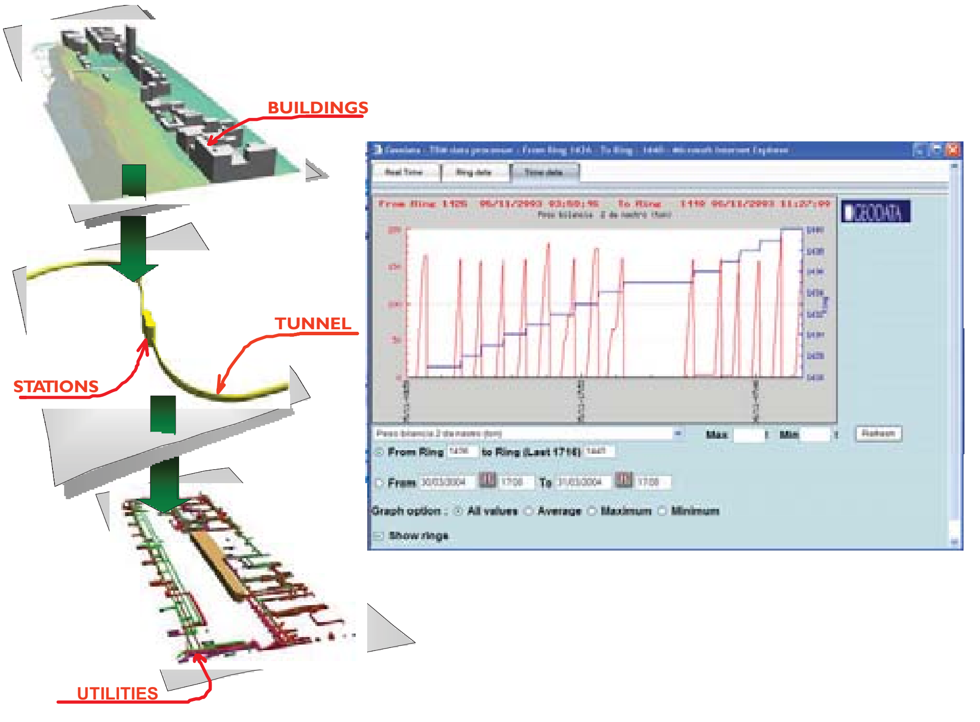


Figure 8.40 An example of Integrated Monitoring System in Turin Metro Project.

GDMS application

Management of all site investigation and design data.

Real-time availability of thousands of geo-referenced monitoring observations, accessible via WEB for checking:

- the advancement of each TBM excavation;
- the settlements on the surface, and
- the TBM performance parameters.

All of the above information is fundamental to the engineer to make decisions for a better execution of the project.

A sophisticated monitoring system composed of more than three thousand instruments was installed in order to control the construction works.

Monitoring system

The monitoring system facilitated a continuous comparison between the on-site situation and the design predictions. Furthermore, predefined actions and countermeasures were associated with the monitored parameters to manage potential, critical situations for guaranteeing the safety of people and structures.

According to the design the following parameters were monitored and recorded:

- stresses, strains, and displacements in the underground structures;
- strains at the surface and at depth;
- environmental data (such as groundwater level);
- displacements of existing buildings and artefacts (see Fig. 8.41), and
- excavation parameters from Tunnel Boring Machines.

The continuous monitoring of important historical buildings, located above the running tunnels, was performed by the installation of optical targets for 1D and 3D surveys, electrolevels for the assessment of the angular distortions, and triaxial vibrometers for the vibration monitoring during tunnel excavation, using the same reading frequency as utilized for the tunnel monitoring equipments.

Special soil conditioning

Many tests were conducted to study the soil-conditioning problems of the Turin soils (Lot 5). From the result of these tests it was clear that the injection of a combination of foam and polymers was not enough to produce a plastic, non segregating muck. As a consequence, it was decided to study the alternative to complement the foam and polymers conditioning with the injection of a fluid mix containing rock powders.

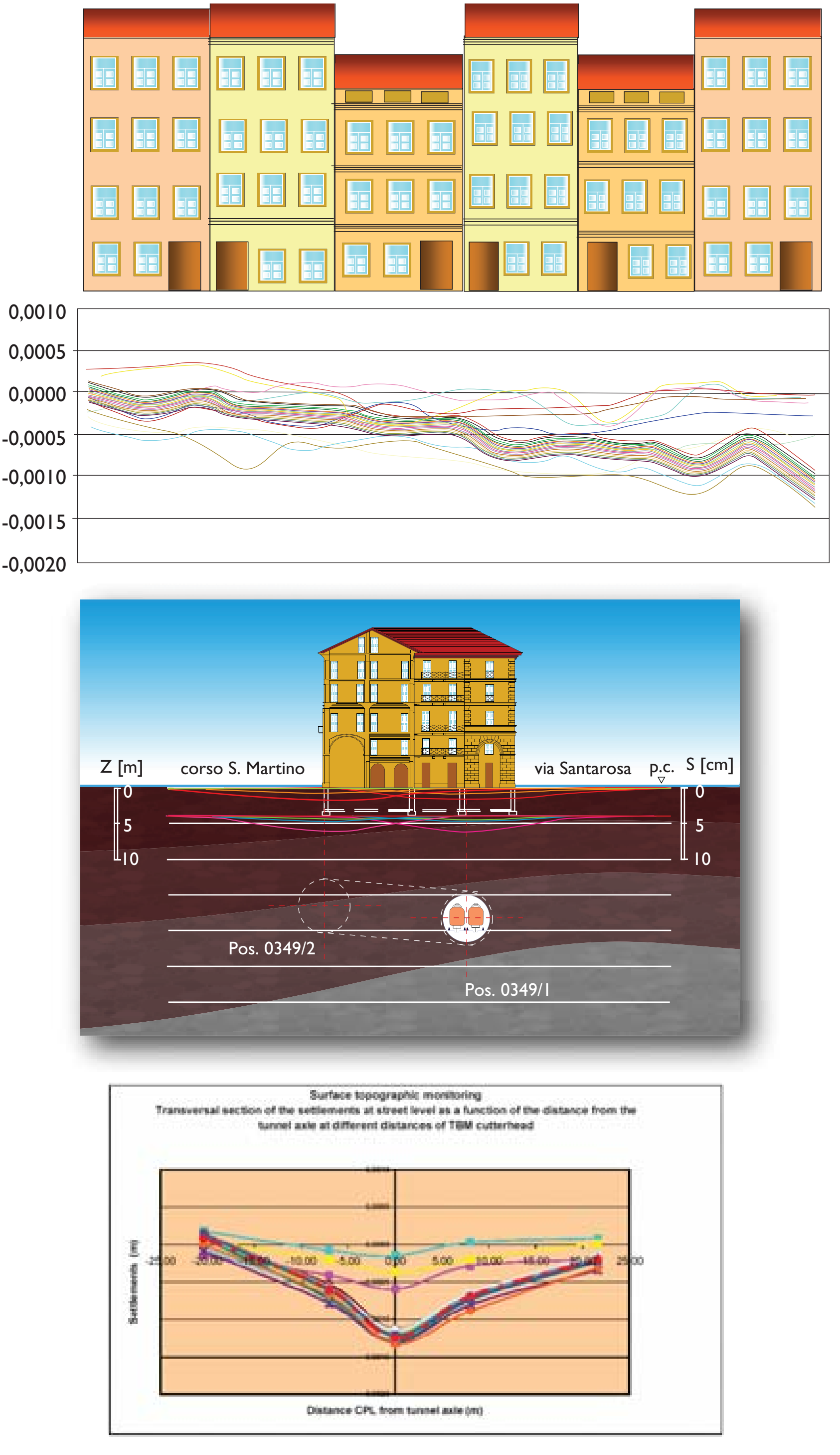


Figure 8.41 Induced settlement monitoring for surface structures in Turin metro project.

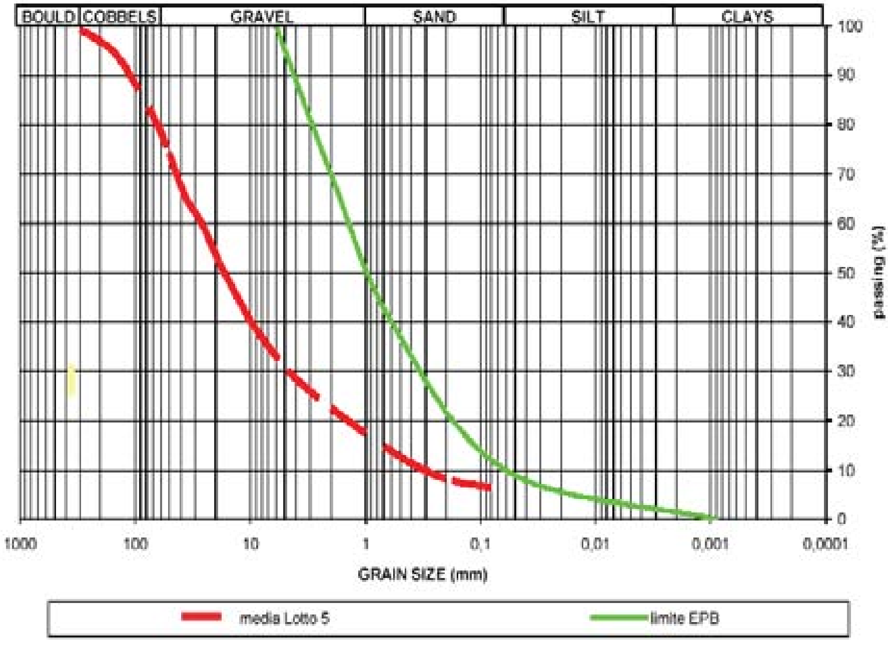


Figure 8.42 Soil particle distribution for Turin metro project.

In order to produce a stable mix, the following main aspects were considered:

- A filler having a fineness modulus and size for "closing" of the curve of the natural alluvial material.
- A polymer able to stabilize the mix and satisfy the environmental aspects at the disposal area.
- A viscosity of the mix suitable to be pumped through the small conduits of the cutterhead rotary joints.

Ground Consolidation

Soil improvement solutions were implemented where the settlements indicated potential risk of damage to the pre-existing structures. Such interventions entail improving the characteristics of the ground, and mitigating the deformation effects induced by tunnelling, by means of low-pressure cement injection.

Different grouting configurations were adopted, based on the mutual position of the tunnel and the pre-existing structures, as well as site accessibility and the type of usage of the surface on the site (see Fig. 8.43).

The project included full-face cement grouting in proximity of the stations where the TBM entered into or exited from the stations. Taking into account the environmental and geological conditions, the drilling and grouting operations were carried out from the surface and/or from service shafts and tunnels as shown in the Fig 8.43.

Figure 8.43 Ground consolidation actions in Turin Metro.

8.5 SMART SOLUTION OF KUALA LUMPUR (MALAYSIA)

8.5.1 Project particulars (Table 8.13)

Table 8.13 Project particulars

Location	Kuala Lumpur, Malaysia (Fig. 8.44)
Project period	2004 – April 2007
Name	SMART Project Stormwater
Owner	Stormwater Management And Road Tunnel Government of Malaysia, Drainage and Irrigation Department and Malaysian Highway Authority
Client	MMC – Gamuda JV (Contractor and Concessionaire)
Geodata activity	Provision of geotechnical and structural monitoring and TBM Data Management System through a GIS-based tool (GDMS)

Figure 8.44 A view of the center of Kuala Lumpur.

8.5.2 General description of the project

The SMART project in Kuala Lumpur responds to the urgent need to solve the city's devastating flooding problem which occurs when climate conditions in the catchment area of the Klang River basin cause flows to surface at the confluence with the Ampang River (see Fig. 8.47). The problem has resulted in the growing peak time congestion on a major traffic route from the south. Central to the project is a 12.8-m internal diameter tunnel that will operate, both as a stormwater retention and bypass channel, to divert floodwater around and away from the city centre, and as a double-deck toll road facility for cars (see Figs. 8.45 and 8.46).

The channel beneath the lower deck will remain open permanently while double sets of water-sealing gates at each end of the road section will protect the roadway as retention of floodwater in the tunnel increases. Under extreme floodwater conditions, traffic will be barred from entering the tunnel and the gates will be opened to allow the passage of floodwater through the traffic compartments as well.

8.5.3 Design criteria

Flood control and mitigation:

Total floodwater retention capacity	3 million m^3
Tunnel depth	Generally 24–31 m below surface
Cover above the tunnel	On average 1–1.5 times the tunnel diameter
Internal operating pressure	2 bar
Extreme flood conditions	One in 100 years probability

Central toll road facility:

Operation	3 km double-deck, two lanes per deck
Operating speed	50 km/hour expressway
Emergency passages connecting the road decks	Nine at 250 m interval
Road tunnel environment	Watertight to 2.5-bar pressure

8.5.4 Tunnel characteristics

Tunnel characteristics

Length	9.7 km
Excavation method	Two Herrenknecht Mixshields and cut-and-cover
Lining type	Segmental lining
TBM 1 drive North	5.2 km north to the Klang holding basin
TBM 2 drive South	4.1 km south to the Kerayong storage reservoir

The dual-purpose tunnel will divert floodwaters away from the confluence of the two major rivers running through the city centre while its central section will double up as a two-deck motorway to relieve traffic congestion at the main southern gateway into the city centre.

MODE 1 operates under normal conditions or when rainfall is low such that no water needs to be diverted into the tunnel.

Moderate storms activate *MODE 2* that will divert floodwater into a bypass tunnel in the lower section of the motorway tunnel, which will remain open to traffic.

Heavy storms will activate *MODE 3*.The tunnel is closed to road traffic and the full tunnel section becomes available to divert the water flows. Extensive monitoring stations will ensure that sufficient time is allocated before the automated watertight gates are opened.

The 3 Modes of operating the tunnel are illustrated in Fig. 8.46.

Figure 8.45 Conceptual view of the operating tunnel.

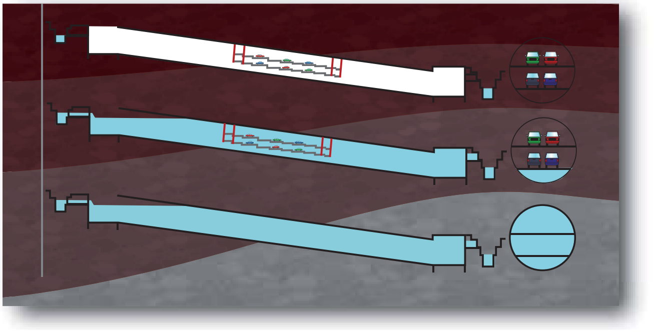


Figure 8.46 Simulation of the three operation modes of the SMART tunnel.

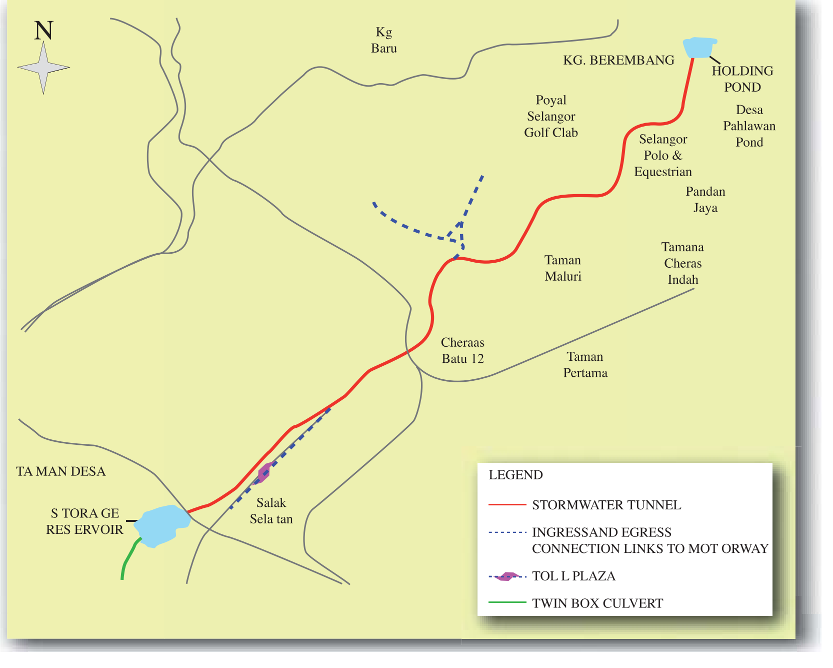


Figure 8.47 General layout of the SMART project.

8.5.5 Environmental and geological conditions

Main constraints

Urban environment.
Cover above the tunnel ranges almost everywhere from 0.8 to 1.8 times the diameter of the tunnel.
High groundwater table.
Mixed ground comprising karstic limestone, alluvial deposits and mine waste.

8.5.6 Method of excavation

About 95% of the tunnel is done using two slurry TBMs and the rest by means of the cut-and-cover method. The 13.2-m diameter TBMs (see Fig. 8.49) were among the world's largest-diameter machines.They work in opposite directions from the middle of the tunnel alignment. Each of the two Herrenknecht TBMs is about 70 m in length and 2500 tons in weight. Concrete segments with 500 mm thickness are installed. The excavation by slurry TBMs in this case proved to be correct choice, being very successful (see Fig. 8.48).

Cavities which are not filled with soil pose a tremendous risk to the project. Consequently, the separation plant has 2000 m^3 of bentonite available to compensate for any loss to the formation.

Figure 8.48 One of the breakthroughs during tunnelling in the SMART Project.

8.5.7 TBM data (Table 8.14)

Table 8.14 TBM data

Manufacturer	Herrenknecht
TBM Type	Mixshield
Cutterhead diameter	13.26 m
EPB pressure (max)	3 bar
Power	8200 kW
Thrust (max)	94500 kN
Torque (max)	24400 kNm
Penetration rate (max)	50 mm/min
TBM shield length	10.24 m
Slurry circulation flow rate	2.40 m^3/h

Figure 8.49 One of the two Herrenknecht (13.26-m diameter) Mixshields.

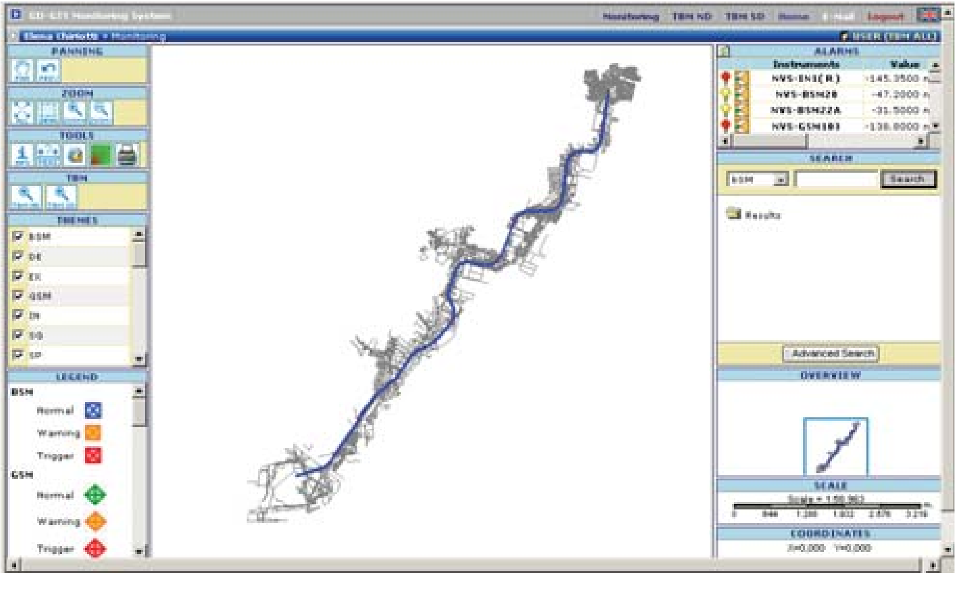


Figure 8.50 The main window of the GDGMS Monitoring System for the Kuala Lumpur SMART Project.

8.5.8 Key factors

The Client considered continuous geotechnical and structural monitoring to be essential, together with monitoring of the TBMs.

The real time availability of monitoring data has proven to be very effective for management of the construction process, particularly where work methods are clearly specified and the countermeasures are defined to face a foreseeable set of anomalous conditions (risk-management operational protocols).

The system GDMS (Geodata Management System) developed using the Web-GIS concepts, is capable of handling the information coming from different sources (monitoring, site investigation, building surveys, machinery performance, and ground treatments) in the frame of a unified modular platform (see Fig. 8.50).

GDMS (see Figs. 8.51 to 8.53).

The GDMS platform is totally modular; the module specifically committed to the monitoring data management is known as GD-GIS. This revolves on a GIS (Geographic Information System) technology and is formed by an integrated system of database and procedures, connected to territorial objects that can be required and that are located on a geo-referenced digital map of the Project (see Fig. 8.50).

GD-GIS objectives

Collect, georeference, and organize all the geographic information related to the ante-operam site conditions before tunnelling (buildings, infrastructures, etc).

Collect, georeference, and organize the data flow generated by the construction process (excavation progress, monitoring data, investigations data, etc).

Validate the collected information.

Allow the analysis and query of stored data based on predefined itemized keys (e.g. to retrieve data about the geotechnical properties of certain grounds, the observed fluctuations of the watertable within a certain district over a certain period of time, etc).

Review and compare different types of data (e.g. baseline geological model vs. actual geological conditions; surface monitoring data vs. in-ground monitoring data; monitoring data vs. building conditions; TBM-operation technical parameters vs. the ground monitored behaviour; comparison of collected measurements vs. reference alert thresholds; forecasted instruments scheme vs. installed instruments; etc).

Provide an easy and reliable tool for automatic data reporting.

In the SMART Project, Geodata was commissioned to supply its GDMS system for handling the large quantities of real-time data produced by the Project.

Data can be retrieved using standard enquiry dialog boxes and via the geographical interface. Data can be displayed as graphs against time or position (distance from the excavation face), in table format (for download), and represented as contour plots (settlement measurements), see Figs. 8.51 and 8.52.

The innovative functions introduced with the Web-GIS monitoring application, include the following:

- The customized management of users (different user profiles, with various levels of operating permission), even on a geographical base (to preserve data ownership).

- The introduction of a synchronized back-up service, on a remote server, to manage possible breakdowns or local system's failure.
- The possibility of an on-demand generation of subsidence contour maps, using different filters for data selection.

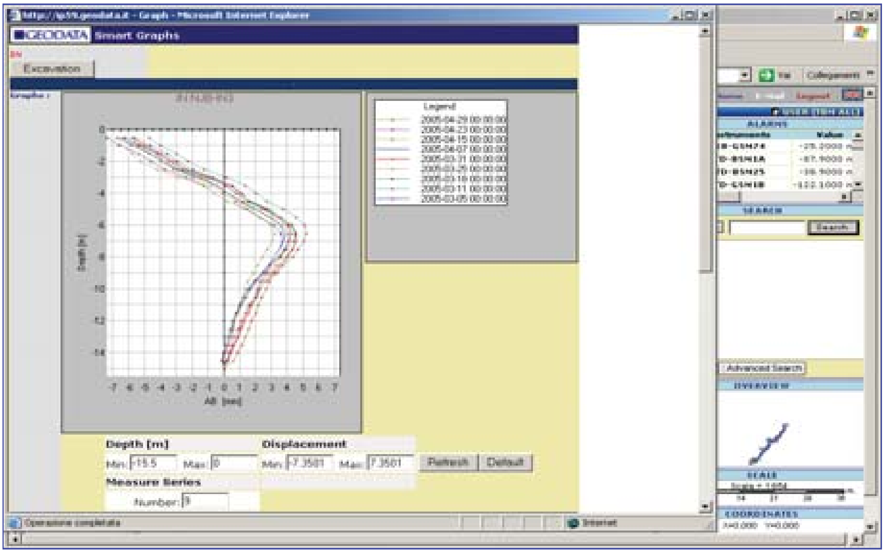


Figure 8.51 Chronological depiction of displacement at various depths at a given section of the tunnel.

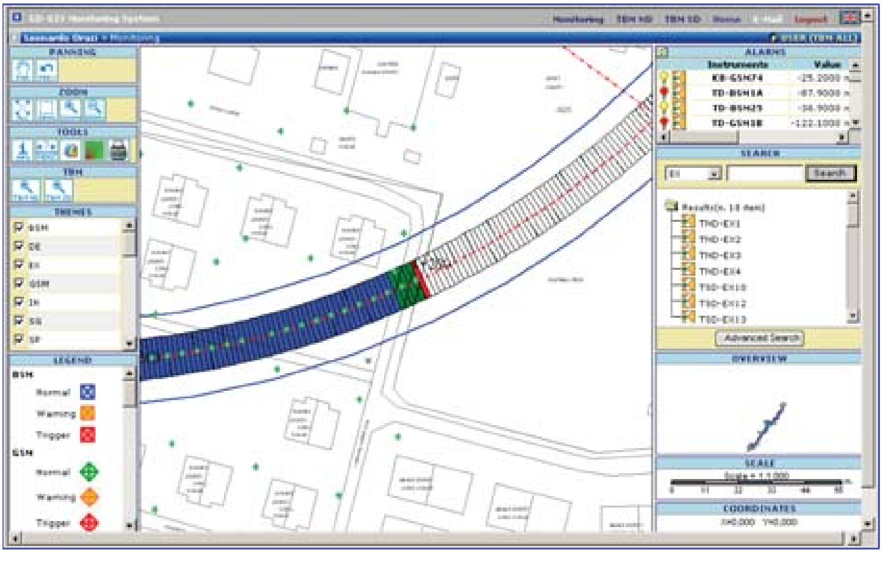


Figure 8.52 Enlargement of Figure 8.50 showing: (1) the position of the face and (2) the installed and the indicated, future rings.

8.6 HIGH SPEED RAILWAY LINE OF NODO DI BOLGNA

8.6.1 Project particulars

Table 8.15 Project particulars

Location	Bologna, Italy
Construction period	2003–2006
Name	Nodo di Bologna High Speed Railway
Owner	TAV.REIS S.p.A
Designer(s)	Italferr S.p.A
Contractor	J.V. San Ruffillo (Acciona-Ghella-Salini)
Geodata's activity	Consultant
Monitoring	Golder Associates
TBM monitoring	Stone (Italy, Milan)

8.6.2 General description

A section of the new High Speed Rail Line Milan-Naples, which crosses the city of Bologna, an intensely urbanized area, is foreseen for the most part to be underground (see Figs. 8.53 and 8.56). In 2000, the joint venture S.Ruffillo (Acciona-Salini-Ghella) was awarded the contract for realizing the Lot 5 of the Project, underpassing the city of Bologna.

The Project starts at the North abutment pier of the Savena Bridge (km.0 + 000), south of the city, and ends at the new Central Station (km.7 + 375); it consists of the following main structures:

- A cut & cover tunnel and a launch shaft (for the TBMs), double track, from km 0 + 000 to km 0 + 958 (see Figs. 8.54 and 8.55).

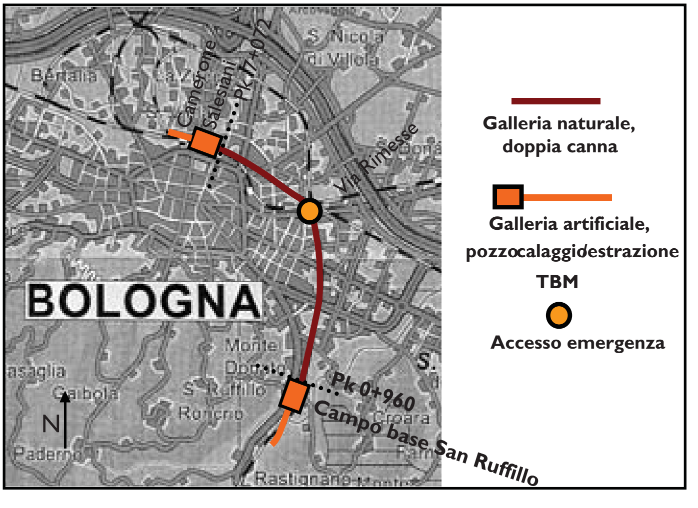


Figure 8.53 Layout of "Nodo di Bologna" project.

Figure 8.54 Overview of the South Portal of the Tunnels.

- Two EPB tunnels ("Pari" and "Dispari" tunnels), single track, 9.4-m diameter, from km 0 + 958 to km 7 + 075.
- An Emergency Shaft ("Via Rimesse" Shaft) at km 4 + 820 and a Ventilation Shaft at km 6 + 857.
- A transition shaft (TBM's exit and "Bologna" parking) from km 7 + 075 to km 7 + 235.
- A mined tunnel, double track, from km 7 + 235 to km 7 + 350, connecting the transition shaft to the Central Station.
- The first EPB started excavation in July 2003 and the second started in November 2003. The completion of the excavation was achieved for both TBMs at the end of May 2006.

The excavation by TBM was done in two distinct periods:

1. From July 2003 to October 2004: from km 0 + 980 to km 3 + 050 (1st TBM) and to km 2 + 440 (2nd TBM).
2. From May 2005 to May 2006 (end of excavation : km 7 + 375).

A number of geotechnical complexities, contractual constraints, and settlement problems stopped the excavation in October 2004 when only 32% of the 1st tunnel and 23% of the 2nd one were excavated.

With the aid of the opinion received from the Dispute Review Board, the excavation stoppage was resolved at the end of May 2005. On resumption, the Contractor set up a rigorous and extensive monitoring system in terms of TBM parameters and effect on surface, implemented with the assistance of a Consultant (Geodata Spa) responsible for EPB-excavation operation control.

8.6.3 Tunnel characteristics (Table 8.16)

Table 8.16 Tunnel characteristics

Total Tunnel Length	6112 m
Tunnel axis distance	15 m
Overburden (min–max)	15–21 m
Excavation diameter	9.4 m
Excavation area	69.81 m^2
Lining type	Segmental lining
Ring type	Universal
Thickness	40 cm
No. of segments	6 + 1
Ring length	1.5 m
Connections	Bolts

The two tunnels start at the S.Ruffillo quarter, south of Bologna, and from km 0 + 960 to km 1 + 500 underpass an electrical power plant and a recently built store. From approximately km 1 + 500 to km 7 + 075, the alignment runs below one of the main Italian railways, the Bologna-Florence line, that lies on an 8–12 m high embankment. The average overburden thickness ranges from 15 to 21 m (at the bottom of the embankment), with a minimum value of 5 m in the first 100 m of excavation.

Figure 8.55 The second Lovat TBM ready to start.

8.6.4 Environmental context and geological condition

Ground conditions were very heterogeneous and comprise of soft marine clays, sands, and alluvial deposits (mainly gravel).

In the first part of the alignment, up to km 2 + 150, the tunnels were excavated in marine clay and loose sandy deposits (Pliocenic Clay and Yellow Pleistocenic Sands) below the water table. In the second part, the tunnels were excavated in Savena river deposits, consisting of mainly gravel and sand layers with a high percentage of fines (lenses of clay and silt).

The alignment was subdivided into nine 'homogeneous' zones (see Table 8.17) based on dominant ground conditions, even if in reality these are more heterogeneous than initially thought.

The heterogeneity of the excavated ground represented a critical aspect because the excavation conditions, in terms of surface settlement response and machine's behaviour, changed very rapidly along the alignement.

Figure 8.56 A view of Bologna surroundings.

Table 8.17 Geotechnical zones along the tunnel

Geotechnical Zoning	Zone Position (from km to km)	Length (m)	Ground types				
			Gravel and silty sandy gravel	Silty sand	Clayey silt	Silty clay	Water table presence
1	0+960 – 1+800	840		X	X	X	Yes
2	1+800 – 2+150	350		X	X		Yes
3	2+150 – 3+400	1250		X			
4	3+400 – 4+200	800		X			
5	4+200 – 4+600	400	X	X			
6	4+600 – 5+350	750		X			
7	5+350 – 6+100	750			X		
8	6+100 – 6+800	700				X	
9	6+800 – 7+072	272				X	Yes

X: Ground types considered for the definition of the reference geotechnical parameters for the calculation of the required face-support pressures

8.6.5 Method of excavation

In order to minimize the impact on the surface, the Client imposed a mechanized excavation methodology (EPB, Slurry shield or Mix shield), i.e. a technology to minimize the needs for modification of the ground and the disturbance to the ground water regime. The Contractor selected two EPB Machines, due to the following considerations:

- geological and geotechnical conditions (variable ground conditions);
- the respective experiences of the two lead Companies in the Contractor J.V. for similar underground works (Acciona in Madrid and Barcelona) and Ghella (in Valencia and Caracas), and
- environmental problems related to the slurry disposal and the treatment plant.

The two tunnels are parallel for most of the alignment (up to km 6 + 650 approx) and the pillar width between them is only 5.6 m. The minimal distance between the tunnels represented one of the most critical aspects of the excavation.

The twin-tunnel solution was imposed by the Client and the Designer (TAV S.p.A. and Italferr S.p.A.) due to safety reasons for the train passengers and because the realization of a unique 14-m diameter double-track tunnel, with minimal overburden (1–1.5 diameter), would have caused critical differential settlements for the existing infrastructures, facilities, and buildings on surface.

As a matter of fact, during the excavation it was verified that, due to the small distance between the 2 tunnels, the ground encountered by the 2nd EPB machine was more disturbed by the 1st tunnel excavation and presented poorer geotechnical characteristics. As a consequence of this effect, during the EPB excavation it was necessary to apply more restrictive control parameters and a higher level of excavation control when the second tunnel was excavated (see Fig. 8.57).

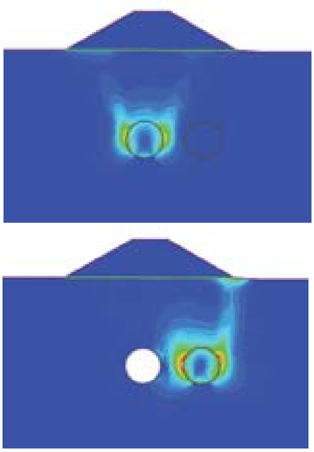


Figure 8.57 The effects of the ground plasticization when the second TBM is excavating near the existing tunnel.

8.6.6 TBM data (Table 8.18)

Table 8.18 TBM data

Manufacturer	LOVAT
TBM Type	EPB (RME-370SE)
Cutting head diameter	9.4 m
Max EPB pressure	3.5 bar
Thrust (max)	100000 kN
Torque (max)	24000 kNm
Max penetration rate	80 mm/min
No. of thrust jacks	36
No. of grouting lines	6
No. of foam injection lines	8
Shield length	10.7 m

8.6.7 Job site organisaton and performaces

In particular, the performances of TBM no. 2, in the two distinct periods of excavation mentioned previously, are reported in Fig. 8.58, while the average performance of both TBMs are listed below.

TBM data

Average performance:	8.2 ring/day (TBM 1) 9.33 ring/day (TBM 2)
Best day for both TBMs:	25 rings (37.5 m/d)
Best month:	612 m (first TBM, March 2006) 635 m (second TBM, April 2006)
Best month (TBMs together):	1,182 m (April 2006)

First phase of the excavation operated 24 h/day, 5 days/week.

Second phase of the excavation (after the restart in May 2005): 24 h/day, 7 days/week without any planned stoppage (except for maintenance and repair).

The performance of the two machines during the last three-month reached 3,200 m of twin tunnell.

All the impressive performances were achieved working strictly in EPB mode with the plenum always full of material and under pressure of 1.5 to 1.8 bar at the crown, following the design specifications.

The intervention and responsibility of Geodata are refered to management of the following Residual Risks:

- Potential increase of settlements given that 2 collapses already occurred in the first part of the alignment.
- Influence of excavation effects due to close proximity of the two tunnels.
- Underpass of many interferences (buildings, bridges, etc) expected along the alignment.

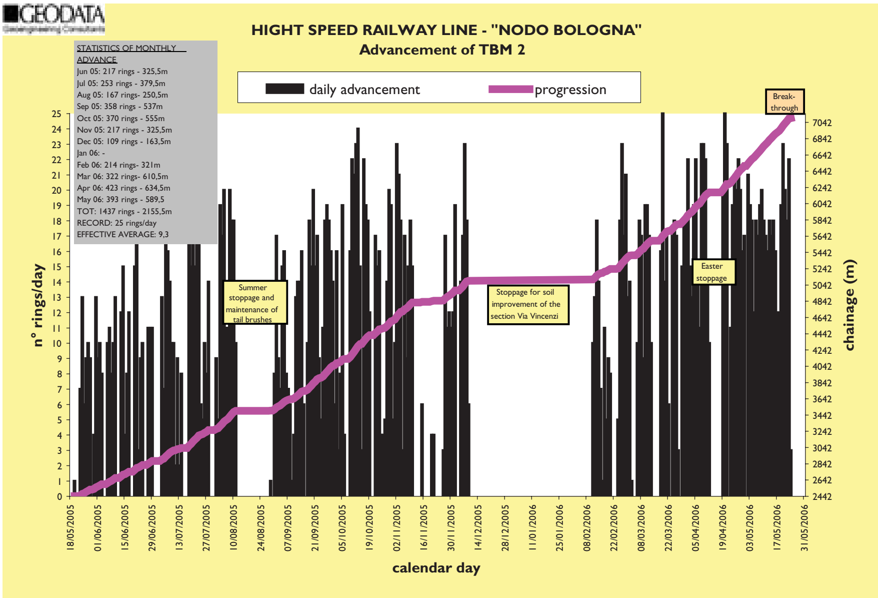


Figure 8.58 High Speed Railway Line "Nodo di Bologna" – Advancement of TBM 2.

8.6.8 Key factors

The excavation control system

The following factors were identified as essential for the re-start and strict control of the excavation:

- Production of a detailed Plan for Advance of Tunnel (PAT) for each 300 m of excavation and for each TBM, so that all the parameters and design issues related to tunnelling were effectively addressed before starting the excavation. During the excavation of every 300 m of the tunnel, the Consultant back-analysed the main TBM parameters, the geotechnical conditions and effects on surface, to check the appropriateness of the used and recommended control values. After considering the interferences with the existing surface structures, the Consultant also defined the excavation procedures and TBM parameters for the next 300 m stretch.
- The on-site presence of a tunnel engineer from the Consultant: to collect all the excavation parameters, to control the adherence to the working procedures for all tunnelling phases, and to ensure that the TBM operations were carried out in a proper manner. The fundamental parameters controlled during excavation were: Face Support Pressure, Muck "Apparent Density", Extracted Muck Weight, Tail-Void-Grouting volume and pressure.
- Setup of a Technical Desk for reviewing and analysing both the daily and weekly TBM excavation reports, the range of operating conditions, recorded parameters, and any significant events.

Monitoring system

A specific monitoring system was developed in order to get all the required information in real time into a specific database on web-platform (GIDIE – Golder Associates Srl), to understand the soil behaviour and the potentially-affected-structures response during the different excavation phases and to decide if any counter measure should be activated.

Almost 200 EPB parameters were constantly recorded (every 5 sec) and made available (in real time through the specific project website) to the Client and the Consultant. As a matter of fact, the back-analysis of the surface settlements, compared with the excavation parameters, was proven to be the most efficient way to evaluate the response of the ground towards TBM excavation and the interferences of one TBM with the excavation by the other later on (see Fig.8.59).

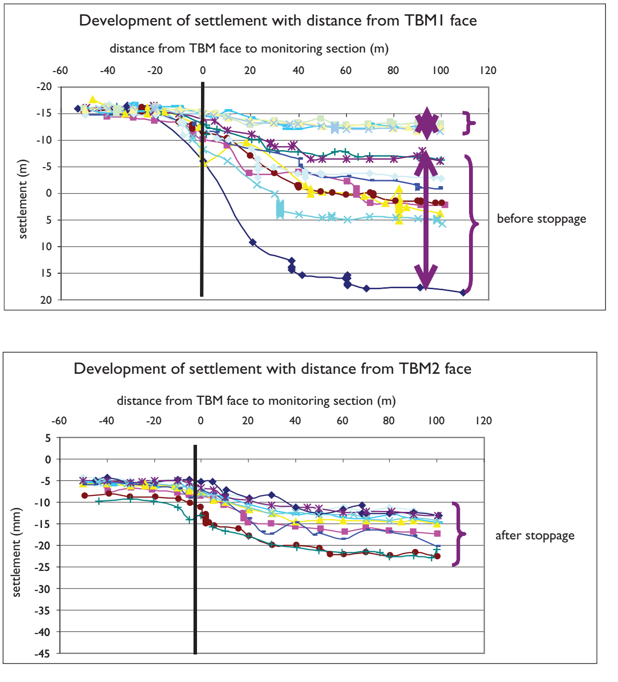


Figure 8.59 Graphs of some measured settlements.

The excavation control consultant analyzed continuously the settlement data to correlate volume loss and settlements with the EPB pressure, also taking in consideration the short/long stops, see Figs. 8.60 to 8.63.

The implementation of stricter TBM excavation controls reduced significantly the settlements to a maximum value of 2 cm and the volume loss to values of about 1.2%, a result absolutely satisfactory considering that most of the excavation was performed in granular soils.

Compensation grouting operations

For the excavation of the twin tunnels underneath the railway line in operation, a series of protective measures had to be implemented in order to minimize the effects of the tunnelling operation to the adjacent structures and, in particular, to railway bridges. These measures comprised of conventional protection like consolidation grouting, as well as the active settlement control with compensation grouting where the potential damage risk was relatively high (see Fig.8.64).

One important structure was the railway bridge in brick called "Ponte Vecchio", which passes Via Emilia Levante, an important inner city street. During the TBMs passages the differential settlements of the bridge piers were limited by controlling the volume and pressure of grout injected through the appropriate grouting valve-pipes and by rigorously controlling the EPB machine excavation parameters.

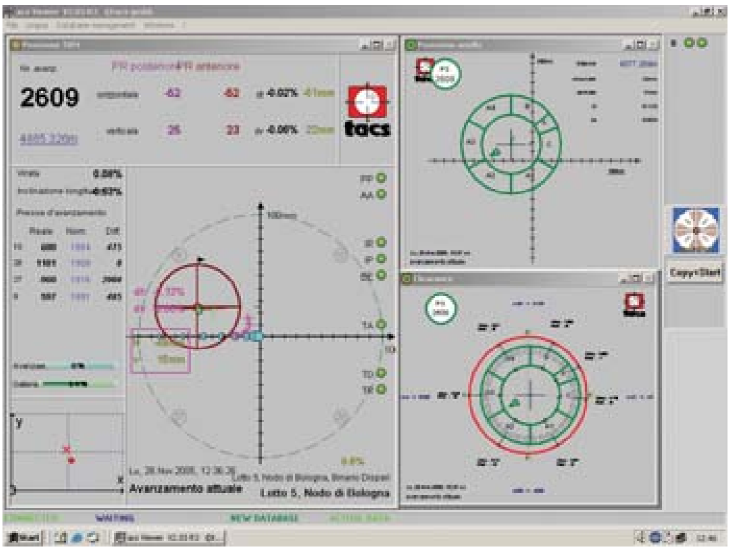


Figure 8.60 The TACS guidance system screen.

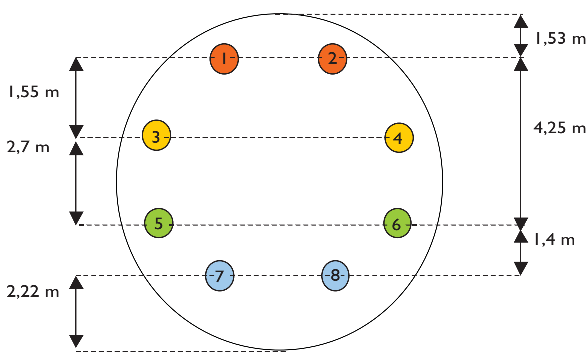


Figure 8.61 The position of the 8 pressure sensors on the Lovat EPB TBMs.

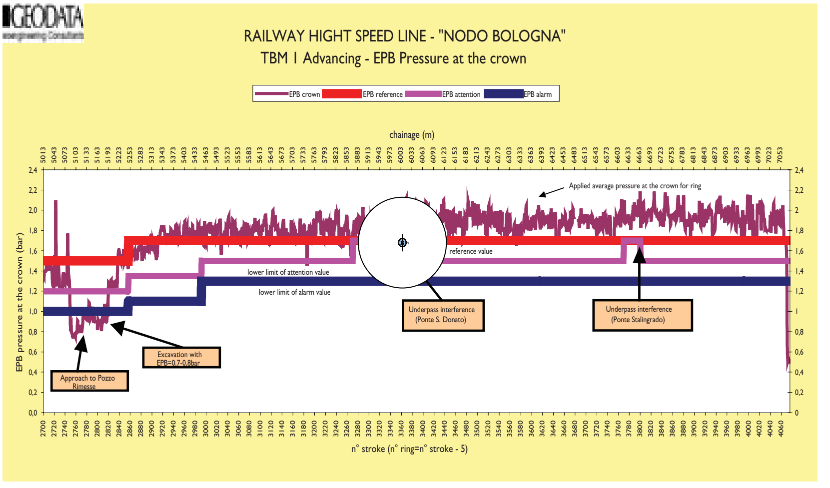


Figure 8.62 The recorded pressure values and the relevant lower and upper limits.

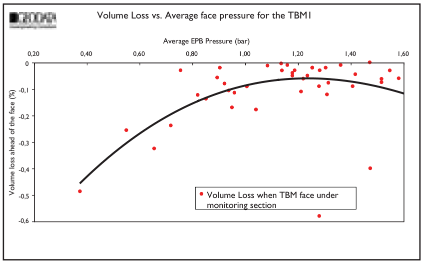


Figure 8.63 "Initial" Volume Loss vs. average face pressure.

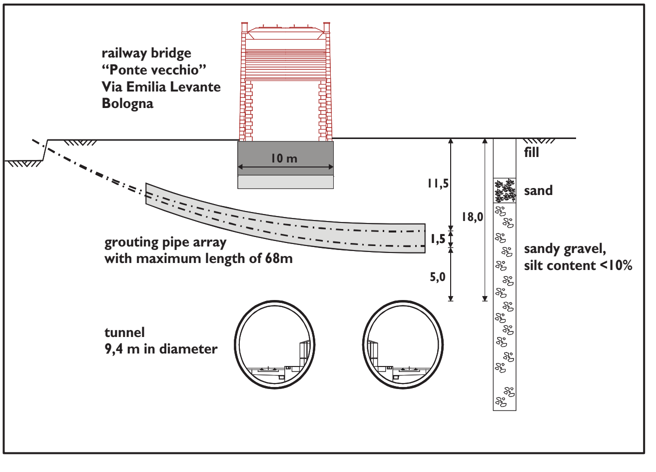


Figure 8.64 Sketch of the "compensation grouting system for underpassing the "Via emilia Bridge".

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Appendix 1

Types and characteristics of TBMs

1 DEFINITIONS AND CLASSIFICATION

For the purpose of this book, it is useful to have a general overview of the various types of tunnelling machines, even though the attention is focused on the types of machines most reliable for working in urban areas.

In the underground technical and scientific community a unitary definition and classification for the Tunnelling Machines (TMs) has not yet been reached.

The "Japan Tunnelling Association" was the first to subdivide the TMs on the basis of the excavation mode that could be full- or partial-face excavation. Afterwards TMs were further subdivided for the presence or not of an excavating wheel and on the basis of the contrast system (grippers or longitudinal thrust jacks). The term Tunnel Boring Machine (TBM) is now universally adopted for all the machines that have a full-face cutting wheel for excavating a tunnel.

The German, Austrian, and Swiss associations that deal with tunnelling (Deutscher Ausschuß für unterirdisches Bauen: DAUB, Österreichische Gesellschaft für Geomechanik und Arbeitsgruppe Tunnelbau der Forschungsgesellschaft für das verkehrs- und Straßenwesen: Fachgruppe für Untertagebau des schweizerischen Ingenieur und Architektenverein) in 1996 adopted a classification in which only the machines that use a full-face cutting wheel for boring rocks were considered.

The French Tunnel Society (Association Française Travaux En Souterrain: AFTES) suggested the following classification based on the support typology that a machine is able to supply during excavation: the TMs that do contrast or the front nor the excavation cavity (open machines); the TMs that contrast excavation cavity (shielded machines); and the TMs that contrast both the cavity and the excavation front (shielded machines with face-support pressure).

The classification scheme adopted in this book, and presented briefly in the following paragraphs, is based on what have been developed by ITA Working Group 14 "Mechanized Excavation" and by the Italian Tunnel Society (SIG: Società Italiana Gallerie). TMs are subdivided according to both the support typology that the machine is able to supply and the type of ground that it is able to operate in.

Like in the AFTES and ITA classifications, in this book the term TBM refers to all machines that have a full-face cutterhead.

The following list shows the most common types of TBMs.

2 ROCK TUNNELING MACHINES

2.1 Unshielded TBMs

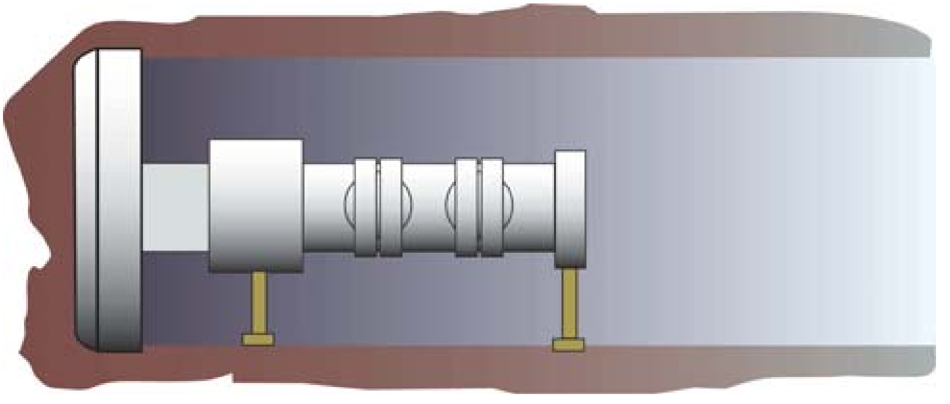


Figure ap.1.1

Typical "rock machine": this TBM is used when excavating in rock with good to very good conditions and it needs to be associated with primary support system as for excavation using conventional method (rock bolts, shot concrete, steel arches, etc).

Function principle A cutterhead, rotating on the same axis as that of the tunnel being excavated, is pushed against the excavation face by a series of thrust jacks, connected to the frame, which is locked at the rock through a series of "grippers"; the cutters (normally disc cutters) penetrate into the rock, creating intense tensile and shear stresses and thus crushing it locally. Special buckets in the cutterhead allow the muck to be collected and then removed by the primary mucking system.

The machine is not equipped with a full circular shield; normally just a small safety crown-shield is mounted at the back of the cutterhead.

The working cycle includes: 1) gripping to stabilize the machine; 2) excavating for a length equivalent to the effective stroke of the thrust jacks; 3) regripping; 4) new excavation.

Main Components

- Cutterhead (equipped with disc cutting tools) and primary mucking system.
- Thrust jacks.
- One or more pairs of bearing pads (grippers) to grip the tunnel periphery.
- Engines, driving gear and other electrical, mechanical, and hydraulic equipments.

Depending on the type of stationary element, it is possible to further classify unshielded TBMs into main beam types or kelly types.

Main field of application Rock whose quality ranges from "very good" to "medium".

2.1.1 Special unshielded TBMs

2.1.1.1 Reaming boring machines – RBMs

Function principle The Reaming Boring Machine is a TM, which allows an axial pilot tunnel (normally excavated using a TBM) to be widened (reaming).

The function principle and the working cycles of the RBM are the same as those for unshielded TBM.

Main components

- Rotating reaming cutterhead, on which the cutting tools are fitted, and primary mucking system.
- Thrust jacks.
- Two pairs of grippers located inside the pilot tunnel opposite the reaming cutterhead.
- Engines, driving gear and other electrical, mechanical, and hydraulic equipments.

The machine is not equipped with a full circular shield.

A special type of RBM is the *Down Reaming Boring Machine*; this machine is used for shaft excavation and enables the top-to-bottom reaming of an axial pilot shaft normally excavated using a Raise Borer (see below).

Main field of application Rock masses whose characteristics range from optimal to moderate with medium-to-high self-supporting time.

2.1.1.2 Raise borer

Function principle The Raise Borer is a TM used for shaft excavation that enables the bottom -to- top reaming of a small diameter axial pilot hole created using a drilling rig.

A cutterhead, rotating on the same axis as that of the shaft being excavated, is pulled against the excavation face by a drilling rod guided through the pilot hole. The cutters create crack and chips with the same mechanism as illustrated for the unshielded TBMs. Debris falls to the bottom of the shaft where they are collected and removed.

Main components

- Rotating cutterhead (equipped with disc cutting tools).
- Drilling rod which provides torque and pull to the cutterhead.
- A main frame, located outside the shaft, which powers the drilling rod for excavation.

Main field of application Rock of “very good” to “medium” quality.

2.2 Single shielded TBMs

Typical “ground” or “weak rock” machine, used when it is necessary to support the tunnel very soon with precast lining.

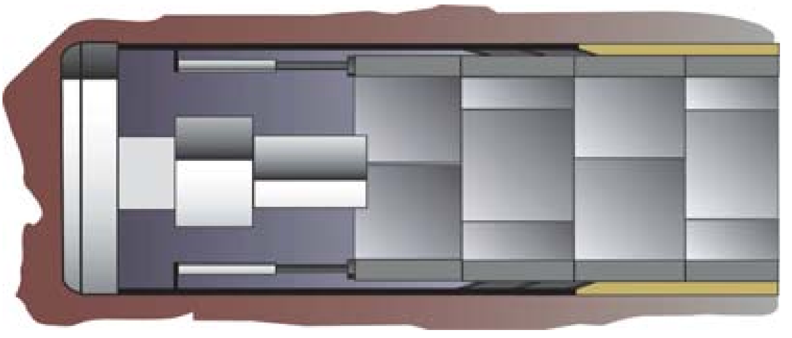


Figure ap.1.2

Function principle The excavation system is similar to the unshielded machines, but the support to the advancing thrust is provided by the precast segments, constituting the tunnel lining. The Single Shielded TBM is equipped with a full round protective shield immediately behind the cutterhead. The working cycle includes: 1) excavation for a length equivalent to the stroke of the thrust jacks; 2) assembling of concrete segments lining and retraction of the jacks; and 3) new excavation stroke.

Main components

- Rotating cutterhead (usually equipped with disc cutting tools) and primary mucking system.
- Protective shield which is cylindrical or slightly tapered; the shield may be monolithic or articulated (the shield is made of two or three pieces, facilitating the formation of curves).
- Thrust system consisting of a series of longitudinal hydraulic jacks located inside the shield, pushing against the tunnel lining.

Main field of application Rock whose quality ranges from "good" to "poor".

2.3 Double shielded TBMs

The following is a quote from the Robbins Company's Catalogue:

"*Year:* *1972*
Project Name: *Orichella*
Country: *Italy*

The contractor for the project, SELI, needed a TBM solution that would protect workers in broken ground and provide a rapid rate of advance, while simultaneously lining the tunnel. To fit their need, The Robbins Company invented the Double Shield TBM."

That was the birth certificate of the double shield TBM, which was born from the collaboration of SELI S.p.A. and Robbins Co. in the early 1970s. This type of machine has demonstrated to be a very flexible machine, useful especially in mixed rock conditions.

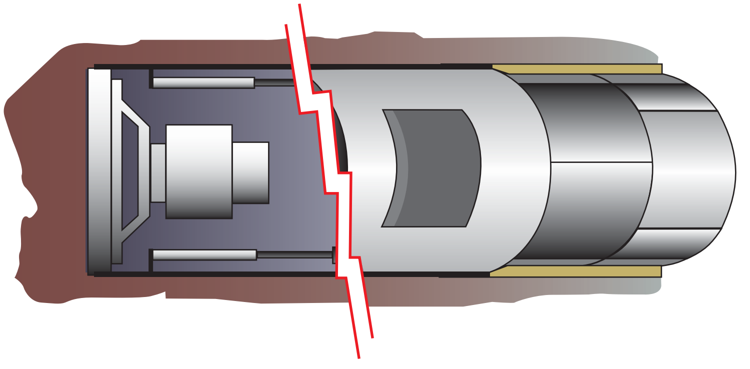


Figure ap.1.3

Function principle Similar to single shielded TBMs, but offers the possibility of a continuous work cycle owing to double/multiple longitudinal thrust reaction system of telescopic sections against ground support system, or by gripping. This machine is more versatile than the single shield, since it can advance even without installing the tunnel lining or install lining segments during excavation, depending on the ground stability conditions. It assures, in any case, short working cycles, that means high advance speed.

Main components

- Rotating cutterhead (equipped with disc cutting tools) and primary mucking system.
- Protective shield which is cylindrical or slightly tapered.
- Double/multiple thrust system which normally consists of:
  - A series of longitudinal jacks.
  - A series of grippers pushed to the tunnel walls to support the jack's thrust.

Main field of application In homogeneous rock whose quality ranges from "very good" to "poor".

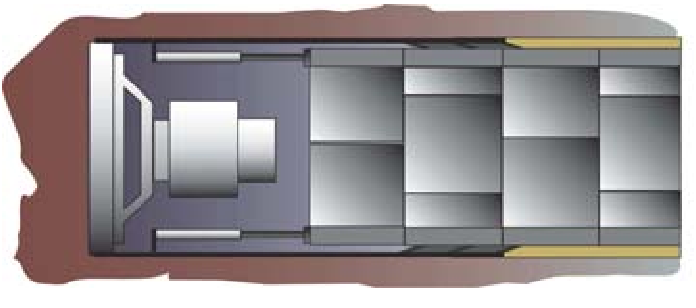


Figure ap.1.4

3 SOFT GROUND TUNNELING MACHINES

3.1 Mechanically supported closed shields

Function principle The mechanically supported, closed shield is a TBM, equipped with a full round protective shield immediately behind the tunnel face. The cutterhead plays the dual role of acting as the cutterhead and supporting the tunnel face, using movable plates and thrust against the face by special hydraulic jacks. The debris is extracted through adjustable openings or buckets and conveyed to the primary mucking system.

Main components

- Rotating cutterhead (equipped with blade and tooth type of cutting tools) and primary mucking system.
- Protective cylindrical shield containing all the main components of the machine.
- Longitudinal thrust jacks.

Main field of application Weak rock, cohesive or partially cohesive ground, self-supporting ground in general. Absence of groundwater.

3.2 Compressed air closed shields

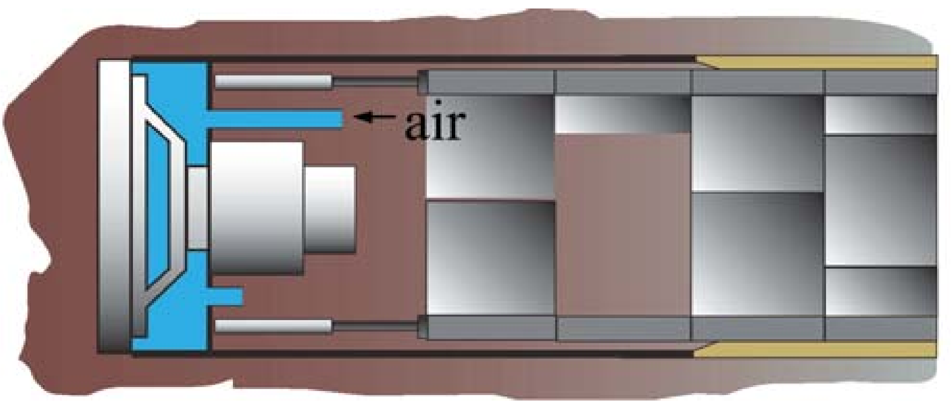


Figure ap.1.5

Function principle In compressed air closed shields, the rotating cutterhead acts as the means of excavation whereas face support is ensured by compressed air at a sufficient level to balance the hydrostatic pressure of the ground. Debris are extracted from the pressurized excavation chamber using a ball-valve-type rotary hopper and then conveyed to the primary mucking system.

Main components

- Rotating cutterhead (equipped with blades and tooth type of cutting tools).
- The protective cylindrical shield containing all the main components of the machine; the front part is closed by a bulk head which guarantees the separation between the excavation chamber (pressurized), housing the cutterhead, and the zone containing the machine components (un-pressurized).
- Longitudinal thrust jacks.

Main field of application Ground lacking self-supporting capacity and with medium-low permeability ($k \leq 10^{-4}$ m/s). Presence of groundwater. Injecting bentonite slurry onto the excavation face can locally reduce permeability.

3.3 SLURRY SHIELDS (SEE ALSO SECTION 4.4)

3.3.1 Slurry shields – SS

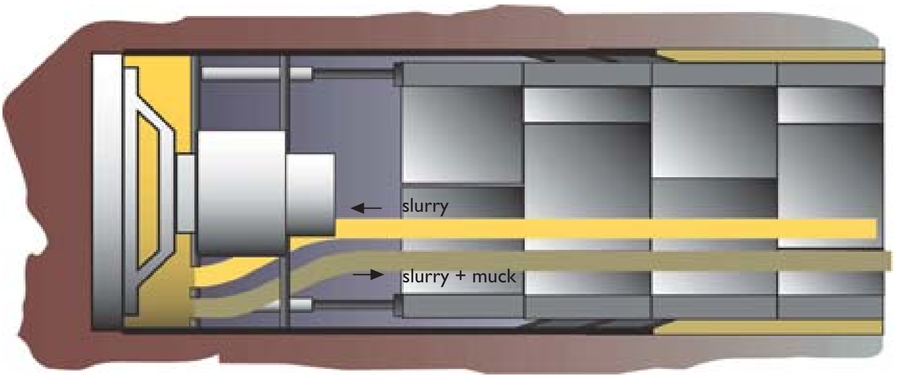


Figure ap.1.6

Function principle The cutterhead supports the excavation tools; face-support pressure is provided by slurry: a suspension of bentonite or clay in water. A bulkhead divides the working chamber from the tunnel. The slurry suspension is pumped into the excavation chamber and penetrates into the ground forming a filter cake, i.e. an impermeable membrane (in fine ground) or impregnated zone (in coarse ground) that guarantees the transfer of face-support pressure to the excavation face.

Excavated debris consists of natural excavated soil mixed with the bentonite- or clay-slurry. The resulting mixture is pumped (hydraulic mucking) from the excavation chamber to a separation plant, which enables the bentonite/clay-slurry to be partially recycled; the separation plant is normally located on the surface.

Main components

- Cutterhead (discs, blades or teeth).
- Protective shield containing all the main components of the machine; the front

part is sealed by a bulkhead which guarantees the separation between the shield and the excavation chamber (pressurized) containing the cutterhead.

- Longitudinal thrust jacks.
- Slurry and debris separation system (normally located on the surface).

Main field of application Soft ground with limited self-supporting capacity. In granulometric terms, slurry shields are mainly suitable for excavation in ground composed of sand and gravels with silts. The installation of a crusher in the excavation chamber allows any lumps that would not pass through the hydraulic mucking system to be crushed. The use of disc cutters also enables the machine to excavate through rock, if present. Polymers can be added to excavate ground containing too much silt and clay. Presence of groundwater.

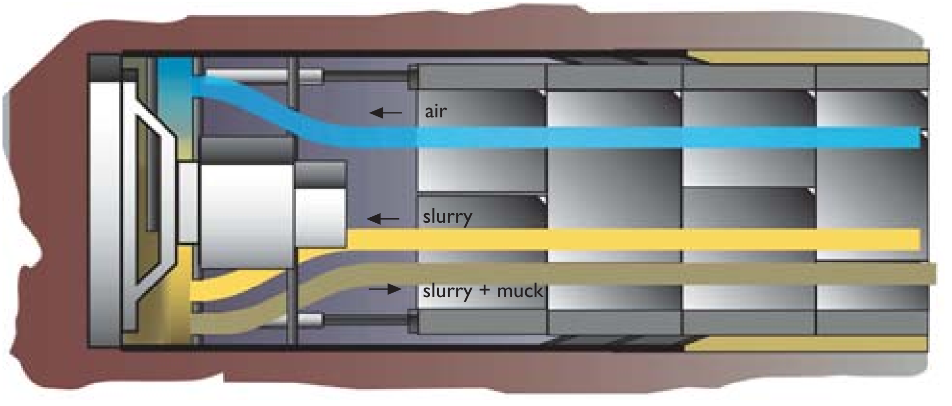


Figure ap.1.7

3.3.2 Hydroshields (HS)

Function principle Identical to SS described before; the difference is the way of transferring the support pressure to the face.

In the Hydroshield there are two bulkheads: one separates the working chamber from the tunnel and the second divides the chamber in two parts, leaving a communication in the lower part. The upper part of this intermediate chamber is filled with compressed air (Air Cushion). Connection to an air compressor and a valves control system allows to adjust the support pressure at the face independently from the hydraulic circuit (supply of bentonite slurry and mucking of slurry and natural ground).

Main components Similar to those described for slurry shields.

Main field of application The same as for slurry shields.

3.4 EARTH PRESSURE BALANCE SHIELDS – EPBS (SEE ALSO SECTION 4.5)

3.4.1 Earth pressure balance shields – EPBS

Function principle The cutterhead supports the excavation tools; face support is provided by the excavated ground that is kept under pressure inside the excavation chamber by balancing the volume of the extracted and excavated material, and by the thrust jacks on the shield. Excavation debris is removed from the excavation chamber by a screw conveyor that allows the pressure control by variation of its rotation speed.

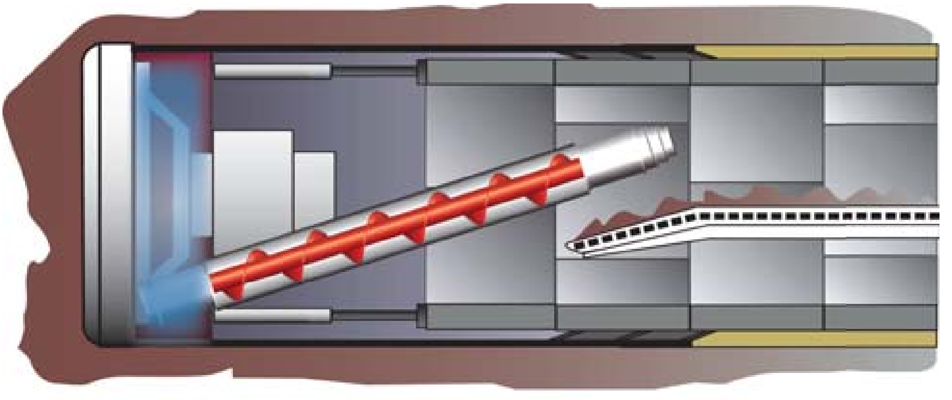


Figure ap.1.8

Main components

- Cutterhead: rotates with cutting spokes.
- Protective shield: similar to that described for closed slurry shields.
- Longitudinal thrust jacks.

Main field of application Soft ground with presence of groundwater and with limited or no self-supporting capacity. The typical application fields of EPBS are silts or clays with sand. The use of additives, such as high-density mud or foams, enables excavations in sandy-gravely or gravely ground. The use of disc cutters enables the machine to excavate in rock.

3.4.2 Special EPBS

DK shield Differs from the earth pressure balance shield because of the geometry of the cutterhead whose central cutter projects further than the cutters on the spokes, thus creating a concave cavity.

Double shield (DOT shield) These are two partially interpenetrated earth pressure balance shields that operate simultaneously on the same plane, creating a “binocular” tunnel.

Flexible Section Shield Tunnelling Method Earth pressure balance shield in which the excavation system is based on the presence of several rotating cutterheads that enable the construction of non-cylindrical sections.

Elliptical Excavation Face Shield Method Earth pressure balance shield in which the combined action of a circular cutterhead and additional cutters enables an elliptical section to be excavated.

Triple Circular Face Shield Tunnel This consists of three shields, operating by means of earth or slurry pressure balance, which allow large excavation sections to be constructed, such as those required to house an underground railway station.

Vertical Horizontal Continuous Tunnel This is a slurry-pressure-balance TM consisting of a main shield, for shaft excavation, which contains a spherical joint housing a secondary shield. When the main shield has reached the appropriate depth, the spherical joint is rotated 90° and the secondary shield starts tunnel excavation.

Horizontal Sharp Edge Curving Tunnel Similar to the Vertical-Horizontal Continuous Tunnel, it enables the construction of two tunnels intersecting at right angles.

Double Tube Shield Technology This is a TM fitted with two concentric shields. The bigger shield excavates a stretch of tunnel with the large section and the small one continuous the excavation of the remaining stretch of the tunnel with a smaller section.

Appendix 2

TBM manufacturers from the new millennium

Canadian company founded in 1972 designs, modifies, manufactures, assembles and tests a complete line of tunnelling equipment utilized in the construction of tunnels.

The company is 100% owned by Lovat family and produced more than 300 TBMs, working in 26 countries with machines of diameters from 0.75 to 15 m.

Lovat produces Rock TBM (single to double), soft TBM (open, semi-closed, EPBS and Slurry Shield), pipejacking, and microtunnelling.

Figure ap.2.1 The LOVAT 9,4 m diameter Earth Pressure Balance Shield machine which operated in Bologna Railway system.

Herrenknecht AG develops, manufactures and sells the entire range of mechanical tunnelling machines, with 1,449 employees and a total turnover of 384 Mio. Euro (2003).

The high tech products can be deployed in almost all geological conditions and provide drilling diameters ranging from 100 to 18,000 mm.

The continuous presence on worksite and the assistance services given, required for efficient utilization of a TBM, is guaranteed by the world-wide presence of Herrenknecht AG high level team of employees.

Figure ap.2.2 The 7,97-m diameter Herrenknecht Earth Pressure Balance Shield which operated in Galleria Quattro Venti project.

The french company NFM TECHNOLOGIES offers proven state-of-the-art technology for boring machines closely adapted to any kind of geology, without any boring limitation up to 15.5 metres: EPB, BENTON'AIR® Slurry machine, single or telescopic shield hard rock TBM, dual mode machine.

To date, NFM TECHNOLOGIES has supplied more than 40 machines for worldwide infrastructure projects. Most of these machines have achieved world class production performances, including a number of world records. (Cadiz, Hong-Kong, Copenhagen, Madrid…)

Figure ap.2.3 The 8.03-m diameter NFM Earth Pressure Balance Shield which operated in Milan railway system.

WIRTH

The internationally active company WIRTH, founded in 1895, is one of the leading manufacturers of heavy drilling equipment and since 1965, constructs and supplies Tunnel Boring Machines.

Machines from WIRTH are in use in tunnel boring projects all over the world and for the excavation of a wide variety of tunnels: from drainage tunnels of the smallest diameter, to pressure tunnels for hydroelectric power plants, cable tunnels and road tunnels.

TBMs are supplied with diameters from 1 m up to 15 m and more, and are applicable in any kind of geologic conditions: hard rock, mixed rock and soft rock or ground, depending on their characteristics.

Figure ap.2.4 The 8.17-m diameter WIRTH Earth Pressure Balance Shield which operated in Channel Tunnel Rail Link.

The Robbins Company is engaged in the design, manufacture, sale and rental of custom equipment for the underground excavation industry, since 1951 when the Company manufactured its first TBM.

Robbins' primary product is tunnel boring machines (TBMs) with diameters from 1.6 to 12.87 meters. Specialized in hard rock TBMs, Robbins also designs and sells a wide range of other equipment and services which are required for efficient utilization of a TBM.

Robbins was the inventor, together with SELI S.p.A. (Rome-Italy), of the "Double Shielded Machine".

Robbins Company has designed and manufactured more than 250 tunnel boring machines.

Figure ap.2.5 The Robbins Hard Rock TBM supplied for the excavation of Venaus explorative tunnel.

OTHER CONSTRUCTORS

Far from Europe and America, Japanese constructors (mainly Kawasaki, Komatsu and Mitsubishi) have always, and nowadays assume an important role in supplying the market far urban area mechanized excavation machines.

Being the creators of EPBS technology, from 60's years to date they have consolidated their position as leaders in the oriental market.

Japanese Companies are continuously increasing their position in European market, covering the entire range of TBM types, from rock to soft-ground, including "city machines".

Appendix 3

Geotechnical investigations for tunnelling in urban areas

1 OBJECTIVES

The choice of the best tunnelling option must be based on the information regarding the impediments and adverse conditions along the tunnel alignment as well as the "geotechnical" (geological, mechanical, and hydrological) conditions along (and above) the tunnel axis.

After a preliminary step of analysis for the selection of a favourable location, systematic geological, hydrogeological, and geotechnical investigations should be carried out along the tunnel axis. Particular care must be taken if construction is to pass under or near buildings, railway embankment, car parks, and other tunnels. Attention must be paid also to the underground occurrence of filled hollows, replaced sediments, rubble from old buildings, refuse, and ancient infrastructures.

The objective of this appendix is to first, outline the various "geotechnical" aspects that can be potentially encountered by the tunnel and second, identify the investigation techniques (performed) in-situ, in the laboratory, and during construction for obtaining data for input to the initial design and subsequent adjustments.

The geotechnical aspects, and their investigation, discussed in this appendix are particularly relevant to the following sections of the book:

Section 2.2: Reference is made to a list of geology and hydrogeology related risks; the need for appropriate (and adequate) investigations, on site and in the laboratory, is underlined; and the update of the geotechnical data is considered a necessity for responding to unforeseen conditions.

Section 5.2: The design of face-support pressure has a fundamental relationship with the type of ground that is excavated, which eventually constitutes a major part of the material in the plenum. Therefore, an accurate (and regularly updated) data on the geotechnical properties of the excavated ground is necessary for calculating the face-support pressure as well as for providing input to the numerical analyses of stability of the tunnel.

2 THE RELEVANT ASPECTS

In general terms, the construction of an underground structure implies a complex interaction between the designed structure and the surrounding soils or rock masses. In particular terms, the excavation of a tunnel requires a complete and precise definition of the geological and geotechnical context in its neighbourhood, in terms of both stratigraphical and mechanical characteristics (shear strength and stiffness). This is necessary not only for a correct design of the final lining and definition of the opportune support measures of the excavation face, but also for the choice of the interventions for protection of the already existing structures.

2.1 The geological aspects

The mostly recurring geological problems are a consequence of some specific geological features as shown in Table ap.3.1.

A "reference geologic model" of the ground can be developed by first, preparing a database and second, by quantifying the parameters (for input to design) by analyzing the data, using statistical means and professional judgement.

Preparation of the database will normally involve literature search, site investigations (borehole surveys and geophysical measurements), and laboratory tests for obtaining geo-mechanical characteristics of the ground. Quantification of the design parameters (observing the need for accuracy) will generate the requirements for details of the data to be collected during the design and construction stages.

Table ap.3.1 Geological characteristics potentially encountered by underground civil works

Geological context	*Specific geologic features*
Lithology, tectonic structures, and geomechanics	Mineralogical and textural characteristics of the lithotypes Structural characteristics of the intact rock and of the rock mass
Geology, structures, and tectonics	Small scale structures Regional scale tectonic structures Flows of temperature, thermal gradient Seismicity Vertical movements at regional scale In-situ stress conditions
Sedimentology	Mineralogical characteristics Genetic, textural, and structural characteristics of the deposits
Hydrogeology	Characteristics of the hydrogeological networks (extension, boundary conditions, sources of recharge, mode of groundwater flow, etc.) Karstic region Presence of gas Characteristics of the circulating fluid (nature, chemistry, temperature, etc.)
Geomorphology	Deformation phenomena of slopes (in the portal areas, along the alignment of tunnels, running under and parallel to the strike of slopes) Geotechnical characteristics of the superficial deposits Identification of outcropping deposits, their lateral limits and contact with bedrock

2.2 The geotechnical aspects

The interpretation of the behaviour of the ground is based on mechanics applied to the soils and rock masses; it is thus possible to define a geotechnical description (profile) along the alignment with the aid of adequate tests and investigations, both in-situ and in the laboratory.

Soils are derived from the deposition in layers of particles after transport by streams, waves, wind or ice. These deposits can be simply classified in respect to the type of the constituent particles: clastic sediment are those derived directly as particles broken from a parent rock source, and non-clastic sediment are from the newly created mineral matter precipitated from chemical solutions or from organic activity.

Mechanical analysis applied to soil, as a continuous medium, allows the definition of the two principal properties of a soil: the mechanical strength and the stiffness, which are the fundamental elements for a geotechnical design.

The mechanical strength is represented by the maximum shear strength that the soils are able to support and is a function of both a friction component, at the solid particles of the ground level (proportionally to the confinement load) and a cohesive component, due to the interparticle connection (which is generally proportional to the degree of overconsolidation and/or the level of cementation). The common representation of the soil strength is provided by the Mohr-Coulomb equation that expresses the shear strength as a function of cohesion, angle of friction, and normal stress.

The in situ soils are subjected to a "natural" stress state due to their own weight. These stresses are known as geostatic stresses and, in the most general case of homogeneous deposits in parallel plane layers and horizontal ground level, the vertical and horizontal forces assume the form of principal stresses, which are functions of the depth and density of the ground, and of the Poisson's ratio, in the case of the horizontal stress.

A list of the principal geological and geotechnical parameters necessary for design of mechanized tunnels is given in Table ap.3.2, which names the parameters and identifies the symbols used for them. The correspondence between the parameters and the related group of variables is provided in Table ap.3.3.

From a geotechnical point of view, mechanized tunnel excavation in urban environment can involve some potentially critical elements.

Interpretative difficulties can arise during the design and/or construction stages because of the complexity of the stratigraphic and geotechnical context. The most important difficulty relates to the excavation in the presence of a heterogeneous excavation face (mixed face) where difficulties can arise in maintaining the face-support pressure in the plenum.

Squeezing or swelling ground. Squeezing ground commonly refers to materials that displace into a tunnel, due to the action of the surrounding stress gradient. The effects of squeezing immediately become evident during an excavation as closure starts to take place at the tunnel face. Swelling behaviour implies that the response of soils to stress changes in the presence of water due to a significant swelling of a clay component (montmorillonite and, to a lower extent, hyllite and kaolinite) Both squeezing and swelling are stress and strain related phenomena, with a time dependent volume increase in the case of swelling.

Running ground and liquefaction. When soil particles can move freely, as for example, in the presence of loose sand, *running ground* phenomena can occur, both in a dry state and in the presence of water, which can cause liquefaction when the soil is disturbed by tunnelling activities. Settlement phenomena can arise due to the loss of ground at a tunnel face, ineffective filling of the tail voids, water inflow with the soil inrush, and poor ground control at the shield. Damage to already existing structures (buildings and utilities) can occur after the settlement phenomena. In the presence of cyclic loads (usually in the case of earthquakes or humanly induced vibrations), some saturated soils of a prevalently sandy granulometry with a low overburden can be subject to *liquefaction* (annulment of the effective forces and instantaneous collapse of the shear strength) with the relative collapse of the ground and propagation of the strain phenomena to the surface.

Adhesion. In soils with a prevalence of fine contents, especially where clayey minerals are present (kaolinite, hyllite and smectites), phenomena of *adhesion* (sticky behaviour) can affect the tools, the walls of the chamber and the mucking plants.

Table ap.3.2 Relevant geological and geotechnical parameters for design of mechanized tunnels*

Mechanical properties	Cohesion (c_u, c') Friction angle (φ_u, φ') Deformation modulus (E_t, E_s)
Sedimentological properties	Mineralogical content (i.e. Quartz and clays) (MC) Grain Distribution (GSD) Porosity (n) Interlamination (IB)
Hydrogeological properties	Natural water content (W_n) Saturation degree (S) Permeability coefficient (K)
Index properties	Atterberg Limits (W_L, W_P) Plasticity Index (I_P) Consistency Index (I_C) Activity Index (I_A) Density (ρ_w, ρ_s, ρ_d)

Table ap.3.3 Geological and geotechnical parameters and their influence on mechanized tunnelling aspects**

Soil pressure	c_u, φ_n, φ_u, φ_w, ρ_w, (E_t, E_s)
Subsidence	c_u, c', φ_u, φ', E_t, E_s, n, S
Tunnel-face stability	c_u, φ_u, GSD, n, IB, K, (E_u, E_s)
Excavability	c_u, φ_u, GSD, IB, I_C, I_P
Alteration/weathering	MC, GSD, IB, W_n
Mucking, muck storage	ρ_s, GSD, W_n, φ', S
Adhesivity	MC, W_n, W_L, W_P, I_P, I_C, I_A, GSD
Separation	GSD, MC, IB, I_P, I_C
Abrasivity	MC, GSD, n, IB

(*, **) Definition of the symbols given in Table ap.3.2 and Table ap.3.3:
c_u = Undrained cohesion, c' = Effective cohesion, φ_u = Undrained friction angle, φ' = Effective friction angle, φ_n = Natural friction angle, E_t = Tangential modulus of elasticity, E_s = Secant modulus of elasticity, MC = Mineral Content, GSD = Grain Distribution, n = Porosity, IB = Interlamination, W_n = Natural water content, S = Saturation degree, K = Permeability coefficient, W_L = Atterberg limit for liquid state, W_P = Atterberg limits for plastic state, I_P = Plssticity Index, I_C = Consistency Index, I_A = Activity Index, ρ_w = Water density, ρ_s = Density of soil particle, ρ_d = Drained density.

2.3 Hydrogeological aspects

The presence of underground water is one of the important elements that characterize the subsoil. If water is encountered at the level of the tunnel, it is obvious that the excavation of the tunnel will be profoundly conditioned by its presence in both the design and construction phases. The study of groundwater is important from both technical and environmental points of view, using quantitative and qualitative approaches, respectively.

The technical analysis must include the intrinsic particularities of the permeated soils, that is, a study of the porous aquifer systems is necessary. An aquifer system can be considered well defined when the relative hydrodynamic parameters (governing the groundwater flow), the geometric characteristics (the hydrostucture) and the flow field are known.

Water circulation underground, within a porous aquifer system, is governed by permeability K (expressed in m/s) and by the type of porosity, referring only to the transit of circulating gravitational water in the saturated zone.

The definition of a hydrostructure can be made by taking advantage of previously performed studies and bibliographic indications, but it must be aided by in-situ investigations with continuous coring and piezometric surveys.

The next step is to determine the permeability and other hydrodynamic parameters of the aquifer system when it is necessary to quantitatively assess the interference of the tunnel with the aquifer and, in particular, when the use of numerical models is foreseen.

From an environmental point of view it is necessary first, to determine the actual state of the water resources and second, to assess the potential deterioration that is induced by the tunnel excavation. In the first case, use can be made of specific thematic studies, which are often conducted by the local water authorities, and this information can be integrated and updated through a dedicated geochemical survey campaign. Later on, interference could be evaluated by determining the intrinsic and integrated vulnerability of the aquifers and, in an even more complex manner, using a flow and transport model that is able to simulate the qualitative evolution of the aquifer.

3 IN-SITU AND LABORATORY INVESTIGATIONS

The investigations for obtaining data on the geological, hydrogeological, and geotechnical variables (and parameters) for design and construction of mechanized tunnels requires the use of tools and planning for making observations and/or tests on site and in the laboratory. The set of investigations relevant to mechanized tunnelling are presented here under the headings: in-situ investigations, laboratory tests, and investigations during construction.

3.1 Geological and geotechnical investigations in-situ

The main type of investigation involves geognostic drilling, preferably continuous logging, which permit the subsoil to be studied in stratigraphic terms through direct checking of the intercepted lithotypes.

During drilling advancement, sampling of the soil can also be carried out and these samples can be sent to the laboratory together with numerous geotechnical tests and determinations in the boreholes.

The most recurrent and rapid of the borehole tests are the dynamic penetration tests (SPT, Standard Penetration Test for the determination the mechanical strength and stiffness of the soils), pressiometric and dilatometric tests (MPM and DTM, mainly dedicated to the determination of the elastic characteristics) and schistometric tests (vane test, for the estimation of the undrained shear strength).

Permeability tests and eventually geophysical investigations (in particular seismic and sonic methods to reconstruct the lithostratigraphic layout) can also be performed in boreholes.

Finally, it is also possible to make use of a certain number of geotechnical tests on a dedicated vertical hole without the help of drilling. These, tests include the static penetrometric tests (CPT, which are very reliable for both the stratigraphic reconstruction of the subsoil and for geotechnical characterisation), dynamic penetrometric tests (like DPSH, which are similar to SPT) and plate loading tests (PLT, for the determination of the elastic characteristics of the investigated ground).

The in-situ investigations and tests allow the soils to be investigated in their effectively natural conditions (in terms of stress state, water content, and so on) while guaranteeing a high reliability of the experimental results. This is true, especially for fine grained deposits, although it is not possible to obtain a complete geotechnical characterisation with just in-situ tests (most tests that can be carried out permit the geotechnical analysis in only total stress terms, that is, in undrained conditions). It is therefore necessary, in these cases, to proceed with undisturbed sampling of silts and clays and with the relative laboratory tests.

3.2 Hydrogeological investigations in-situ

Boreholes are again the key to access the subsoil and are essential for obtaining the knowledge that is used to recognise the hydrostructures. The tests must be performed through continuous coring for a precise determination of the hydrostratigraphic limits.

The measurement of the water levels in boreholes, even during drilling, is particularly important for a piezometric survey: a wise modulation of the logging, with respect to the lining that follows, allows the groundwater to only enter the borehole in sections without lining (usually from 1 m to 3 m). In this way, it is possible to appraise a preferably stabilised piezometric level (during a pause in the drilling) which is specific for each depth.

Geophysical investigations can be made in order to integrate the geognostic drillings, especially at a preliminary stage, to recognise the hydrostructure and identify any aquifer levels. The most suitable methods change from one location to another according to the involved lithologies. However, taking into consideration the possible depths of investigations in urban contexts (generally up to 30–40 m) and the delicacy due the presence of water, electric tomography is often the most utilised.

Permeability can be estimated through specific tests in boreholes, both during drilling and once the piezometer has been installed, if it is an "open pipe" type. Permeability coefficient tests also exist, which can be made from specific wells, but they are not considered very useful.

The Lefranc test (for both a variable and constant load), the Lugeon test and generally all the slug tests that are performed on a limited section of the borehole, free of lining, can all be used as permeability tests during drilling. In all cases, these tests offer an estimate of the permeability that is limited to the uncovered stretch of the borehole and which is often not significant for the overall permeability of the entire aquifer.

Pumping tests are performed in wells that very often must be specifically managed. These wells should have a minimum diameter of 250–300 mm and be equipped with a certain number of inspection piezometers positioned diametrically to the well, according to a cross-scheme. The test involves drainage using discharge steps, each of which is of constant discharge, and the last of which lasts for some time (usually 36 hours). Finally, at the end of the drainage, the piezometric levels are checked (hydraulic jump tests). All the piezometric recording operations are usually performed automatically, with programmed measuring intervals.

3.3 Geotechnical tests in laboratory

Geotechnical laboratory tests allow the physical and plastic characteristics of the soils to be quantitatively determined and this leads to their classification. The main parameters that can be determined include the evaluation of the natural density, the water content, the granulometric composition, and the Atterberg limits.

For estimating the mechanical strength parameters, the laboratory test should be carried out with the triaxial compression cell. This test can be performed in three different ways: unconsolidated undrained (UU, for the determination of the undrained shear strength), consolidated undrained (CU, with which it is possible to obtain the failure envelope parameters in both effective terms and total stress terms), and consolidated and drained (CD, with the failure envelope only in effective parameter terms).

The results of the triaxial tests can be integrated with the results of direct shear tests (Casagrande box tests) using suitable corrections that take into account the different failure conditions.

The elastic characteristics can be derived from the previous tests and can also be directly obtained through edometric tests with which it is also possible to obtain the consolidation coefficient and the permeability of the sample. It is also possible to test the uniaxial compressive strength for the estimation of the undrained shear strength.

Finally, dedicated tests can be carried out to determine specific characteristics, according to the requirements of the case. Some of the most frequently used tests are related to the evaluation of the swelling index of the soils, which is used for evaluation of the variations in the consequent pressures associated with swelling. Table ap.3.4 lists the investigations required for input to a geohydrogeological model.

Table ap.3.5 lists the specific in-situ and laboratory tests that are relevant for mechanized tunnelling in urban areas (note: [lab/situ] for laboratory and in situ test; R for test on rock/rock mass, S for soil test). Procedures and practice for most of these tests are closely standardized by the ASTM (American Society for Testing and Materials Standards), AASHTO (American Association of State Highway and Transportation Officials), ISRM (International Society of Rock Mechanics) and others. A comprehensive description of these standards could be also found in the recent European Norms, EN (see References).

Table ap.3.4 Investigation for the reconstruction of the Geological Reference Model

GEOLOGY	
Geology and Geomorphology	Topography, photogrammetry and photointerpretation, remote sensing Regional/detailed lithological, structural, geomorphological surveys/studies
Characterization of rock/soil (type, structure, texture)	Detailed lithological, structural and geomorphological surveys/studies Boreholes
Characterization of rock faulting/jointing (type, structure)	Detailed lithological, structural surveys/studies Boreholes (with or without log, etc.)
Degree and depth of weathering	Detailed lithological, and geomorphological surveys/studies Boreholes Geophysical methods
Characterization of karst phenomena (type, geometry, infilling, water)	Detailed lithological, structural and geomorphological surveys/studies Speleological surveys/studies Boreholes Geophysical methods (micro-gravimetric and radar survey)
Location and geometry of cavities, infilling	Speleological surveys/studies Boreholes Geophysical methods (micro-gravimetric and radar survey)
HYDROLOGY AND HYDROGEOLOGY	
Hydrologic condition	Topography Photogrammetry and photointerpretation Regional/detailed hydrologic studies
Hydrogeologic condition	Topography Photogrammetry and photointerpretation Regional/detailed hydrogeological surveys/studies Boreholes
Hydrotermal condition, gas presence	Regional/detailed geological surveys/studies Boreholes
SEISMICITY	
Seismicity	Regional/detailed geostructural surveys/studies Study and interpretation of historical data

Table ap.3.5 Specific geological and geotechnical tests in-situ and laboratory

STATE OF STRESS	
Horizontal/Vertical stress ratio	Oedometer test with lateral pressure control (S/R) [lab], Triaxial test with lateral deformation control (S/R) [lab], Dilatometer (R) [situ], Marchetti dilatometer (S) [situ]
Consolidation degree	Oedometric test (S) [lab]
INDEX PROPERTIES	
Unit volumetric weight	Density tests (S/R) [lab], Gamma-densimeter (R) [situ]
Water content, saturation degree, void ratio	Laboratory index tests (S/R) [lab]
Plasticity index	Atterberg limits determination (S) [lab]
Granulometric characteristics	Grain-size/sedimentation analyses (S) [lab]
Activity	Mineralogic analyses (S) [lab]
Residual strength	Residual strength test (shear, riaxial tests) [lab]
Deformability, elastic constants	Plate loading test (R) [situ], Directional dilatometer test (S/R) [situ], Uniaxial-Triaxial compressive tests (S/R) [lab/situ], P-S wave measurement (S/R) [lab/situ], Deformation measurements (extensometer, convergence, settlement) (S/R) [situ]
Compressibility characteristics (consolidation index, edometric compressibility index)	Oedometric test [lab]
Viscous behaviour constants	Flat jack method (R) [situ], Long-time plate loading test (R) [situ], Creep load test (S) [lab], Cycle dilatometer test (S/R) [situ], Deformation measurements (S/R) [situ]
Swelling	Swelling test (S/R) [situ], Oedometric test (L)
OTHER PROPERTIES	
Abrasivity	Abrasivity (R) [lab], Cerchar test (R) [lab], Abrasivity (Norwegian Institute of Technology) (R) [lab], LCPT test (S/R) [lab]
Hardness	Hardness (R) [lab], Schmith hammer (R) [lab], LCPT test [lab], Knoop [lab], Cone Indenter test (NCB) [lab], Punch test (Colorado School of Mines) [lab], Drop test (Norwegian Institute of Technology) [lab], Los Angeles test (S/R) [lab]
Drillability	Siever's test [lab], Drillability test [lab]
Mineralogic and petrographic features	Mineralogic analyses (S/R) [lab], Petrographic analyses (S/R) [lab], Physico – chemical analyses (S/R) [lab]
Sensibility to water, solubility	Mineralogical analyses (S/R) [lab]
Sensibility to thermic/ hygrometric variations	Mineralogical analyses (S/R) [lab], Heating test (S/R) [lab] Freezing test (S/R) [lab]

(*continued*)

Table ap.3.5 (*Continued*)

MECHANICAL PROPERTIES	
Shear strength	Casagrande shear box test (S) [lab], Direct Shear test (R) [lab], In situ direct shear test (R) [situ], Triaxial test (S/R) [lab], Scissometer/Vane test (S) [situ]
Uniaxial compressive strength	Uniaxial compression test (S/R) [lab], Point load test (R) [lab/situ]
Tensile strength	Direct tensile test (R) [lab], Brazilian test (R) [lab], Point load test (R) [lab/situ]
Sticky behaviour	Mineralogical analyses (S/R) [lab], Atterberg limits (S) [lab]
HYDROGEOLOGICAL PROPERTIES	
Permeability	Observation during borehole drilling [situ], Permeability tests (Lefranc, Lugeon) [situ], Injection test [situ], Pumping test [situ], Oedometric test [lab]
Piezometric level, hydraulic gradient	Piezometer (close/open type) [situ]
Water flow	In situ measurements (in tunnel, springs) [situ]
Water Fisic and chemical characteristics	Salt content, aggressivity, hardness, pH value, temperature, etc

4 INVESTIGATIONS DURING CONSTRUCTION

The use of a TBM for construction of a tunnel does not permit continuous, direct observation of the tunnel face. Therefore, the evaluation of the ground conditions ahead of the tunnel face are normally obtained using indirect methods.

Through these indirect methods, it is possible to derive the principal characteristics of the soil/rock mass because variations in TBM performance parameters are usually correlated with changes in the geotechnical-geomechanical situations. For this purpose the TBM must be equipped with appropriate instrumentation.

The collected data must be stored in a dynamic database (based on Geographical Information System, G.I.S.), continuously updated, and transferred to a group of skilled engineering geologists for interpretation, extrapolation, and forecasting. From a tunnel excavated by TBM, it is possible to investigate the ground ahead of the tunnel face using the methods listed in Table ap.3.6.

Table ap.3.6 Investigation ahead of the tunnel face

Lithological investigation, core recovery, water measurement, void identification and characterization	
	Horizontal or inclined core recovery boreholes *
	Directional core recovery boreholes *
	Horizontal or inclined destruction boreholes * (with determination of drilling rate, pressure on drill bit and of the drilling fluid, torque)
Sedimentological/geostructural mapping of the face/sidewalls (type, structure, texture) **	

(*continued*)

Table ap.3.6 (*Continued*)

Borehole logs	Gamma ray log Neutron logs Geoelectric logs Georadar
Geophysical methods from tunnel face	Georadar Seismic

* with/without water preventer, drilling rods should be of the aluminum type.
** when the TBM stops excavation.

Appendix 4

Description of the principal elements of the 12 analytical methods for defining face-support pressure

4.1 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 (Fig. ap.4.1).

The model does not provide indications for practical applications. However, it has been used by several authors as a basis for further development (see methods 4.9 & 4.10).

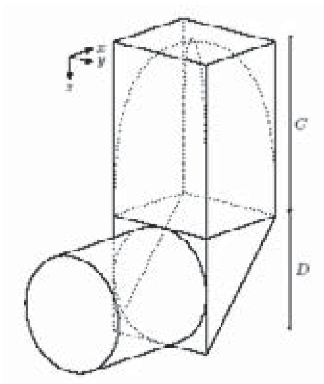


Figure ap.4.1 The tunnel-free stability model of the method of Horn.

4.2 METHOD OF MURAYAMA (1966)

The soil weight (q_B) acting on the pressure wedge (*abd*) is calculated in accordance with the Terzaghi (1943) theory, the failure surface is a logarithmic spiral (Fig. ap.4.2).

The face stability requires the equilibrium between the moment of the acting weight forces (q_B+W) and the resistant forces [force applied on the tunnel face (P) and shear strength along the failure surface].

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

The basic equation is:

$$P = [W \times l_W + q_B \times B_1 \times (l_B + B_1/2) - c(r_d^2 - r_a^2/(2\tan\varphi)]/(2R \times l_p) \qquad \text{(ap.4.1)}$$

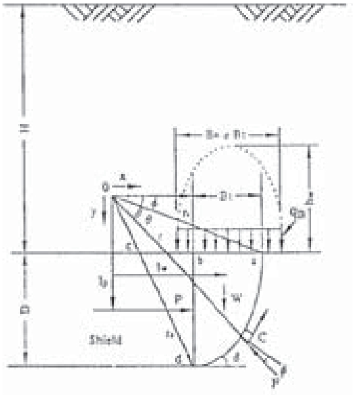


Figure ap.4.2 The tunnel-face stability model of the method of Murayama.

4.3 METHOD OF BROMS & BENNEMARK (1967)

It provides a relation for the stability analysis of an unsupported opening in a cohesive un-drained material (Tresca criteria, 1864). The stability ratio N is defined as:

$$N = (q_s - \sigma_T)/c_u + (C + R)\cdot \gamma/c_u \tag{ap.4.2}$$

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

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

$$\sigma_T = \gamma \cdot (C + R) + q_s - N \cdot c_u \text{ with } N \approx 6 \tag{ap.4.3}$$

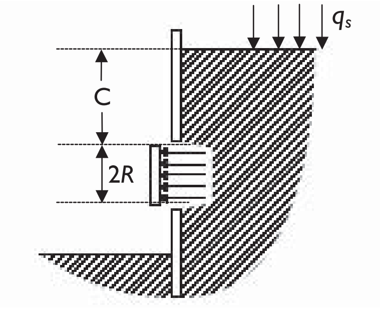


Figure ap.4.3 The tunnel-face stability model of the method of Broms & Bennemark.

4.4 METHOD OF ATKINSON & POTTS (1977)

The minimum support pressure for an excavation face in incoherent drained soil is determined considering two limit conditions: 1) $\gamma = 0$ *e* $q_s > 0$; 2) $\gamma > 0$ and $q_s = 0$, where γ = soil density and q_s = 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:

$$s_{min} = [2k_p/(k_p^2 - 1)] \times \gamma \times R \qquad \text{(ap.4.4)}$$

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

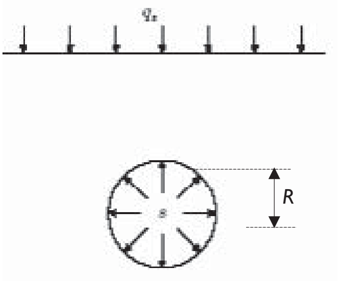


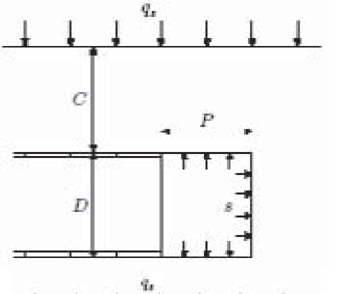
Figure ap.4.4 The tunnel-face stability model of the method of Atkinson & Potts.

4.5 METHOD OF DAVIS ET AL. (1980)

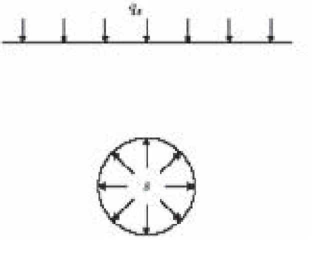
This method allows the stability analysis of a tunnel with radius *R*, in a cohesive soil, where a rigid support is installed at a distance P from the face. Lower and upper limit solutions under general conditions are provided through diagrams, and two particulars cases are analysed: $P = \infty$ (Fig. ap.4.5b) 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:

$$N = 2 + 2\ln(C/R + 1) \text{ [cylindrical]} \qquad \text{(ap.4.5\}}$$

$$N = 4 \cdot \ln(C/R + 1) \text{ [spherical]} \qquad \text{(ap.4.6)}$$



(a)



(b)

Figure ap.4.5 The loading schemes of the method of Davis *et al.*

4.6 METHOD OF KRAUSE (1987)

It provides the minimum support pressure for the different failure mechanisms reported in the Figures ap.4.6 (a), (b), and (c).

The model with the failure surface consisting of a quarter of circle (Fig.b) gives the maximum value of the stability pressure:

$$s_{min}[\max] = (1/\tan \varphi) \cdot (D \times \gamma'/3 - \pi \cdot c/2) \tag{ap.4.7}$$

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

$$s_{min} = (1/\tan \varphi) \cdot (D \cdot '/9 - \pi \cdot c/2) \tag{ap.4.8}$$

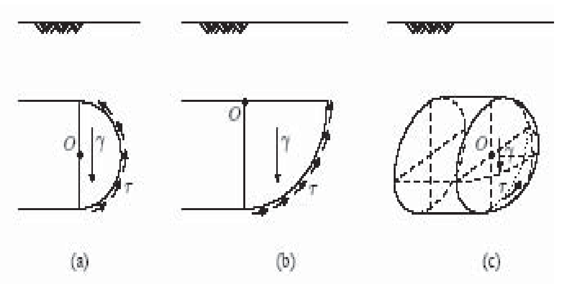


Figure ap.4.6 The various modes of instability as assumed for the tunnel face in the method of Krause.

4.7 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. ap.4.7a & b) and the other involves the tunnel wall (Fig. ap.4.7c), along the failure surface, respectively, logarithmic spiral and cylindrical.

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

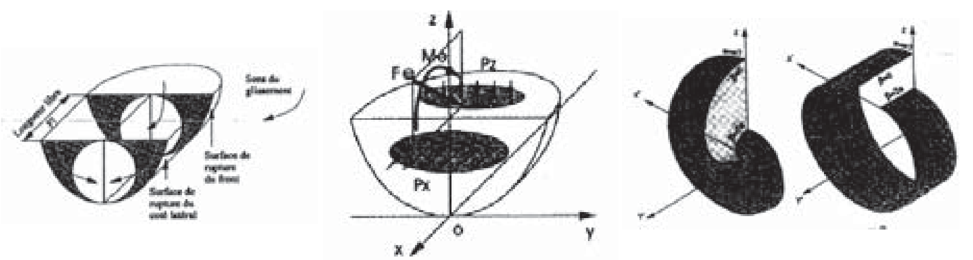


Figure ap.4.7 The failure mechanism assumed for the method of Mohkam *et al.*

4.8 METHOD OF LECA & DORMIEUX (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 σ_T is (Ribacchi, 1994):

$$\sigma_T = - c' \cdot ctg\varphi' + Q_\gamma \cdot \gamma \cdot D/2 + Q_s \cdot (\sigma_s + c' \cdot ctg\varphi') \qquad \text{(ap.4.9)}$$

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

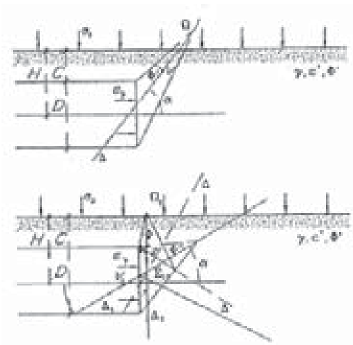


Figure ap.4.8 The tunnel-face stability model of the method of Leca & Dormieux.

Note: a third failure mechanism refers to the so-called "blow-out" failure in very shallow tunnels (where σ_T is so large that the soil is heaved in front of the shield).

4.9 METHOD OF JANCSECZ & STEINER (1994)

According to the model of Horn (1961), Method 1, the three-dimensional failure scheme shown in Figure ap.4.9 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.

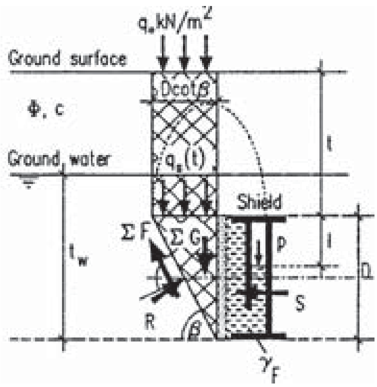


Figure ap.4.9 Method of Jancsecz & Steiner scheme.

A three-dimensional earth pressure coefficient k_{a3} is defined as:

$$k_{a3} = (\sin\beta \cdot \cos \cdot - \cos^2\beta \cdot \tan\phi - K \cdot \alpha \cdot \cos\beta \cdot \tan\phi/1.5)/(\sin\beta \cdot \cos\beta + \sin^2\beta \cdot \tan\phi) \quad \text{(ap.4.10)}$$

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

4.10 METHOD OF ANAGNOSTOU & KOVARI (1994 & 1996)

4.10.1 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 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') is:

$$\sigma' = F_0 \cdot \gamma' \cdot D - F_1 \cdot c' + F_2 \cdot \gamma' \cdot \Delta h - F_3 \cdot c' \cdot \Delta h/D \quad \text{(ap.4.11)}$$

where F_0,F_1,F_2,F_3 are non-dimensional factors derived from normograms, which are function of H/D and φ'.

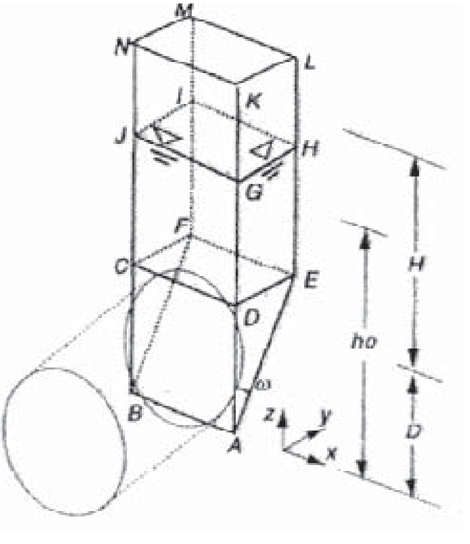


Figure ap.4.10 The tunnel-face stability model of the method of Anognostou & Kovari.

Note: The original analysis considers two values of k_0, 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.

4.10.2 Solution for Slurry Shield

In case of a Slurry Shield, the work pressure (p_b) must be greater than the external water pressure (p_W) in order to avoid the water flow in the plenum. The stabilization pressure or, the delta pressure (Δp), depends on the degree of penetration of the bentonitic slurry in the soil. The minimum value of Δp is associated with the formation at the face of an impermeable membrane, the cake, (case "a" in Fig. ap.4.10.2). 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.

The symbols used in the above equations are showed in Fig. ap.4.10.1.

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

The German norm DIN 4126 suggests, moreover, the following empirical formulation: $f_{so} = 2\tau_f/d_{10}$ where τ_f is the shear strength of the bentonitic slurry and d_{10} 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.

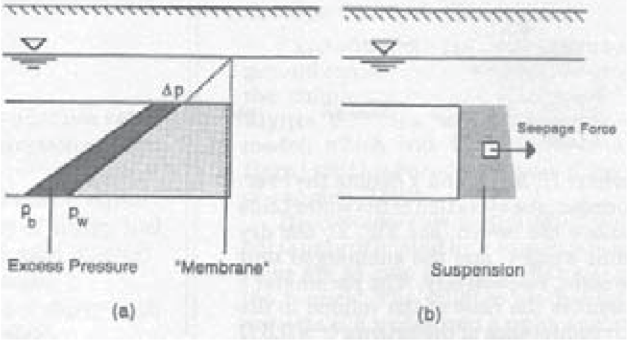


Figure ap.4.11 Membrane and filtration model (solution for slurry shield).

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 ($f_s = f_{scr}$ 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.

4.11 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.

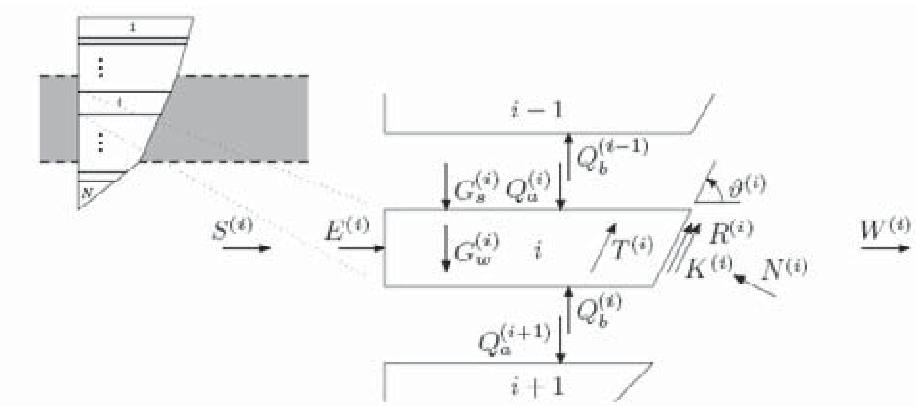


Figure ap.4.12 Definition of symbols for the multilayered wedge model of the Broere method.

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, suggest that part of the column above the wedge does not act as a load on the wedge.

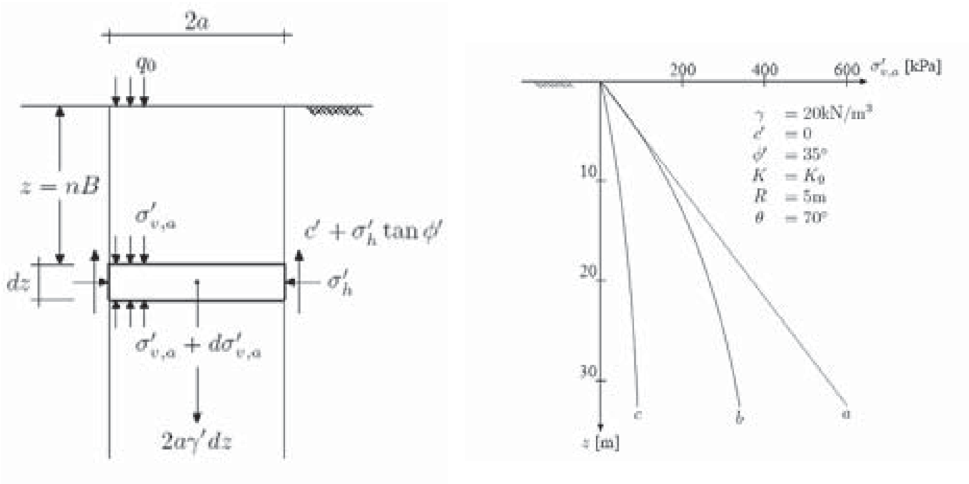


Figure ap.4.13 Definition of forces acting on a strip of soil in an arching soil column according to the Terzaghi theory (Broere, 2001).

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

$$\sigma'^{(i)}_{v,a} = \frac{a\gamma'^{(i)} - c'^{(i)}}{K^{(i)} \tan\phi'^{(i)}}\left(1 - e^{-K^{(i)}\tan\phi'^{(i)}\frac{z}{a}}\right) + \sigma'^{(i-1)}_{v,a}(t_i) \cdot e^{-K^{(i)}\tan\phi'^{(i)}\frac{z}{a}} \qquad \text{(ap.4.12)}$$

The symbols used in the equation are explained in Figures ap.4.11 and ap.4.12.

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. ap.4.11.2. Furthermore, specific considerations have been formulated also for the evaluation of the horizontal stresses acting along the wedge sides.

4.11.1 Vertical-stress distribution under the different assumption of arching (Broere, 2001)

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 4.10 by A-K), different mechanisms can occur

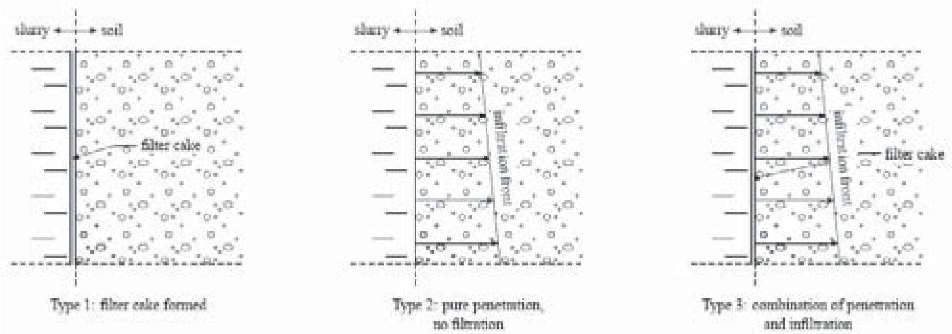


Figure ap.4.14 Typical cases of slurry infiltrations (Broere, 2001).

depending on the permeability of the soil and the density of the support medium (see Fig. ap.4.13).

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^{-3}m/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:

$$s'_{(zt)} = [G_s - P_s + G_w + K' + 2T' - 2P_T + S'_{\text{dev}}]/Z \qquad \text{(ap.4.13)}$$

where: G_s = overburden force on wedge; P_s = uplift force due to excess pore pressure; G_w = effective weight of the wedge; K' = effective cohesive force along sliding surface; T' = resultant friction force on the wedge side; P_T = 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:

$$s_{(z)} = s'_{(z)} + p_{(w(z),z)} \qquad \text{(ap.4.14)}$$

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 p_0.

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 Onderground Bowen) during the construction of three tunnels in Netherlands (2 by Slurry Shield and 1 by EPBS) 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.

4.12 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 p_s can be applied inside the tunnel, while a surcharge q_s (from infrastructures or embankments) acts on the ground surface. For the situation presented in Figure ap.4.12 Caquot's solution defines the value of internal pressure (p_s) as the minimum or critical pressure below that the tunnel will collapse. The Caquot generalised solution for dry conditions (which include the factor of safety, *FS*), can be represented by the following equation developed by Carranza-Torres (2004):

$$\frac{p_s}{\gamma a} = \left(\frac{q_s}{\gamma a} + \frac{c}{\gamma a}\frac{1}{\tan\phi}\right)\left(\frac{h}{a}\right)^{-k\left(N_\phi^{FS}-1\right)} - \frac{1}{k\left(N_\phi^{FS}-1\right)-1}\left[\left(\frac{h}{a}\right)^{-k\left(N_\phi^{FS}-1\right)} - 1\right] - 1\frac{c}{\gamma a}\frac{1}{\tan\phi} \qquad \text{(ap.4.15)}$$

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

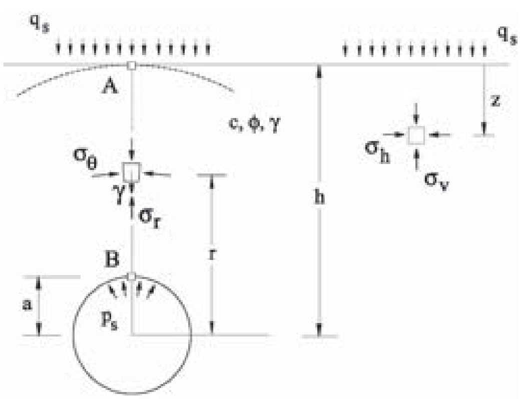


Figure ap.4.15 Basic scheme for the Caquot-Kerisel solution (Carranza-Torres, 2004).

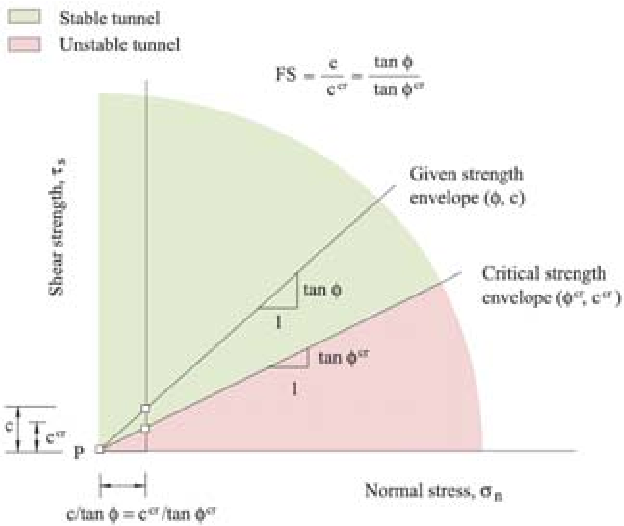
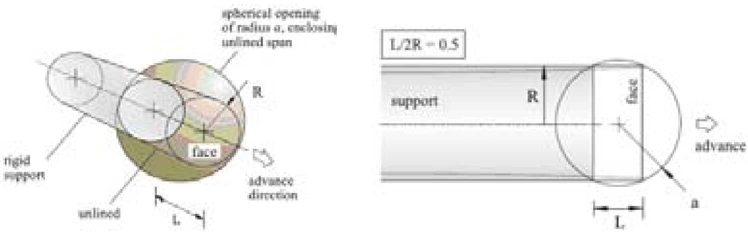


Figure ap.4.16 Strength reduction method (Carranza-Torres, 2004).



$$a = \sqrt{\frac{L^2}{4} + R^2}$$

Figure ap.4.17 Calculation of the modified tunnel radius for face stability analysis (C. Carranza-Torres, 2004).

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.

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 ap.4.15, this approach assumes a proportional reduction of the Mohr-Coulomb parameters.

$$N_\phi^{FS} = \frac{1+\sin\left(\tan^{-1}\dfrac{\tan\phi}{FS}\right)}{1-\sin\left(\tan^{-1}\dfrac{\tan\phi}{FS}\right)} \qquad FS = \frac{c}{c^{cr}} = \frac{\tan\phi}{\tan\phi^{cr}} \tag{ap.4.16}$$

For the stability analysis of tunnel face, a procedure to take into account the tridimensional effect of tunnel face and the eventual unsupported distance (L) is suggested by the Author, as represented in Figure ap.4.16.

Appendix 5

An example of risk management plan for a job using slurry shield

1 INTRODUCTION

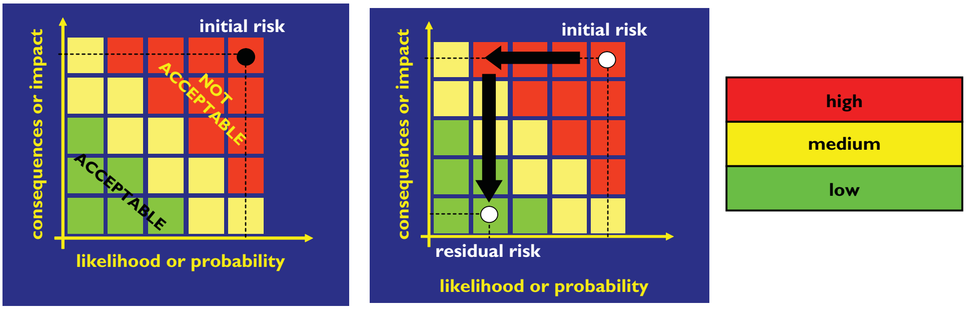
The scope of this appendix is to provide a plan or methodology for risk management in a job using slurry shield for tunnel excavation. The plan refers to the ground and construction conditions related to the project "St. Petersburg Metro", Line 1 described in Section 8.2.

The job-associated risks are divided into four main categories according to the sources of these problems:

1. Soil.
2. Lining.
3. Resources.
4. Existing structures.

The next section gives a summary of the application of risk management plan, with the results arranged in the format of a table for each category of the risks. Each of the four tables is divided into two parts, A and B, in order to maintain the vertical format of the page. Part A refers to the events (hazards), background (main features), and assessment of the risk (as a function of likelihood of occurrence, consequence or impact, and rating of the initial risk: high, medium, or low). Part B refers to the aspects of detection of the events, definition of the mitigation measures, and rating of the residual risk after mitigation.

At this point, it is useful to recall the differences between the mitigation measures and countermeasures. The "mitigation measures" consist of a set of predefined



measures to be systematically implemented at various stages of a project in order to reduce each unacceptable initial risk, with respect to the acceptability criteria, by acting on its probability and/or its impact. The risk remaining after the implementation of the mitigation measures is called residual risk. On the other hand, "countermeasures" are the actions, defined at the design stage, which will be activated during construction according to the predefined triggering-criteria, should the key-parameters reach the predefined alarm-thresholds.

Table ap.5.1A Risk category: soil

Event No	*Event or Hazard*	*Background (Main features)*	*Assessment of the risk*		
			Likelihood of occurrence	*Consequence or impact*	*Initial risk*
1	Face stability versus sand liquefaction	Incoherent water-bearing sand with variable characteristics, ancient beds of the valley subjected to the glaciations effect. Likelihood of face instability is considered as low with no other cause of possible liquefaction. The consequence is of a major impact	low	high	medium
2	Face stability versus sand lenses	High density sand and/or highly consolidated sand, which appear either in form of thick banks (some meters) or thinner lenses confined in clay or silt strata. The instability can be defined in terms of loss of face stability; settlements in the surface and/or deformations in some utility creation of chimneys; unexpected loss of volume. The likelihood of occurrence of such natural soil is high. The consequence is of a high impact.	high	high	high
3	Face stability versus high pressure	Maximum operating front pressure of TBM more than 5.5 bars is source of the higher pressure detected. Front pressure is fundamentally chosen based on the hydrological and geomechanical conditions. While the likelihood of this event is low, the consequence is of a major impact.	low	high	medium
4	Face stability versus transition clay to sand	This transition occurs rarely in sub-vertical strata, rather occur in sub-horizontal strata. The sub-horizontal transitions between compacted argillite and sandy strata embedded into clay layers are commonly encountered; in zones internal to the glacial valley, it is common to find transitions between over consolidated clay and lenses or channel of sand.	medium	low	low
5	Excavation soil stuck to the cutterhead	Due to the consistency differences between excavated ground (especially cohesive soils), it is possible for a TBM cutterhead to got stuck. This event can be considered as potentially likely, however, at the same time the relevant consequences are minimized by countermeasures already installed in the machine.	high	low	medium
6	Finding boulders	In morenic soils, boulders can be easily found, in variable sizes according to the thickness of the stratum and decreasing as much as depth increases. If the tunnel crown is excavated somehow closed to the lower part of the morenic level along the tunnel alignment, the occurrence of boulders will be limited to the upper part of the tunnels cross section. The probability of being encountered with small and big size boulders is high and low, respectively.	high (small size) / low (big size)	medium (small size) / high (big size)	medium
7	Finding artificial interferences	Existence of previously drilled boreholes in the site investigation period at depth of tunnel alignment can be regards as the artificial interferences because the slurry caked from in front of the cuttrehead can easily run away to the borehole cavities, leading to a sudden distortion in the slurry and compressed air balance. If not avoided, the face undergoes instability. Those borehole were perforated with steels have a severe impact on cuttrehead damage. The probability of being suddenly encountered with such interferences is low, however, major impact would follow if such event occurs, due to sudden chamber pressure decrease and face instability.	low	high	medium

Table ap.5.1B Risk category: soil

Event No	*Detection of the event*	*Definition of the mitigation (Measures)*	*Residual risk**
1	TBM penetration rate, confinement pressure, return slurry density control, mucked soil at the separation unit, identifying water content, surface subsidence control, Inclinometers and extensometers measurements would indicate localization and relevance of this occurrence.	Natural liquefaction itself cannot be deleted nor controlled. The principles of risk analysis do not consider appropriate implementation of the specific resources for this event except, of course, the respect and the strict application of the operating procedures carried out in a quality control process.	medium
2	Slurry confinement pressure in chamber, measurement of solid mucked away soil, slurry level, density of the mucked materials, penetration rate, surface settlement control, variation of piezometric levels and unforeseen measures of inclinometers and extensometers installed from the surface, quantity of mortar for grouting operation of segmental lining.	The different possible solutions are related to soil intervention carried out from the surface or from the tunnel under excavation. Should the standard procedures not be enough to prevent local instabilities of the face, the cavity created would be filled by slurry that replaces the mucked soil. The residual risk associated with the void remained can be mitigated by filling up during segments grouting operation taking into account of monitoring volumes injected and relevant pressure. Equipped TBM with high pressure injector (preventers with 6 to 10 bars).	medium
3	Level of slurry in the chamber, slurry confinement pressure in the chamber, measurement of solid mucked away soil, penetration rate, surface settlements monitorde by topographic measures, variation of piezometric levels and unforeseen measures of inclinometers and extensometers installed from the surface.	High pressures are due to the water and/or to the soil, therefore actions to reduce directly those factors have to be done. Ground consolidation should be carried out, improving the mechanical characteristics of the soils and lowering the relevant permeability. Another solution to reduce high pressures could consist in lowering the water table by installation of a relief wells system from the day surface.	medium
4	Composition of the mucked materials recorded at the mud treatment plant, slurry levels of probes placed on each side of the main bearing, mud outflow parameters (density variation in time, flow, …), torque.	Continuous monitoring of TBM and slurry circuit parameters, good co-ordination between the operating and follow up teams and experiences gathered during tunnel advancement.	low
5	Discrepancy in the theoretical value of excavated materials for the relevant advancement, underpressure in the slurry return circuit, minor flow through the return pump, minor density (increase of water contents), torque increase, slurry levels on probes located on each side of the main bearing.	To avoid this phenomenon, which is more probably localized in the centre of the face, where openings are necessarily reduced and rotation speed is lower, pipes to inject water and/or an aggregating admixture will be installed. These are lubricant and detergent specially designed for hydroshield. Other injectors are mounted as well in order to clean the picks by means of bentonite pressure jets.	medium
6	A possible way to detect presence of boulders in the face is throughout sensibility and experience of the operator, who would realize an abnormal TBM behaviour, such as vibrations, noises, asymmetrical advance or stoppage. In case boulders partially block the return flows, immediate increase of pressure and bentonite level in the chamber will be detected. Geophysics methods carried out ahead of the tunnel face are the other appropriate means.	Equipped with the crusher, TBM is capable of driving and dealing with small-medium sized blocks (around 500–700 mm). Encountered with bigger boulders, it would not be enough to installed also the water and bentonite sprinklers, rather it is practical to access to front of the cutterhead for a mechanical demolition. However, in the case of latter mitigation, safety requirements (i.e. working pressure maximum 3 bars) and soil treatments must be taken into consideration.	medium
7	In case the machine will cross (and cut) a borehole where some instrument is installed, the fact will be marked by the relevant reading.	Using plastic or aluminum casing for investigating boreholes, all boreholes placed within the tunnel alignment must be recorded and they have to be cleaned and plugged cautiously prior to the departure of TBM.	medium

(*) The residual risk after the mitigation can fall in the medium or low levels. The residual risk at the low level is acceptable. If the residual risk of the medium level is not acceptable (see section 2 of the main text of the book), the predefined countermeasures should be applied, during construction, when the key parameters reach predefined alarm thresholds.

Table ap.5.2A Risk category: lining

Event No	*Event or Hazard*	*Background (Main features)*	*Assessment of the risk*		
			Likelihood of occurrence	*Consequence or impact*	*Initial risk*
1	Segments geometrical wrong installation	Each segment will be erected and linked to the previous elements by means of conex. Conex are capable to assure both tensile and shear resistance and therefore they may guarantee a correct installation, with minimum tolerances (10 mm max). A wrong installation might appear in two different time i.e. after mounting by erector and after the mortar injection around the ring. In each case, the consequences could affect the stresses on the concrete and/or the reinforcement and the watertightness of the lining along the joints. The likelihood of occurrence of this risk is between unlikely and likely while the consequence of this risk, related to the structural aspect, falls in a major.	medium	high	high
2	Ring ovalization	Abnormal ground behavior or asymmetrical confinement could bring some section to figures that reach and overpass the pre-defined thresholds deformation values. This effect is probable were sandy meanders are present and whether their course might be sub-parallel to the tunnel alignment. Design consideration causes the probability of this event to be of very rare, however, the consequences of this event is very serious.	low	high	medium
3	Lining ring floating	If the mortar hardening (which in this specific case may be identified with the capability to avoid internal material flows) will begin within approximately 8 to 10 hours since its injection, the likelihood is actually low. Otherwise, a first consequence of offset position (the difference between axis of lining and tunnel) will be an effect of abnormal pressure on the upper shield brushes, and at the same time possible flow out of bentonite in the bottom contact ring-brushes: this may lead in turn to loss of pressure in the chamber and therefore the instability risk mentioned above.	low	low	low
4	Segments watertightness	The appropriate concrete mix definition and curing process in the design of the precast segments, guarantees the watertightness of those elements. Specific laboratory tests verify segments for installation. Therefore, no leakage should appear along the tunnel throughout the segments. Nevertheless, it is impossible to be 100% certain for some potential conditions for leakage not occurring due to 1) laboratory tests are punctual and some dispersion of values and quality level is normal in each industrial production 2) induce fissures stem from uncontrolled handling that escape from visual control and 3) the high levels of water pressure at the tunnel depth. The occurrence may fall in between likely and unlikely, however, because the consequences of this risk are quite small, the overall risk rating would be low.	low to medium	low	low
5	Joints watertightness	Joints are the most delicate point of a segment in terms of structural and watertightness aspects. The foreseen hydraulic conditions and mounting sequences should overcome any risk of water inflow inside the tunnel during the exploitation phase. In order to increase the safety factor against an event whose occurrence is not remote and consequences not negligible, a preventive measure has to be applied to reduce the risk of water leaking. It consists of an additional defense made of a double system: a hydroswelling profile nearby the extrados (dedicated to a long term effect) and a band of swelling grease (for a short term action) spread in the space between the two aforementioned gaskets.	medium	medium	medium
6	Segments overstress	The long-term (permanent) stability of a tunnel is mainly based on correct final lining. Therefore, the risk of external load that may induce inappropriate stress in the precast segments must be taken into account and a adequate safety factor must be chosen. Due to the seriousness of the problem and of its consequences, the construction process envisages measures capable to monitor the stresses that the segments are subjected to and, previously, the real status of the segments themselves.	low	medium	low

Table ap.5.2B Risk category: lining

Event No	*Detection of the event*	*Definition of the mitigation (Measures)*	*Residual risk**
1	Control of the occurrence of this phenomenon in visual.	Strict procedure of installation, continuous control of the segment injection operations and applied pressure, maximum care as regards mechanical operations, quality control of conex elements and on concrete resistance, experienced follow up team and designers. Special actions such as especial mixes for segment grouting or installation of steel ribs are to be taken if necessary.	medium
2	Assiduous topographical check section by section will give monitoring all along the tunnels. Specific instruments (marks on surface, pressure cells, strain gauges) are furthermore installed in predefined sections: they will measure both geometric conditions and stresses.	No measure that can mitigate the risk can be reasonably found. The remedial countermeasure to the occurrence of the detected problem consists in physically blocking the ovalization. For this purpose a series of circular steel ribs shall be available on site for their prompt installation against the intrados of the ring, they will be forced against the lining by means of jacks.	medium
3	This fact would be immediately detected by the personnel of operating team, mainly due to the high pressure of the blowing out bentonite. Topographical surveys will define exactly the absolute lining displacement, whose effect is temporarily and physically limited by the shield itself.	A general mitigation measure (modification of the injection scheme) shall be implemented. From the other hand, again, the respect of the operating procedures – in this case applied to the mortar characteristics (including capability to avoid pipes cloggage and mortar washout in the presence of water) – corresponds to the sole condition to keep this risk into low levels.	low
4	The only way to detect the occurrence of this event is visual and when it actually occurs. In fact, it is unlike to monitor any problem during mortar grouting being injected around, on the contrary, this operation will result benefit as the fine and cementations contents will eventually penetrate and close existing flow path across the segments.	Once the possible future problem have been detected through the segment control in the production plant, mitigations are restricted to a surface treatment with resins and/or waterproofing paintings, as referred in specific procedure. Furthermore, local repairs could be carried out on the internal face of the segments, even during the exploitation phase. Hence, the main influence is on cost and quality control level.	low
5	There is no way to monitor this event before it actually occurs. When mortar around the segments is under injection, some fine contents will be capable to cross the first "barrier", which doesn't react immediately, reaching the second one (grease) that – on the contrary – immediately expands in touch with water and blocks any further movement towards the tunnel. Consequently, the phenomenon of the matter will appear (if any) only in a later stage.	Three different actions can be undertaken: 1) local injections behind the damaged joint. 2) the installation of an emergency circular steel rib, coupled with a membrane that will be strongly kept between the leaking joint and the rib extrados; it is also envisaged the application of a shaped steel plate equipped with hydro-tight seal, to be fixed to the lining by means of anchor bolts. 3) direct repairs on the internal face of the lining and along the damaged joint, with a direct blockage of the water flow path.	medium
6	The installation of some instruments inside the monitored segments becomes indispensable to be able to detect the risk and to correlate between external loads and lining behavior and opportunity to develop interpolations or extrapolations, eventually updating and examine some back analysis.	Monitoring team will interfere immediately with the follow up team to follow the development of the instruments' measures in case they start to reach values higher than foreseen. In case this process will not stop, the designer shall be informed and, jointly with the project management corrective actions could be undertaken. The most immediate will consist in reinforcing that cross section by means of steel ribs. Eventually, as a limit countermeasure, localized consolidation grouting could even be carried out, by drilling through the lining and injecting cement based contents in order to improve the soil characteristics in that area.	low

(*) The residual risk after the mitigation can fall in the medium or low levels. The residual risk at the low level is acceptable. If the residual risk of the medium level is not acceptable (see section 2 of the main text of the book), the predefined countermeasures should be applied, during construction, when the key parameters reach predefined alarm thresholds.

Table ap.5.3A Risk category: resources

Event No	*Event or Hazard*	*Background (Main features)*	*Assessment of the risk*		
			Likelihood of occurrence	*Consequence or impact*	*Initial risk*
1	TBM	Risk of mechanical or electrical failures of the front and shield sections of a TBM must be considered. Especial attention must be taken for discs in soft or clayey grounds where they cannot properly rotate and tend wear out or even break. Due to existence of high clay content usually found in different strata, the probability of these events to take place is between unlikely and likely and the consequence is medium due to difficulty in repairing the discs. Those equipment related to safety aspects such as of devices applying counter pressure to the face and keeping it constant with appropriate values, must be double checked and if necessary the emergency devices must be installed and spare parts on site must be available. The appropriate slurry level and air pressure must be kept constantly carried out by operator and his assistances with references to technical documents given to them. All the pieces have been checked before its start up and will be checked again in the chamber after the first run.	medium	medium	medium
2	Segments grouting system	This point is related to the risk consequent to failures of the grouting system around the mounted ring. The injecting system is managed by an automatic program which as well gives information about both the pressure and the quantity of injected mortar per each injector. Risks may be associated to the following: 1) mechanical failures 2) lack of power 3) lack or inappropriate mix of mortar supply 4) delays in injection performances 5) inappropriate injection pressure. Delays both in starting and during the performance of injection activity could have negative consequences on the mortar characteristics and on the circuit pipes inside the shield part, whose obstruction could cause serious problems. Improper pressure could caused either incorrect movements of the ring due to injection or insufficient backfilling.	low	low	low
3	T.B.M. back-up and service equipments installed	Back up is composed of six wagons. Auxiliary equipments installed on it are: grease pumping system; segments mortar injection pumping system; main and emergency lighting system; cooling system; secondary ventilation system; stocking area and handling system for mud circuit; cables and utilities reels; ducts and cables waterproofing control; fire protection system; high and low voltage boards; transformer; segments portal crane; segments conveyor belt. Taking into account 1) all the elements that make part of tunnel excavation and lining installation are guaranteed by the relevant supplier; 2) before the start of the machine all relevant tests (single and integrated) shall be performed; 3) daily controls and standard maintenance will be carried out, according to predefined procedures and with clear reference to actions and responsibilities; 4) all the foreseeable spare parts will be available on site, the rating risk would be at low level.	low	medium	low
4	Slurry circuit and muck away system	The system has to provide adequate pressure to balance earth and water pressure at the tunnel face. The constraints associated with the production are: TBM advancement rate(amount of soil to be evacuated and then treated), capacity of the treatment unit(slurry production with appropriate density), and pipe diameter (appropriate mucking speed by slurry conveyance ducts). It is foreseen a figure of 900 m^3/h as nominal value for mucking product (excavated soil mixed to slurry) with a value of 1.000 m^3/h as extreme condition. All the operation phases of the slurry circuit are strictly managed by knowing information about the slurry in the feed circuit, slurry in the chamber and finally muck in the return ducts). As a result, the whole system responds to strict control, both automatic and manual, with emergency devices installed on the machinery. It means that any unexpected event that can damage the construction processes is low.	low	medium	low
5	Slurry production and treatment unit	The amount of material to be treated by this unit is 900 m^3/h, but a peak of 1.000 m^3/h must be guaranteed. Due to the external constrains (geometrical, slurry and soil characteristics, TBM advance rate), a margin of approximately 25% at least is then envisaged. Slurry production must be sufficient not to limit the requirements of TBM advance: that is referred of course to all the successive steps in which slurry is involved, from production itself (dry bentonite storage in silos, mixing plant, fresh slurry deposit pools, primary pumps) up to the proper treatment (cyclones and other refreshing equipments) for its recovery; it means to dimension the unit according to peak moments, e.g. with maximum advance rate of the TBM and when mucked soil has the maximum contents of fines. Furthermore, taking into account that this plant is constituted of two independent units, the excavation progress is anyway granted – even if in reduced speed – also in case of damages to part of the treatment plant. Therefore the only risk is related to the consequences of some failure or incorrect operation.	medium	medium	medium
6	Human mistakes	A high level of specialization and experience of the personnel represents the main guarantee to limit any kind of operational mistake. The local personnel have been identified through a careful selection along the mobilization period. Experience says that mistakes – and sometimes consequent accidents – happen when organization is weak, machinery is obsolete or insufficiently maintained, personnel is tired for hard working conditions and timetables, progress required by a tight planning is actually overestimated.	low	low	low
7	Lack of resources	The possible lack of resources are attributed to : Personnel; Machinery; Materials (segments accessories, monitoring instruments; Consumables (power, water, and bentonite) and Third Parties (segment manufacturer and specialized companies). It can be said that for each of the situations, likelihood of occurrence could be likely (this is the rating of risk could be assumed between low and medium, detection of its occurrence easy and immediate, definition of mitigation measures almost automatic.	high	low	medium

Table ap.5.3B Risk category: resources

Event No	*Detection of the event*	*Definition of the mitigation (Measures)*	*Residual risk**
1	Some parts of machine are not functioning, alarm is red.	Definition (such as to detect the event) of mitigation measures corresponds to a standardized procedure. Therefore the actions that shall be taken to lower the risks mentioned for cutters wearing will not be repeated in here.	medium
2	If pressure in some injector is lower than the programmed, automatic warning is given to the operator who can lower the flow in speed. Ring floating can be detected by an operational rotation scheme and sensors installed and risks of incorrect localized values of mortar volumes and pressures are minimized.	An independent generator will easily delete the risks associated to an eventual lack of power. To reach a proper mix design characteristics, comprehensive studies and test must be done. To avoid improper injection pressure, alarm sensors are to be installed apart from the operational rotation scheme. If rings floating risks are detected, it could be convenient give time priority to the upper pipes injections. Control procedures against risks occurrence are: 1) Volumes: injected quantities must be correlated with the theoretical ring voids 2) Pressures: check whether the final pressure is consistent to the design reference, which is correlated to the confinement pressure 3) Check that mortar respects constantly the design characteristics.	low
3	Some parts of machine are not functioning, alarm is red.	A good job site organization in terms of back-up, pre-check all parst before starting excavation, provision and presence of a professional mechanical team in job site, prediction and provision of adequate amount of spare parts.	low
4	Alarm is red when an abnormality in chamber or slurry circuit is detected by densimeter, pressure sensor, flow meter, electric level detectors.	Continuous monitoring of pressure and density plus laboratory tests on viscosity, filtrat, yield and pH will be carried out and recorded at the control panel. Specific densimeters are installed along the circuit and at the excavation front for automatic measurements. Difference between flow in and flow out density – varying according to the ground – can give information about the excavated soil and eventual presence of water coming out from the excavation face. Pressure sensors will monitor both pressure at the top of the chamber and in the return circuit. Quantities of product entering and coming out of the chamber will also be verified by means of flow meters. That measure permits an analysis of excavated ground quantities and verification of the mucking speed. The level of slurry inside the chamber will be managed and measured by electric detectors and must be kept constant . Visual monitoring is possible, by means of colour LED indicators, for attention and alarm thresholds. This monitoring system will detect, if low levels are shown, abnormal slurry absorption at the face or/and over excavation.	low
5	A mechanical problem occurs in the slurry treatment unit and alarm is red.	Existence of a specific and equipped laboratory that will test and control the slurry parameters is mandatory. It is necessary to verify preliminarily the characteristics of the water that will be used for the bentonite slurry preparation, through careful testing, in order to verify its saline contents, Ph, etc.	medium
6	Unforeseen.	Prior to operational activities, personnel will be properly instructed through two different learning/improving/updating courses. A good organization must be made to foresee any out-of-rules conditions. Attention is immediately called by monitoring system to the predefined personnel responsible and the coverage to all possible events given by overlapped teams. It will guarantee additional control versus possible human mistakes. A brand new hardware and software system is to be applied for each specific project.	low
7	Re-examine the site organization chart, responsibilities, and required resources during first stage of the work.	The following are mitigations to be well-provided according to possible lack of resources: organization chart with flexibility for unforeseen events, adequate number of spare parts tested on factory and job site, segments' accessories such as sealing gaskets, conex, guiding rods, mortar and its various components, instruments, market research and purchase orders, transportation, suppliers directly influence the TMB namely electrical power, water and compressed air supplies.	medium

(*) The residual risk after the mitigation can fall in the medium or low levels. The residual risk at the low level is acceptable. If the residual risk of the medium level is not acceptable (see section 2 of the main text of the book), the predefined countermeasures should be applied, during construction, when the key parameters reach predefined alarm thresholds.

Table ap.5.4A Risk category: existing structures

Event No	*Event or Hazard*	*Background (Main features)*	*Assessment of the risk*		
			Likelihood of occurrence	*Consequence or impact*	*Initial risk*
1	Existing sewage ducts, water pipes and so on	High degrees of settlement can occur due to interference of tunnel and existing sewage. The tunnel excavation will induce unavoidable vertical subsidence, whose distribution and amplitude will vary from the tunnel depth to the surface. In consequence, the soil surrounding the sewage will be subject to settlements whose value will vary along its development. Resulting displacements and tensions must be predicted and verified based on an adequate reliability of the structure. For the same reasons, settlements of the collector shaft could appear, thus inducing inadmissible movements in the connection joints with the adjacent pipelines. Another risk is the creation of a chimney that would damage the sewer and propagate settlements up to the surface. In this case, a remedial action is filling the voids as quick as possible by gravity or pressured injection from the surface to be carried out by a specialized team.	low	high	high
2	Existing buildings in the surrounding area	The existing buildings in the influence area of the tunnels excavations are main concern. The relevant analyses include: soil characteristics of the ground where the buildings lean; type of buildings foundations; structural characteristics of each building; status of art of each single building. The profile of surface subsidence section by section, surface rotation angle, sensitiveness of surface structure, mechanical and physical characteristics of existing structures and buildings must be clarified. The available design data together with existing information about the buildings, lead to conclude that the induced subsidence will not affect any of those structures, so the probability is unlikely. Anyway, in extremely negative conditions (as it was previously described and in coincidence with loss of volumes/face instability which could create chimneys in the subsoil up to the surface) the consequences would be remarkable.	low	high	medium
3	Road clearance and availability	The public services on the surface must be kept clear during the progress of the Project. The present point is related to the risk that some of events occurring during works obliges to occupy an extra part of the areas adjacent to the tunnels alignment (beyond the zones already occupied by the monitoring layout) so as to drill the boreholes, to install instruments and so on. The risks associated with this point can be identified as: additional monitoring to be performed; ground improvement for tunnel stability; ground improvement for buildings/utilities; relief well system to reduce the underground water pressure; repair and maintenance activities to buildings and roads.	low	low	low

Table ap.5.4B Risk category: existing structures

Event No	*Detection of the event*	*Definition of the mitigation (Measures)*	*Residual risk**
1	Instruments for levels and deformations measurement shall be directly installed onto the sewage duct, and their reading shall be assiduous and co-ordinated. They will be collected and recorded in a safe specific room and even external visual alarm will be installed to warn about eventual alarm values detected. Incremental inclinometers-extensometers and piezometers from the surface will be installed as well, and their measurements cross checked with duct measurements. Finally TBM parameters and tunnel behaviour will be analysed at the same time in order to have a complete panorama. Measurements will start when the excavation face is about 50 m from the vertical of duct and continue increasing up to their maximum frequencies when TBM is below the sewer, then decreasing again.	Propagation of settlements in soil lasts around three days, thereby each decision will taken by extrapolating measurements. Preventive mitigating measures may be studied and performed for the second drive. In fact, the experience gathered during the first tunnel excavation will be applied, indicating ground consolidation is required? In this case, a practical access will be the first tunnel where drilling and injection could be performed with no interference the surface utilities. Ground improvement could be carried out from 1) the tunnel by equipped TBM, but this would cause delay the tunnel progress 2) the surface that would present difficulty in operation with interference with road, utilities, buildings. An intermediate solution could be a preventive action of consolidation grouting performed from the shaft. From the TBM operational point of view: reduction of plasticisation ahead of the face, reduction of plasticisation along the shield, and reduction of shield volume loss, a component of settlement, is ascertainable by controlling the longitudinal pressure grouting through tail-skin shield.	medium
2	Along the tunnels horizontal alignment various instruments will be installed at the surface. Fix points on the walls of the houses, surface topographic points linked to benchmarks, level measurements in transversal sections, increx extensometers and inclinometers, piezometers. Their measurement will interface in real time the parameters of the TBM and behaviour of the tunnel excavation for a necessary correlation that would confirm or update the design assumptions. Of course, sudden collapses cannot be anticipated if not through a deep knowledge of the interaction tunnel/surface behaviour.	It is clearly preferable to take actions working from the tunnel: solutions and resources have been already described in the points related to face instability. According to the measured figures (design will identify the different attention and alarm thresholds), different activities will be performed, starting by stopping the machine and keeping the excavation front stable. The successive control of surface monitoring will define whether other decisions will take place (including houses evacuation for safety reasons). For this reason, once more has to be underline the importance of strict coordination between all the teams involved in the project. Actions from the surface are highly unadvisable and therefore are considered as extreme counter measures at this stage.	medium
3	Re-examine the site organization and responsibilities during first stage of the work.	The eventual mitigation measures that reasonably could be performed are related to local repair works inside or around the houses, with temporary people evacuation and local diversion of pedestrians or vehicles due to restriction of the involved lane or carriageway.	low

(*) The residual risk after the mitigation can fall in the medium or low levels. The residual risk at the low level is acceptable. If the residual risk of the medium level is not acceptable (see section 2 of the main text of the book), the predefined countermeasures should be applied, during construction, when the key parameters reach predefined alarm thresholds.

Appendix 6

Typical excavation procedures for EPB shield

1 INTRODUCTION

The present document defines the operations to be carried out, to correctly advance with Tunnel Boring Machine (TBM) of Earth Pressure Balance Shield (EPBS) type, as well as the excavation controls and the operational sequences needed to face the anomalous situations.

This document applies to each excavation cycle, constituted by the tunnel excavation, during which pre-cast ring erection and backfill longitudinal grouting are also carried out.

The principal reference documents for this Earth Pressure Balance Shield, EPBS, procedure are:

- The technical documentation of the Tender Design (TD).
- The technical documentation of the Final Construction Design (specifically that related to the tunnels).
- The Operating Manual of the EPB-TBM.

The *design data* integral to the preparation of this procedure are:

- The data contained in the Tender Design technical documents;
- All the control procedures foreseen by the Special Conditions (SC), also known as Conditions of Particular Application (CPA).
- The data deriving from further investigation, if any, carried out by the Contractor after award of the Contract.
- The technical data of the EPB-TBM.
- The data relating to the experience so far accumulated in the execution of TBM excavation and lining.

In addition, data should be available regarding the following specific conditions.

- Excavation face support pressure, assessed as the pressure exerted by the excavated material temporary filling the excavation chamber (plenum) and monitored by the pressure sensors installed on the rear bulk-head.
- Injection pressure and volume of the backfill grout behind the lining.
- The plan and profile taken along the tunnel axis of the tunnel route.

- It is noted that excavation face support pressure and the grouting pressure behind the lining are design data that the Contractor has to define, in an appropriate construction design document, to be submitted for the Engineer's approval.

2 ORGANIZATION AND RESOURCES

2.1 Personnel

The functions of the various key-staff for the works described in the procedure are as follows.

The Project Manager (PM) is responsible for the management and supervision of the works being carried out and defines (and clarifies) the technical and/or management aspects of the project.

The Safety Manager (SM) is responsible for controlled distribution (on site) of the project documentation; he evaluates the risks connected with the various activities and prepares a Safety Plan, which he updates continuously in relation to the varied site needs. He is responsible for the training and information of the workers and of all the other procedures required by the law in his sphere. He also collects and keeps safe all the certifications, all the user manuals, and whatever is necessary for the proper management of the plant and equipment present on site.

The Tunnel Manager (TM) (or Tunnel Superintendent, TS) reports directly to the PM and is responsible for the planning, budget, and performance of all the underground works, for all the preparatory works and for the supplies necessary to perform the work.

The EPBS Superintendent (or Machine Superintendent, MS) reports directly to the TM and is in charge of the appropriate management of all personnel and plant and equipment employed for the tunnel excavation. The MS supervises both production and plant maintenance, plans with the Work-shop Master Mechanic (WMM) the maintenance under his supervision, and guarantees during shift change that the information is correctly transferred between homologous personnel.

The Shift Superintendents (SS), one for each working shift, are responsible for carrying out the following operations, under the supervision of Machine Superintendent:

- organization of the work activities strictly connected with the excavation,
- precast-lining segments ring erection, and
- grout backfill behind the lining.

The EPB Machine Operator (MO) is responsible for the following operations:

- face-support pressure control,
- control of the weight and volume of the extracted material coming from the excavation chamber,
- excavated ground conditioning, according to the instruction received from the MS, and
- control whether any anomalous excavation situations occur.

The Erector Operator (EO) is responsible for the correct erection of the precast rings according to the positioning advised by MS.

The Lining-backfill Operator (LO) is responsible for:

- grout supply control,
- injection pressure control, and
- injected volume control.

2.2 Plant and equipment (EPBS components)

The principal plant and equipment necessary for making up the EPBS and its back-up are:

- Cutterhead.
- Excavation Chamber, or Plenum, which contains temporarily the muck excavated from the face, thus giving support to the face.
- Screw conveyor: evacuates the muck (ground + additives) from the excavation chamber.
- Conveyor belt scales: provide the cumulative weight value for each excavation cycle and the instantaneous debit of the conveyor.
- Pressure sensors: supply the face support pressure values, e.g. earth pressures in the excavation chamber.
- Precast segments erector: positions the precast segments, thus erecting the lining ring.

2.3 Monitoring and parameters control of the EPBS

The controls are composed of the following actions that are carried out by the relevant dedicated staff:

- Surface monitoring and subsidence measurement (Monitoring Office, MO).
- Graphic compilations of the excavation data of the EPBS (EPBS control office).
- Comparative analysis of the EPBS data and of the monitoring data for cyclical interpretation and detail design (Design Consultant); if anomalies are registered during tunnel advance, the Design Consultant, must advise TM and PM. In grave cases, he can oblige the MO to comply with procedures, and can go as far as ordering the EPBS stoppage, if necessary.
- Daily reports elaboration (Machine Superintendent Assistants, MA).

2.4 Materials used

The main materials necessary for ground conditioning, the tail grease, and the backfill grout in this procedure are:

- Foam derived from tension-active agent, used to treat the ground at the face and/or in the excavation chamber and/or in the screw conveyor, to reduce:

  - ground plasticity and stickiness,
  - head torque,
  - muck permeability in the plenum,
  - screw conveyor torque, and
  - cutterhead friction.

- Bentonite, normally used to:

  - make the excavation face impermeable to compressed air, thus permitting to empty the plenum to allow hyperbaric maintenance operations;
  - maintain design pressure in the plenum during stoppages of shorter or longer duration, and
  - compensate possible grain-size deficiencies of the natural ground.

- Polymer, added to bentonite and/or foam to help stabilise the mix.
- Tail grease, pumped in the gap between tail skin and lining surface and between adjacent brushes, to improve their resistance to grout infiltration.
- Cement grout, used to fill the gap between the extrados of the precast lining and the excavation profile.

3 OPERATING PROCEDURES

3.1 Background

Tunnel excavation using EPBS is based on the principle of face support using the same excavated material, going in the excavation chamber, or plenum, and putting it under pressure, through the balance between the material entering and the material exiting and with the added machine thrust.

The muck in the chamber will be brought to a level of pressure appropriate to the surrounding conditions, geotechnical characteristics of the ground, water-table position in relation to the tunnel-axis level, hydraulic gradients of the filtering water, ground permeability, presence of any potentially interfering structures, etc.

Under the thrust applied to the shield and, thus, to the rotating cutterhead, the ground is shorn from the face and flows in the excavation chamber, from which it is then extracted through screw conveyor in the desired quantities so as to compress it until the required pressure is obtained; this pressure will support the face.

Concurrent with excavation, the shield slips away from the previously erected ring mounted inside and the annular void between the extrados of the lining ring and the excavated profile is backfilled with longitudinal grout injections at an appropriate pressure, using a volume that is at least equal to the theoretical volume of the annular void. The grouting injectors are embedded in the tail skin, which is protected (toward the interior) from the risk of grout ingress via several brush rings, into which (both within brushes and rows of brushes) appropriate grease is continuously pumped.

During the excavation, one has to extract the volume of material entering the chamber, i.e. the theoretical volume plus any additive injected at the face or in the chamber. It is of paramount importance to control the volume extracted by the screw conveyor, to be able to intervene both in the case when more material is extracted than the theoretical (over-excavation) and in case when less material is extracted (under-excavation). For more detailed explication, reference can be made to Sections 4 and 6.

3.2 Definition of normal and anomalous conditions

Normal excavating conditions are considered all those conditions, whose EPBS excavation characteristic parameters fall within the "attention" thresholds (as defined in section 6).

Normal conditions include the conditions that are intrinsic in the excavation restart after the maintenance interventions in the excavation chamber.

Anomalous conditions are associated with:

- Water inflows under pressure through the screw conveyor.
- Sudden oscillations of the torque of the cutterhead.
- Blockage of the cutterhead.
- Anomalous pressure values in the excavation chamber.
- Sudden and significant variations of the muck density in the excavation chamber.
- Weight of the muck extracted by the screw conveyor surpassing the "attention" threshold.
- Insufficient pressure and/or volume of the grout injected behind the lining.

3.3 Excavation parameters control

The EPB Shield Superintendent, or Machine Superintendent (MS), by examining the control parameters of excavation and their attention and alarm thresholds, verifies whether the tunnel advance condition is normal (see section 3.4) or anomalous (see section 6).

The parameters, to be verified via the sensors and sensing equipment, are:

- Face-support pressure (pressure value in the plenum given by the chamber sensors).
- Pressure and volume of the backfill grout of the annular void between the extrados of the lining and the excavated profile.
- Weight of the extracted material, with relevant values for attention and alarm thresholds.

The Machine Superintendent, MS, has to verify also that the calibration of the scale (or scales), weighing the muck extracted by the screw conveyor, has been performed at the required time intervals and that the readings are reliable.

The start of the excavation depends on the check of any mechanical or electrical anomaly and the above mentioned parameters.

If one of the excavation control parameters is higher than the attention threshold value (defined hereafter) MO has to inform MS immediately, who shall then give instructions to the staff on duty.

If the control parameters reach alarm thresholds, the excavation must stop, until the necessary countermeasures are implemented.

Similarly, if during the longitudinal backfill, the mandatory quantities and/or pressures are not reached, the Lining Operator shall inform the Machine Superintendent immediately, who shall then take the relevant decisions.

3.4 Tunnel progress in normal conditions

The three principal operations composing the production cycle are: excavation, grout backfill behind the lining (concurrent with excavation), and precast lining-ring erection.

3.4.1 Excavation

The MO carries out the following preliminary operations in sequence:

- Start-up of the electrical motors and of the hydraulic groups necessary for the excavation operations.
- Start-up of the foam and/or polymers and/or bentonite injection plant, injecting them directly on the face and/or the excavation chamber and/or the screw conveyor.
- Start-up of the cutterhead rotation until it reaches the foreseen speed.
- Pressurization of the thrust cylinders.

With the start-up of the screw conveyor, the controlled extraction of the muck from the plenum is started. The debit control, which is performed through regulating the rotating speed (variable between zero and a maximum – generally of 12 to 18 rpm), is aimed to maintain the designed face-support pressure, in the plenum.

The Machine Operator shall regulate the screw rotation speed (proportional to the extracted volume) according to the penetration rate of the EPBS (proportional to the excavated volume), to maintain the design pressure, and thus match the muck flow exiting from through screw with the muck flow entering the excavation chamber (natural ground + additives).

The task of the Operator is to maintain such an "equilibrium" condition, as much as possible, by intervening as described later.

The EPBS direction and position are set by regulating the pressure of the thrust jacks, in the various sectors in which they are subdivided. The EPBS steering system continuously visualizes, graphically and numerically, the position of the axis of the EPBS in relation to the tunnel route axis, thus providing a constant reference to the Operator.

The guidance system also supplies the 3-dimensional coordinates of a point on the tail axis and another near the cutterhead, the vertical and horizontal inclinations of the EPBS axis with respect to its theoretical position, and the rotation of the shield with respect to its own axis. Specifically, MO can visualize, instant by instant, the offset (vertical and horizontal) of the centre of the cutterhead from the theoretical centre of the tunnel on a vertical section. The Operator can also observe the offset of the centre of the shield tail from the theoretical centre in that section. The guidance system visualizes, graphically and numerically, the vertical and horizontal "trends" of the EPBS in relation to the theoretical axis and it calculates the required correction curves, visualizing the EPBS position in respect of such curves.

The control of the actual tunnel axis shall be made through periodical topographic surveys, to verify that it is within the permitted tolerances.

When the thrust cylinders have been extended to the limit of the cycle length (equal to the longitudinal dimension of the lining ring), MO shall stop the inflow of conditioning materials into the ground (except when it is necessary to inject bentonite to guarantee the maintenance of the design pressure), reduces the cutterhead rotation speed until its stoppage, reduces the cylinders thrust and the screw conveyor rotation until its stoppage, and closes the rear gate of the screw conveyor, thus bringing the excavation phase to a halt.

3.4.2 Grout backfill behind the lining

This process and procedures are common to both types of TBM – HS and EPBS.

During the entire excavation phase and concurrently with the TBM advance, the grout is injected behind the lining. Through the double feed-pumps on the back-up, the grout is injected via the grout lines (usually 6) constituted by pipelines ending past the series of metallic brushes (usually 3), which are fitted on the inner circumference of the tail shield and through which a special grease is continuously injected. The grout injection occurs at the shield tail, directly on the extrados of the precast lining ring. The grout lines installed on the tail shield are duplicated (with a spare for each) so that it is possible to deviate the grout flow on the spare pipe, should the one in use become blocked, to permit continuous operation and a clean-up of the blocked line.

The need to control the volumes of injected grout is essential for controlling the surface subsidence. The theoretical volume of the ground to be injected, under optimal conditions with new cutters, is equal to the difference between the excavated section and the extrados section of the lining ring, multiplied by the length of the ring.

The actual injected quantity of the grout is a function of variables such as the slight (a few cm) difference between theoretical and actually excavated length, the route trend (whether straight or curved, with or without over-excavation), the excavated material behaviour and its permeability, the gauge cutters' wear, and the grout characteristics (especially fluidity). The most important variables are: the cutters wear, which causes the quantity to diminish, and the ground permeability, which may increase the quantity depending on the fluidity of the grout.

The grout transport mixer will be loaded with an adequate number of batches, sufficient to guarantee the maximum quantity necessary, ensuring that the mixing blades are regularly rotating and that there are no solid remnants of the previous mix at the bottom, which can compromise the proper grout pumping.

The lining injection shall (usually) be carried out through all the lines using the following positions on the face of a 24-hour clock: usually 1 at 12 hours, 2 at 10 and 14 hrs, 2 at 16 and 20 hrs and the last at 24 hrs. In case one of the pipelines is blocked, one must switch immediately to the spare. Particular attention is to be given in case of "floating" phenomena of the lining ring (uplift until squashing the upper brushes), in which case pumping in the bottom line should be avoided.

Before the excavation is started, pumping of the grout should begin so as not to have voids at the shield tail.

The activities of the Machine and Lining Operators shall be strictly coordinated and they shall have constant telecommunication during the various work phases.

Should grout quantity be insufficient due to bigger-than-theoretical consumption, the face excavation shall be stopped and the additional grout quantity shall be procured before initiating any further advance.

Once the excavation is completed, grout pumping shall continue until the minimum reference pressure for each position is reached. The pumping system must be fitted with a maximum pressure valve, which cuts-off the pump when the maximum pressure safety-threshold is reached. The LO is responsible for this entire operation.

The minimum and maximum pressure values for each tunnel stretch shall be supplied together with the face-support pressure values in a purposely-prepared design document. For the backfill grout, different pressure values shall be provided for the respective injection points and also as a function of the litho-static and hydraulic loads, if any.

3.4.3 Precast lining ring erection

The EPBS guidance system is equipped with software which allows to calculate the optimal positioning of the ring to be erected to assure the match between the actual and theoretical axis of the tunnel falls within the design tolerances according to the "universal" ring logic. The following sequential steps are used for erecting the precast lining-ring:

- MO inserts in the EPBS computer the data allowing to evaluate the "position" in which the lining ring has to be erected.
- The computer provides output data on the "key" segment position, which is used to define the mounting sequence.
- By the time the excavation cycle is finished, the precast lining segments, which had meanwhile been positioned in the correct erection sequence in the supply bay, are ready to be handled by the erector for ring mounting into its final position.
- MO advises the Erector Operator, EO, about the position in which the ring has to be erected. The EO then starts the ring mounting from the segment opposite the "key" segment.
- While the erector catches the segment, the thrust cylinders, which correspond to the position where the segment will be positioned, are concurrently retracted.
- The segment is positioned adjacent to that erected previously and to which it is to be fixed. When the bolt recesses are aligned, the bolting is carried out.
- The previously retracted cylinders are re-extended until contact with the erected segment.
- The same procedure is followed for all the other segments, bolting them with each other and with those of the preceding ring.
- The "key" segment is mounted last, bolting it with a single bolt to the previous ring.
- At the end, (if foreseen) a further precast element, the base segment, is positioned to act as support for the rails.

In case of a mucking system by train, from the beginning of the excavation cycle the train is positioned in the back-up area with:

- The necessary quantity of wagons (empty) for muck transport, positioned in the area below the back-up conveyor. The mobility and/or the double-direction rotation of the belt allows a complete filling of the wagons.

- One grout wagon (fitted with agitator) which feeds the holding tank for the grout mix.
- Three flat-wagons for segment transport, positioned in the segment-loading bay. During the excavation, the segments are unloaded from the flat-wagons and located on a "feeder" close to the erector.
- One supply flat-wagon.

Once the excavation cycle is completed, the train can return to the portal/shaft for muck offloading and new supplies loading.

4 MACHINE STOPPAGES

The EPBS can have a stoppage for various situations, due both to programmed (maintenance, control) events and unforeseen (breakages or geological) problems. In any case stoppages can be divided in two categories:

1. Stoppage not foreseeing the plenum being emptied.
2. Stoppage of any nature which includes emptying, partially or totally, the plenum.

The procedures hereafter detailed for any type of stoppage are adopted, as a function of whether or not the tunnel is below the water-table.

4.1 Stoppage without emptying the excavation chamber

4.1.1 Below the water-table

The Machine Operator, MO, completes the shift with the last advance in the pressure conditions foreseen in that stretch by the design document.

It is not necessary to close the hydraulically-controlled flood doors on the cutterhead, even if they are present (as in some EPBS).

MO shall remain on the EPBS even during the stoppage and shall verify that the face pressure remains within the predetermined range. Should the pressure drop below the attention threshold due to the normal "relaxation", the Operator shall pump bentonite slurry until the pressure level is brought back to the reference level, if the automatic equipment is not present. This operation shall be repeated every time is necessary, taking care that the pressure never drops below the attention threshold, $P_{att.}$

The TBM re-start shall begin by making the cutterhead turn and thrusting the shield against the face, without rotating the screw conveyor, thus increasing the pressure in the plenum until the attention threshold value; only then the screw conveyor will be activated, and the excavation may re-start.

4.1.2 Above the water-table

All the operations shall be carried out as already indicated, except that during the last shift, before the stoppage, some bentonite slurry could be injected in the plenum, or the quantity of foam and polymers could be increased, for maintaining the required pressure during the stoppage.

Similarly, the MO shall inject bentonite slurry into the plenum (if not automated) to compensate the muck-pressure drop within the chamber, every time the "low-attention" threshold is reached.

Again, it is not necessary to close the hydraulically controlled flood doors on the cutterhead, even if they are present (as for some EPBS).

The TBM re-start will be realized by following the procedure mention in the previous subsection.

4.2 Stoppage with emptying of excavation chamber

4.2.1 Under the water-table

Should some activity need to be carried out in the excavation chamber, the MO will begin to mix bentonite slurry with the ground to obtain a "dough" as homogeneous and watertight as possible. When the first traces of bentonite are noticed at the screw outlet, the flood doors of the cutterhead (if present) have to be closed and the chamber is progressively emptied to the level necessary for the required intervention. In order to guarantee the stability of the face, the void being created by empting the plenum is progressively filled with compressed air at the pressure indicated in the detailed design. For cutterhead intervention, in this case the hyper-baric chamber shall be used.

If present, the flood doors will remain closed to provide additional safety for the workers intervening in the plenum.

The re-start implies filling the excavation chamber with a bentonite mix and/or a mix of sand, water, and bentonite, progressively letting the compressed air out of the chamber via the release valves while controlling that the pressure does not drop below the alarm threshold. Afterwards, the flood doors (if any) are opened and the excavation restarts.

As an alternative, when ground is stable, the flood doors (if any) can be opened, beginning excavation without extracting muck (i.e. with screw stopped) so that, opening the safety-valve air outlets, the ground progressively substitutes the air for filling the plenum.

The choice between the two methods depends from the ground conditions and shall be made by an accord between the Contractor, the Design Consultant, and the Engineer.

The screw gate will then be opened and mucking will begin when the upper sensors' pressure is higher than the reference pressure.

4.2.2 Above water-table

Should doubts exist about ground stability even when excavating above the water table, the procedure shall be the same as the previous one.

If the ground stability is proven, the operations could be as follows:

MO completes the shift with the last thrust in the excavation pressure conditions foreseen for the stretch by the detailed design document.

After closing the flood doors (if any), the screw conveyor will be operated with the machine stopped, emptying from the chamber the quantity necessary to reach the optimal level that will permit the intervention on the cutterhead.

The re-start shall be by refilling completely the chamber void with a mix of sand, water, and bentonite until the minimum threshold pressure is reached.

Then, the doors (if any) shall be opened and excavation will start, mixing the entering material with sand and bentonite slurry, and keeping the screw conveyor closed in order to maintain the pressure. During progress of the excavation, when the pressure has reached the average reference levels, the screw gates are opened and the mucking begins by extracting the "mixed" muck, thus leaving space for the natural muck.

4.3 Long stoppages

If for any reason the stoppage should continue for a few days, in the last phase of the excavation (if the stoppage was programmed) or by executing an excavation 'ad hoc', the MO shall mix bentonite slurry with the muck in the plenum (to obtain a "dough" rich in bentonite) to stabilise and waterproof the face. When the first traces of bentonite mixed with muck are noticed in the screw outlet, the screw gate is closed and excavation is continued until the end of the cycle. After the last cycle, and after waiting for some hours, depending on the setting time of the backfill grout behind the lining, the EPBS is advanced by a few centimetres (e.g. 5 cm) and bentonite slurry (instead of grout) is injected in the tail grouting pipes to fill the relatively small void created by the small advance. This is done to prevent the grout from setting on the brushes, which would damage them.

Then the flood doors (if any) shall be closed, and they must remain closed during the entire stoppage period.

During the prolonged stoppage, controls must be made to assure that the chamber pressure does not drop below the attention threshold, intervening with further bentonite slurry injections, if necessary.

The re-start occurs as for the previous cases.

5 CONTROLS PLAN

The controls carried out during excavation comprise the following operations and parameters:

1. Face support pressure;
2. Muck density in the excavation chamber;
3. Injected grout pressure and volume;
4. Excavated material weight and volume, and
5. Ground conditioning.

The control of abnormal or potentially dangerous situations occurring during excavation is discussed in section 6.

5.1 Face support pressure

The pressure to be applied in the excavation chamber to support the tunnel face (P_{face}) is calculated on the base of:

- Tender Design, TD, data and contractual prescriptions (with particular reference to Special Conditions, SC).
- Geotechnical 'homogeneous' stretches and relevant characteristics along the route (with particular reference to the ground affecting the tunnel and the layers just above the tunnel crown).
- Particular conditions and potential interferences along the tunnel route.
- Excavation experience accumulated with the EPBS (operating conditions, EPBS parameters, rock mass deformation response such as volume loss and subsidence).
- Specific prescriptions of the detailed design document "Pressures to be applied to the tunnel face by the EPBS in the various homogeneous stretches".

The above components shall form part of a specific document (PAT) finalized to provide the values for P_{face}, together with the threshold values (attention: $P_{face\text{-}attention}$ and alarm: $P_{face\text{-}alarm}$), for distinct stretches of tunnel.

The PAT shall supply reference, attention, and alarm values of the pressure in the plenum for the next stretch of about 300 m and shall be supplied when the EPBS is within 100 m of the stretch. PAT shall be updated for the following stretches on the basis of the actual-face advance experience obtained with the EPBS during the excavation of the preceding stretches.

The threshold values of P_{face} are assessed so that the corresponding pressures assure an adequate safety coefficient in any event.

Note that the values of P_{face} to be kept under control are those of the sensors positioned closest to the crown of the tunnel (sensors 1 and 2 in Fig. ap.6.1).

When P_{face} is within the attention thresholds, the Machine Operator, MO can autonomously manage the EPBS advance (with the means at his disposal) to guarantee

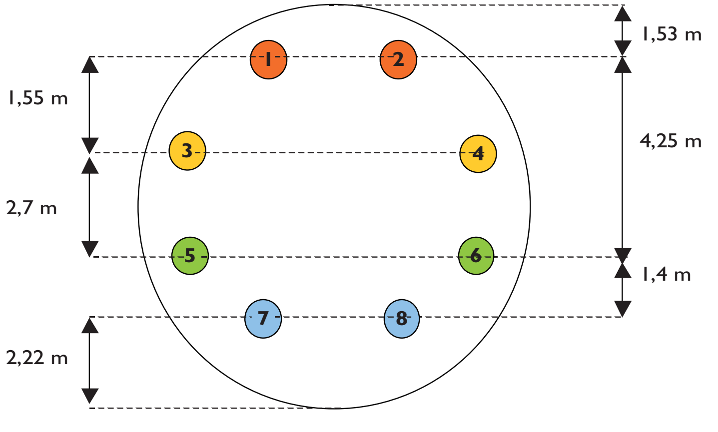


Figure ap.6.1 Position of the pressure control sensors.

the correct excavation procedures (maintenance of the design pressure, through the regulation of the screw conveyor rotating speed, and the EPBS rate of advance). Should it not be possible to maintain P_{face} within the attention thresholds, the MO shall immediately inform the Machine Superintendent, MS. Consequently reference shall be made to the "Conditions of anomalous excavation", which are discussed in section 6. The control of P_{face} by the MO will be a continuous operation during the entire period of tunnel excavation.

5.2 Muck density in the excavation chamber

The presence in the excavation chamber of a "soil-foam-air mix", i.e. of a material with varying density, entails a P_{face} which, though possibly the same as the design one, does not guarantee the face support. In particular, "voids" in the upper part of the excavation chamber could occur due to prolonged stoppages, and consequent settlement of the muck due to its own weight and the migration of the "gaseous" components of the foams toward the upper part.

The consequence of the above is the necessity to verify that the apparent density (γ) of the material contained in the excavation chamber at the various levels is adequate and possibly constant. To achieve this, the pressure difference between the values of a pair of sensors ($P_{sup.\ sensor} - P_{inf.\ sensor}$) is divided by the vertical distance between the pair of sensors ($\Delta h_{sup.\text{-}inf.}$):

$$\gamma = (P_{sup.\ sens.} - P_{inf.\ sens.})/\Delta h_{sup.\text{-}inf.}.[\mathrm{kN/m^3}] \qquad \text{(ap.6.1)}$$

For example, to verify that the material density of the muck present in the upper part of the chamber is at least

$$\gamma = 15\ \mathrm{kN/m^3} \qquad \text{(ap.6.2)}$$

assuming that

$$P_{sup.\ sens.} = 1.25\ \mathrm{bar}, \qquad \text{(ap.6.3)}$$

being $\Delta h_{sup.\text{-}inf} = 1.55$ m (see Fig. ap.6.1), the pressure measured at the sensors below has to be:

$$P_{inf.\ sens.} = 1.25 + (1.55 \cdot 15)/100 = 1.48\ \mathrm{bar} \qquad \text{(ap.6.4)}$$

The MS is the person in charge of the density control of the chamber muck at the end of each excavation process.

5.3 Injection pressure and volume

At the end of the excavation cycle, if the mandatory values for grout backfill quantity and/or pressure have not been reached, the Lining-backfill Operator, LO, will continue to pump, even during the ring erection, until the maximum pressures are reached. Should that not occur before "re-start" of the excavation, the LO shall advise the MS, who shall engage the necessary procedures, if necessary, for delaying the beginning of the next excavation cycle and advise the Tunnel Manager, TM.

The grout injection procedure is correct, if the grout pressure, for each cycle, reaches the foreseen maximum and then drops to minimum values.

The pumping system must allow to regulate the number of pumping cycles per minute so to be able to keep up with the penetration rate of the EPBS.

The grout injection pressure is measured by appropriate sensors on the grout line close to the shield connection. The injection pressure and the injected volume are transmitted via PLC and are indicated on the control panel monitor to LO. Thus a continuous control of both parameters is possible. The LO can regulate the pumping cycles per minute, while the maximum and minimum grouting pressures are defined as "excavation parameters" according to the TM indications.

If the grout volumes are different from the foreseen values, the MS must be informed immediately and he shall then also advise the TM, should the difference be high enough to cause an excess over the minimum and maximum thresholds. For example if the theoretical grout consumption volume is 6.6 m^3, TM must be advised immediately when injection quantities are less than 5.8 m^3 or more than 7.6 m^3. A grout volume decidedly greater than the theoretical, for filling the annular gap between lining extrados and excavation profile, can indicate either over-excavation or grout dispersal towards natural cavities.

Should either of the limits, higher or lower, be exceeded, then one or two core-sampling perforations must be made to verify the correct backfill of the lining, taking necessary precautions if the perforations are done under the water table. Consequently, if the investigations prove it necessary, secondary grouting, agreed with the Engineer, will have to be made via the holes existing in the precast segments.

5.4 Weight and volume control of the excavated muck

The control of the material extracted from the chamber by the screw conveyor is a matter of paramount importance for it could indicate the possible over- or under-excavation.

A scale, located on the conveyor belt just downstream of the screw conveyor outlet, allows to measure the debit of the material extracted by the screw (ton/hr). The cumulative weight for each stroke is calculated through on-board PLC. This weight has to be compared with the theoretical value deriving from multiplying the excavated cross section by the length excavated and the relative density of the natural ground (γ_{insitu}). Should the main scale malfunction, on the same belt a "spare" scale needs to be installed, whose readings are to be taken into account only when abnormal weighing occurs on the main one. The weight of additives introduced in the plenum for conditioning should also be taken in account.

The value for γ_{insitu} to be taken as reference for the calculations shall be proposed by the Contractor during the periodical meetings with the Engineer and shall be ratified jointly, on the basis also of grain sizing results and density measurements of the muck collected from the conveyor.

The MO, through PLC, has at his disposal the value of instantaneous cumulated weight and the total value at the end of each cycle, and is thus able to keep under control the quantity of extracted material in real time. Should the cumulated weight exceed the attention threshold, MO shall act to bring the situation within "normal" values by operating on the screw speed or on the penetration rate of the EPBS or on both.

If the "attention" conditions persist or the alarm threshold is reached, the MO shall stop excavation immediately, and inform TM and PM. In this case reference shall be made to the instructions included in section 6. The control of the extracted weight is under the responsibility of MO.

5.5 Control of the ground conditioning

The ground conditioning during excavation is performed using foam, polymers and bentonite.

Foam is obtained by mixing water, tension-active agent and air in variable proportions according to the ground type and the foam type utilised. The degree of ground treatment is defined by the F.I.R. (Foam injection ratio): foam volume injected per cubic meter of excavated terrain (expressed as a percentage). However, the foam properties derive from the F.E.R. (Foam Expansion Ratio): ratio between air volume and liquid phase volume. Another parameter, "dosage", refers to the percentage of tension-active agent in the water. These parameters can be varied any time by the Machine Superintendent, MS, when he considers it necessary to modify the extent of ground conditioning. The control is based on the visual observation of the muck exiting the screw conveyor.

Polymer can be added to the foam to chemically stabilise it. Its use can guarantee, where necessary, a longer persistence of the foam properties, especially reducing the effects due to absorption by the ground of the foam water (in case of low water content of the ground).

Bentonite can partially substitute the foam in treating the ground in special instances.

The responsibility of the choice of conditioning type belongs to the Tunnel Manager, TM, while the MS shall oversee that each shift conforms to the prescribed instructions.

6 EXCAVATION CONTROL IN ABNORMAL CONDITIONS

Should any of the abnormal or anomalous conditions occur, the MO must immediately advise the MS, who is responsible for the correct implementation of what is defined in this section. The MS must inform the Project Manager, PM, about the procedural action.

The following conditions are considered to be abnormal or anomalous

1. Water inflows under pressure through the screw conveyor.
2. Sudden oscillations of the cutterhead torque.
3. Cutterhead blockage.
4. Abnormal pressure values in the excavation chamber.
5. Sudden and significant variations of the muck density in the chamber.
6. Over- and under-excavation of the muck, at the alarm level.

7. Failure to reach the reference pressure and/or the grout volume injected behind the lining extrados.

The excavation control in any of the above conditions shall follow the operating procedures described in the following paragraphs and, in any event, it must be agreed with the Design Consultant, responsible for the tunnel advance control.

6.1 Water inflow through the screw conveyor

The tunnel excavation could involve stretches that are below the water table and/or below loose sand lenses, which could constitute a "suspended water table", if confined between low permeability layers. In such instances, the hydraulic load of the excavation chamber may suddenly jump-up, with consequent high hydraulic-gradient causing filtration into the head and in the plenum, and pressurising the conveyor screw. The result is the water spillage under pressure from the screw's rear gate. The situation becomes more dangerous when fines are also brought into the plenum.

The video camera located on the first conveyor under the screw's rear gate allows the MO to notice such an event immediately. Then, the MO must immediately advise the MS who has the responsibility for the correct application of this procedure and who, in turn, advises the Tunnel Manager, TM. The sequence of the operations is as follows:

- If water is being injected to the face, stop it immediately.
- Close the watertight rear gate of the screw.
- Increase the ground treatment (increase the foam percentage being added, lower the foam expansion, increase the tension-active additive dosage by adding more polymer).
- Continue the excavation with the screw stopped, thus trying to increase the density of the muck in the plenum, while controlling that the pressure does not increase excessively.
- Try and re-start the screw, after opening the rear gate;
- Should the water inflow persist, inject directly at the screw bottom, either polymer or bentonite, until the muck reaches a plastic consistency.

All these activities are registered in the excavation report.

6.2 Sudden oscillations of the cutterhead torque

Under normal conditions, the excavation parameters in maintain constant values and do not undergo sudden variations. A sudden variation or an unjustified oscillation of such parameters could be an indicator of a possible instability at the face, or a sudden variation of the geological and mechanical characteristics of the ground. The cutterhead torque is the principal parameter to signal such type of occurrences.

The MO alerts the MS for controlling these events and the MS, which has the responsibility for the correct application of this procedure, proceeds as follows:

- Keep the face pressure constant.
- Reduce the rotation speed of the head to <1 rpm.
- Reduce the EPBS penetration rate to <20 mm/min.
- Reduce the screw-rotating speed as a consequence of the reduced penetration rate and, thus maintain a constant face pressure.

If the problem persists, the MO stops the excavation and the MS informs the TM.

6.3 Cutterhead blockage

The cutterhead blockage could be due to various causes such as face instability with collapses, cutter tool or other blocking the head, and material poorly conditioned either at the face or in the plenum.

This is a dangerous phenomenon of which the TM and the highest rank manager on site must be immediately advised. If the blockage is not a total blockage and rotation is possible to some extent, it could be that a big rock mass or a broken metallic piece in the muck-collecting structure (within the head) has caused the blockage. In such an event, it could be possible to hear through the bulkhead the sound of it hitting the metallic piece. If this is the case, operations have to stop and an intervention into the excavation chamber must be organised, with prior authorisation from the PM. Should the head be totally blocked and there is no indication that the cause is either a big boulder or some other extraneous material, the procedure to be adopted is as follow:

- Stop the screw and close its rear gate. Do not empty the excavation chamber and, if necessary and as indicated by a chamber pressure reduction, inject bentonite to the face.
- Arrange communication with a worker to be stationed near the bulkhead, to try and notice possible noises coming from it, in that the blockage could be caused by pieces of rock not yet loaded by the loading scrapers or by metallic carpentry elements (scrapers or other parts broken).
- If the head cannot be unblocked, is to be retracted for 10–15 mm and an attempt is made to rotate it alternatively clock- and anti-clock-wise.
- If the un-blockage is not achieved, bentonite is injected directly to the face through the foam injection sprays.
- The head is retracted again for another 10–15 mm and rotation is tried again.
- If the un-blockage again is not achieved, the “maximum over-torque” is applied;
- If even the latter action does not succeed, all operations must be stopped and the PM – previously advised shall have the responsibility to activate a “technical coordination” attended by the technical staff of the Contractor and of the Design Consultant as well as the Engineer Representative.

The procedure and actions undertaken, the quantity of injected material, and the times needed for the various activities during the blockage must all be recorded on the “Excavation activities report”.

6.4 Abnormal pressure values in the excavation chamber

Sudden variations of the face-support pressure could be the warning signals resulting from torque increases or head blockages. The following operations will respond to the problem:

In case the pressure increases:

The head rotating speed is reduced to <1 rpm.
The thrust is reduced so that penetration rate, Vp, is <10 mm/min.
The foam flow is increased by 20%,without increasing the muck discharge from the screw.
The MS is notified.

In case the pressure diminishes:

Bentonite is injected to re-establish the design support pressure.
If pressure still does not increase, excavation is stopped and the screw gate is closed.
Bentonite and polymer injection is continued until the designed support-pressure is achieved.

When the procedure followed and the actions taken, the quantities of injected material and the times taken by the various operations must be recorded on the "Excavation activities report".

6.5 Sudden variations of the muck density in the plenum

The density of the muck in the plenum (apparent density) has to be kept close to the foreseen value (14 kN/m^3). The apparent density can be calculated using the pressure differences between sensors pairs positioned at different elevations (see section 5.2).

The MS shall take care, during ring erection, to evaluate the pressure differences at the various levels. If the pressure differences between sensor pairs $P_{sens.3,4}$ and $P_{sens.1,2}$ is less than 0,22 bar, then MS shall have bentonite injected in the chamber, concurrently opening the release valve to verify if any air and/or foam are present in the crown and, if necessary, eliminate them.

6.6 Over- and under-excavation of the muck

If it is noticed that over-excavation is occurring, and the attention threshold has been exceeded the following operations must be carried out:

- The MS must be advised;
- Screw rotation is reduced;
- Head rotation is reduced to <1 rpm;
- Thrust is reduced and so is the penetration rate. If the excavation cycle is completed without problems, the details of the incidence are recorded in the excavation report.

If the over-excavation alarm-threshold is reached, excavation must cease immediately and PM must be advised. Through a "technical coordination" meeting the PM will select the type of intervention be used.

If the under-excavation alarm-threshold is reached, the MS must be advised and he, in turn, advises TM.

The excavation is stopped. It is noted that one of the potential reasons for under-excavation is the incorrect estimation of the in-situ density. This could be dangerous, because it indicates that the in-situ characteristics of the ground could have changed in worst conditions.

6.7 Inadequate pressure and volume of the lining backfill grout

The grouting procedure is correct if the level of grouting pressure, for each pumping cycle, reaches the foreseen maximum and then drops to the minimum in a regular manner.

The pumping system allows to regulate the number of pumping cycles per minute in order to match the penetration rate. If the latter is too high, the grouting must be continued, even after the excavation cycle is finished, until the design pressure is achieved.

The LO and the MO must keep in continuous contact for managing the following three types of events.

1. Injection pressures and grout quantities below normal values: the number of pumping cycles per minute has to be increased until the required values are reached.
2. Injection pressure is low but the grout quantities are correct: the number of pumping cycles per minute is increased until the required values are reached. If the pressure does not increase, the thrust is reduced and so is the *penetration* rate. If the pressure still does not increase, the excavation is stopped and grouting is continued until the required pressure is achieved.
3. Injection pressure is achieved, but the grout quantities are below normal values. Control the grouting lines for a possible blockage. If only one line is blocked, the grout flow is deviated in the spare line (which all the six lines have). Concurrently, the blocked line gets cleaned and the blockage removed. These activities occur without stopping the excavation. The data relating to grout volumes and injection pressures are registered.

The above events will be recorded in regard to the quality and quantities in the "Excavation report".

The grouting pressure is measured via sensors located on the injection lines close to inner surface of the tail shield. The injection pressures and volumes are shown on the panel, which is available to the LO, and are transmitted through the PLC to the monitor. Thus, the two parameters are constantly controllable. The LO is thus able to regulate the number of pumping cycles per minute, while the minimum and maximum values of the grouting pressure are defined as excavation parameters according to the prescriptions by TM.

Appendix 7

The Italian experiences

7.1 Metro of Rome - Line A 1
7.2 Metro of Rome - Line B
7.3 Railway Ring of Rome Aurelia Tunnel
7.4 West Naples Sewer
7.5 Naples Rapid Tramway Line
7.6 Milan Rail Connection
7.7 Metro of Rome - Line A 2
7.8 Metro of Genova Line 1
7.9 Metro of Milan Line 1
7.10 Rome - Viterbo Railway Quattro Venti tunnel
7.11 Metro of Naples Line 1
7.12 North Milan Railways Castellanza tunnel
7.13 Metro of Brescia Prealpino S. Eufemia Lot

Metro of Turin Line 1 (see Section 8.4)
Bologna - Florence HS Railway Bologna tunnel (see Section 8.6)

Appendix 7.1

Metro of Rome – Line A

Tunnel construction period 1970–1980

Colli Albani – Termini and Termini – Flaminio stretches

Table ap.7.1.1 Project information

Location	Rome
Name	Metro di Roma Linea A. Tratta Colli Albani – Termini e Termini – Flaminio
Owner	Comune di Roma (Ente Concedente) Intermetro (Concessionaria)
Designer	Intermetro – Sefer
Contractor	Fiat Impresit (Colli Albani – Termini stretch); Metroroma (Termini – Flaminio stretch)

Figure ap.7.1.1 The TBM excavating the Line A tunnel.

1 GENERAL DESCRIPTION

The Metro of Rome Line A starts from "Anagnina" Station, located in the south of the city, passes through "Colli Albani" and "Termini", and then run towards North-West until "Flaminio" Station, where it turns towards West, until the terminal station of "Ottaviano". In "Termini" Station, Line A underpasses the Metro Line B. The 22 metro stations along Line A have an average distance of about 670 m.

The metro tunnels were principally located under the main streets, with rail level about 8 meters below the ground surface, and in these conditions the construction methodology was cut & cover. Deeper stretches were excavated using a mechanized tunneling methodology: single-track, twin-tube tunnels with 5,50 m diameter and 24 m^2 section.

2 TUNNEL CHARACTERISTICS

Table ap.7.1.2 Project characteristics

	Colli Albani – Termini	
	value	*unit*
Length	2 x 4370	[m]
Excavation diameter	5,44	[m]
Lining type	precasted segments	
Lining thickness	0,30	[m]
N°segments/ring	5 + 1	
Ring length	1,00	[m]
Ring connections	bolts	

(contd.)

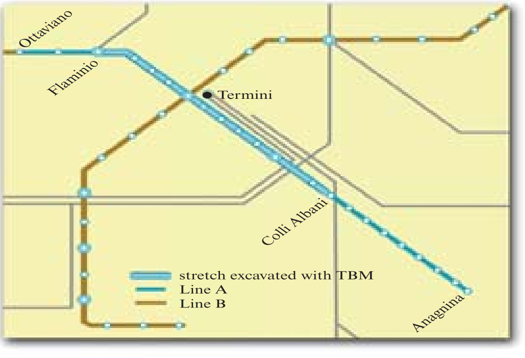


Figure ap.7.1.2 The layout of the Line A.

Table ap.7.1.2 (*Continued*)

	Termini – Flaminio	
	value	*unit*
Length	2 x 2500	[m]
Excavation diameter	5,50	[m]
Lining type	precasted segments	
Lining thickness	n.a.	[m]
N°segments/ring	4 + 1	
Ring length	n.a.	[m]
Ring connections	bolts	

Figure ap.7.1.3 A picture of the tunnel.

3 GEOLOGICAL-ENVIRONMENTAL CONTEXT

From the southern end of the line until "Colli Albani" Station, the ground interested by the tunnels excavation was mainly vulcanic deposit (Pozzolana, except for some short stretches interested by Tuff) and alluvial deposit filling fractures.

From "Colli Albani" Station, until "Ponte Lungo" Station, tunnels were excavated in Tuff and blue clay. The presence of some fractures filled by heterogeneous materials, caused construction problems and required ground impermeabilization and reinforcement by grouting. After "Barberini" Station until "Flaminia" Station the interested ground was blue clay.

The tunnels were excavated below the water table. Due to the very low permeability of the blue clay, only the stretches excavated in volcanic deposits were interested by the presence of water.

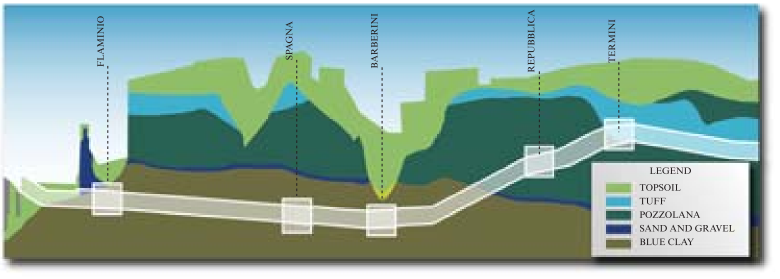


Figure ap.7.1.4 The geological profile of the Line A.

4 JOB SITE PERFORMANCES

Table ap.7.1.3 Job site performances

Average advance speed	12.5	[m/d]
Best month advance	487	[m]
Best daily advance	40	[m]

Figure ap.7.1.5 The line A today.

Appendix 7.2

Metro of Rome – Line B

Termini – Rebibbia extension, Termini – Lecce stretch

Tunnel construction period 1981

Table ap.7.2.1 Project information

Location	Rome
Name	Metro di Roma Linea B Prolungamento Termini – Rebibbia
Owner	Comune di Roma (Ente Concedente) Intermetro (Concessionaria)
Designer	Intermetro
Contractor	Girola

Figure ap.7.2.1 The TBM for excavating the Line B tunnel.

1 GENERAL DESCRIPTION

The Line B was the first metro line of Rome, in operation since 1955 from "Laurentina" to "Termini" stations, at the city centre.

The extension from "Termini" Station to "Rebibbia" Station is 7.9 km long with 10 of the 22 stations included in the Line B (average distance between stations: 750 m).

Different section types and construction methodologies were used for Termini – Rebibbia extension. In particular, from "Termini" Station to "Piazza Lecce" Station a mechanized method was used in order to realize the single-track, twin-tube running tunnel.

2 TUNNEL CHARACTERISTICS

Table ap.7.2.2 Project characteristics

Length	2 x 1800	[m]
Lining type	precasted segments	

Figure ap.7.2.2 The layout of the Line B.

3 ENVIRONMENTAL CONTEXT

Figure ap.7.2.3 A picture of the tunnel.

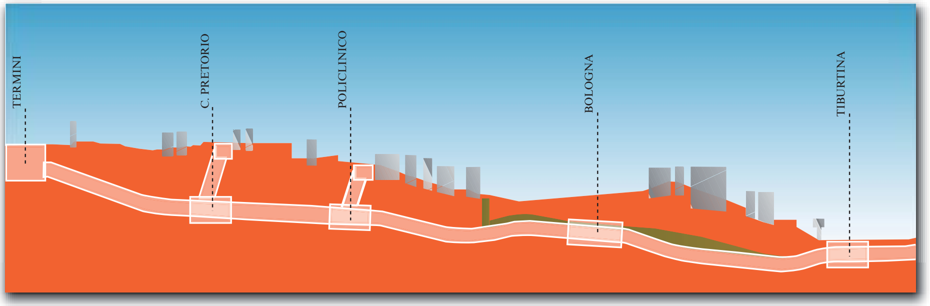


Figure ap.7.2.4 The profile of the Line B.

Figure ap.7.2.5 The line B today.

Appendix 7.3

Railway ring of Rome — **Tunnel construction period**

Aurelia Tunnel — 1981–1983

Table ap.7.3.1 Project information

Location	Rome
Name	Galleria ferroviaria "Aurelia"
Owner	Ministero dei Lavori Pubblici Ufficio Nuove Construzioni Ferroviarie
Contractor	Ferrofir JV (Astaldi, Dipenta, Lodigiani, Sogene Lavori)

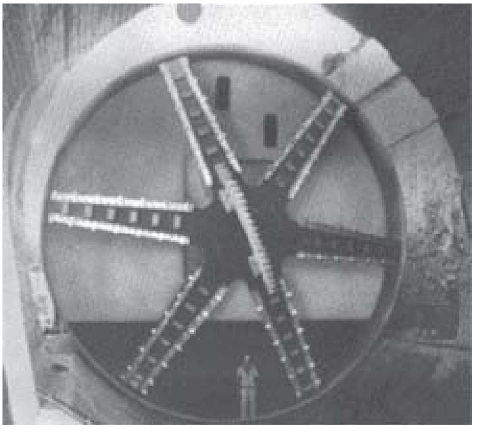


Figure ap.7.3.1 The TBM for excavating the Aurelia tunnel.

1 GENERAL DESCRIPTION

The increasing traffic in Rome forced the authorities to improve the Express Ring Railway (ERR) around the city and to build an efficient rail connection to the Fiumicino international airport.

The most important element of the ERR was the tunnel between the San Pietro Station and an existing partially constructed tunnel with the purpose to connect finally the Maccarese Station with Smistamento Station: "Aurelia" tunnel.

2 TUNNEL CHARACTERISTICS

Table ap.7.3.2 Project characteristics

Length	2246	[m]
Excavation diameter	10.64	[m]
Lining type	precasted segments	
Lining thickness	0,50	[m]
N°segments/ring	8 + 1 (key) + 1 (base)	
Ring length	1,25	[m]
Ring connections	bolts	

Figure ap.7.3.2 The layout of the railway ring.

Table ap.7.3.3 Job site performances

Average advance speed	6	[m/d]

3 GEOLOGICAL-ENVIRONMENTAL CONTEXT

"Aurelia" tunnel undercrosses a build-up area, the overburden is between 20 and 30 m with the exception of a short stretch where it is as shallow as 4m.

The tunnel runs through differing soil conditions. Approximately the first 1 km is in watertight clay; the remaining 1,5 km is situated in water-bearing loamy, clayey sands with interlayer of loamy clays.

The tunnel was constructed entirely below the freatic water table with a water pressure on tunnel crown ranging between 0,5 to 1,0 bar. In sand lens it was locally found a second confined water table with pressure up to 3 bar.

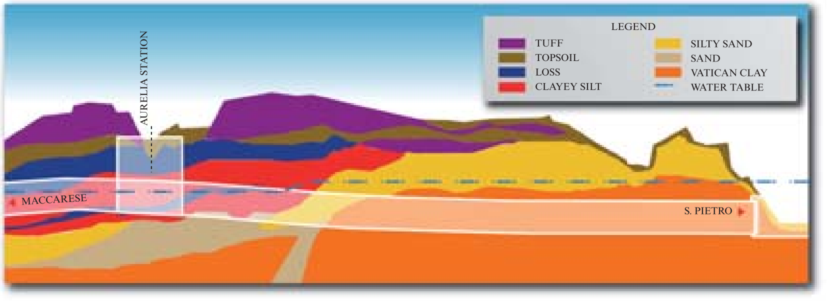


Figure ap.7.3.3 The geological profile of the Aurelia Tunnel.

4 TBM DATA

Table ap.7.3.4 TBM data

Manufacturer	Bade Theelen (Germany) Voest Alpine (Austria)	
Type and model	Hydroshield HDS 1064 OS	
Cutting head	star shaped, six spokes	
Power installed (electric)	1600	[KW]
Thrust (max)	64000	[KN]
Torque (max)	4500	[KNm]
Shield length	8.6	[m]
Additional informations	the erector is able to hoist load up to 7 tons. New developed tail seal: rubber piece bolted to the tail and emergency seal system	

5 JOB SITE ORGANIZATION AND PERFORMANCES

The normal mucking system includes two lobster-arm loader, chain conveyor, conveyor belt as well as trailing sledges with power packs, segment crane and segment conveyor.

Driving in clay was executed without problems: the face did not need any support, and the tail seal worked well against the grouting mortar.

After 1035m the shield was stopped in order to transform the machine into Hydroshield: this operation took 3 months.

Appendix 7.4

West Naples Sewer – Tunnel construction period
Outlet sewer of Bagnoli 1987–1989

Table ap.7.4.1 Project information

Location	Bagnoli (Na)
Name	Collettore fognario Napoli Ovest. Emissario di Bagnoli
Owner	Regione Campania
Designer	CO.RI. (Consorzio Ricostruzione)
Contractor	Lodigiani

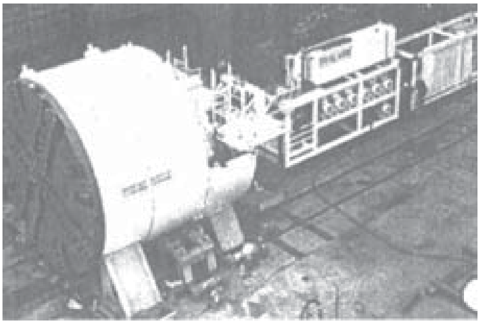


Figure ap.7.4.1 The TBM for excavating the tunnel.

1 GENERAL DESCRIPTION

Bagnoli outlet sewer was designed by CO.RI. (Consorzio Ricostruzione), in the ambit of the Extraordinary Program of "Edilizia Residenziale" – Legge 219/81 – to increase the capacity of the western city sewerage which became inadequate for the sudden urbanization of the area.

This sewer should receive at its head pluvial water from via Padula through a couple of vortex shafts, then after a short junction it continues under via Cinthia to reach the Mediterranean sea in Bagnoli.

Construction of the Bagnoli outlet sewer was completed in April 1991 and in September 1993 the new sewer system was commissioned.

2 TUNNEL CHARACTERISTICS

Table ap.7.4.2 Project characteristics

Length	2600	[m]
Excavation diameter	6,49	[m]
Lining type	precasted segments	
Lining thickness	0,30	[m]
N°segments / ring	6+1	
Ring length	1,20	[m]
Ring connections	bolts	

Figure ap.7.4.2 A picture of the area of Bagnoli.

3 GEOLOGICAL-ENVIRONMENTAL CONTEXT

The soil in which the outlet sewer of Bagnoli was excavated is mainly sand and pozzolana, with some gravels (pomici, lapilli).

The whole excavation was done above the water table.The cover thickness was within 7 to 20 m; the maximum slope was 0,3%; the minimum radius of curvature was 200 m.

4 TBM DATA

Table ap.7.4.3 TBM data

Manufacturer	FCB – Kawasaki	
Type and model	Mechanical shield	
Cutting head	8 spoke disks, 123 cutting bits, 2 overcutting tools	
Power installed	800	[KW]
Thrust (max)	36000	[KN]
Torque (max)	1770	[KNm]
Shield length	6,44	[m]
Additional information:	annular type segment erector, segment handling by hoist. Back-up of 7 trailers	

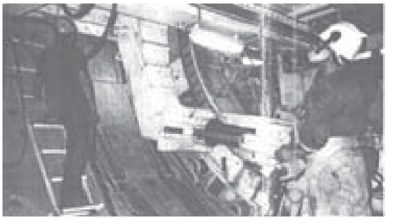


Figure ap.7.4.3 Workers operating in Bagnoli tunnel.

5 JOB SITE ORGANIZATION AND PERFORMANCES

Table ap.7.4.4 TBM data

1° phase advances speed (average):	8,17	[m/d]
2° phase advances speed (average):	13,10	[m/d]
Total advance speed (average):	11,20	[m/d]
Max advance speed (day):	16,80	[m]
Max advance speed (week):	84,0	[m]
Additional information:	Mucking by means of 3 belts belt conveyor of 7,5 kW each. Flow rate: 320 ton/h.	

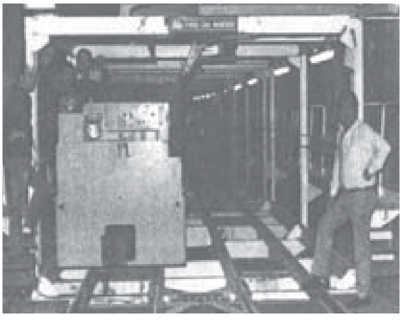


Figure ap.7.4.4 Picture of the back up.

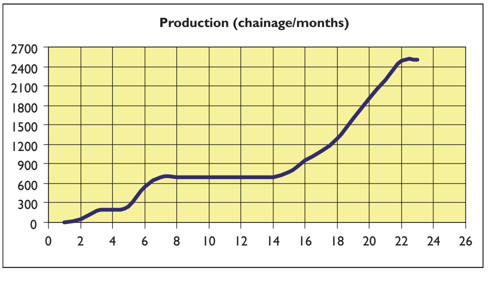


Figure ap.7.4.5 TBM performance.

Naples Rapid Tramway Line (LTR) – Lala – Mergellina stretch

Tunnel construction period
1989

Table ap.7.5.1 Project information

Location	Naples
Name	Linea Tranviaria Rapida (LTR)
Owner	Ansaldo Trasporti (Napoli)
Contractor	Lodigiani

Figure ap.7.5.1 The TBM for excavating the LTR tunnel.

1 GENERAL DESCRIPTION

In the 1980's a project for a modern and efficient tramway line connecting East with West Naples was developed: the "Linea Tranviaria Rapida" (LTR). The first designed layout of LTR foresaw the link between "Piazzale Tecchio" Station and "Ponticelli" Station with a mixed path passing from underground to surface or on viaduct. The tunneled stretch was a 5,5 km long, twin-tube section, excavated by cut & cover technology for 4 km and mechanized shield technique for 1,5 km (Hydroshield).

Nowadays new tunnels have been excavatd for 1,6 km from "Politecnico" Station to "Lala" Station using cut & cover method, and for about 250 m from "Lala" Station to "Mergellina" St using one Hydroshield. This stretch will be part of the Line 6 of the Metro of Naples, connecting "Mostra" Station with "Municipio" Station through 5,8 km of running tunnel and n° 8 stations.

2 TUNNEL CHARACTERISTICS

Table ap.7.5.2 Project characteristics

Length	1570	[m]
Excavation diameter	9,25	[m]
Lining type	precasted segments	
Ring length	1,70	[m]
Ring connections	bolts	

Figure ap.7.5.2 A picture of the LTR train.

3 EOLOGICAL-ENVIRONMENTAL CONTEXT

The "typical" ground in Naples consists of deposits derived from the volcanic activity in the area. Material deposited is cemented on the top (Tuff) or is left in the loose state (*Pozzolana*).

In the specific case of LTR Lala – Mergellina stretch, the excavation was done under a very crowded part of the town, in Pozzolana ground, always below the water table.

The minimum curve radius was 600 m while the, minimum overburden was 8,5 m.

Figure ap.7.5.3 The LTR alignment.

4 TBM DATA

Table ap.7.5.3 TBM data

Manufacturer	Voest Alpine (Austria)	
Type and model	Hydroshield HDS 925 OS	
Cutting head	2 copycutters, central blade	
Power installed	525	[KW]
Thrust (max)	72000	[KN]
Torque (max)	45000	[KNm]
Shield length	8,75	[m]
Back-up length	87	[m]

Figure ap.7.5.4 Piazza del Plebiscito interested by works.

5 JOB SITE ORGANIZATION AND PERFORMANCES

Table ap.7.5.4 TBM data

Best daily advance:	5,1 [m/d]
Stretch excavated:	247 [m]

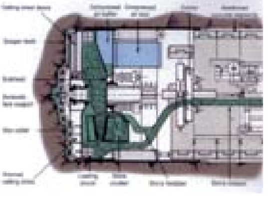


Figure ap.7.5.5 The Hydroshield functioning scheme.

Appendix 7.6

Milan rail connection – Garibaldi – Villapizzone stretch

Tunnel construction period
1992–1994

Table ap.7.6.1 Project information

Location	Milan
Name	Passante Ferroviario di Milano
Owner	Regione Lombardia, Comune di Milano
Designer	MM Metropolitana Milanese
Contractor	Passante JV (Torno, Cogefar, Impresit, Lodigiani, Tettamanti, C.M.B., Collini Progetti e Costruzioni)

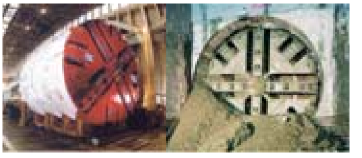


Figure ap.7.6.1 The TBM for excavating the Passante tunnel.

1 GENERAL DESCRIPTION

The "Passante Ferroviario di Milano" is an underground railway crossing the town of Milan from "Certosa" St. (North West) to "Porta Vittoria" St. (South East). It includes various tunnel stretches, constructed with different tunnelling methodologies, and stations connecting the "Passante" with the urban transport net of Milan.

The most-used tunnelling method was cut & cover (the so-called "Metodo Milano") but, for some sections due to the heavy surface and shallow underground interferences, other tunnelling technologies were required.

In particular, from "Villapizzone" Station to "Porta Garibaldi" Station a mechanized tunnelling methodology was used for some sections, due to the presence of surface infrastructures (in particular existing railway lines).

2 TUNNEL CHARACTERISTICS

Table ap.7.6.2 Project characteristics.

Length	3860	[m]
Excavation diameter	8,03	[m]
Lining type	precasted segments	
Ring Type:	universal rhomboidal	
Lining thickness	0,30	[m]
N°segments/ring	6 + 1	
Ring length	1,20	[m]
Ring connections	plastic dowels	

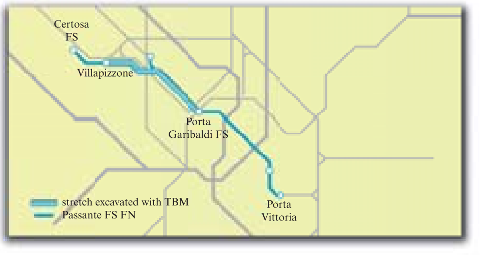


Figure ap.7.6.2 The layout of the Milan crossrail.

3 GEOLOGICAL-ENVIRONMENTAL CONTEXT

Material interested by excavation was the "typical" Milan gravel with silty sand, under water table. Tunnel overburden was between 4 m and 16 m, with an average of 8 m. The min radius of curvature was 360 m.

Table ap.7.6.3 TBM data

Manufacturer	NFM (Mitsubishi license)	
Type and model	EPBS	
Power installed	1600	[KW]
Thrust (max)	55100	[KN]
Torque (max)	13500	[KNm]
Shield length	10	[m]
Back up length	70	[m]
Additional informations	backfilling with extruded concrete	

Figure ap.7.6.3 TBM assembling in Milan job site.

4 TBM DATA

Table ap.7.6.4 TBM data

Working cycle:	3 shift per day, 7 days per week	
Mucking system:	trains	
Advance speed (best day, second section):	24	[m]
Average (monthly):	468	[m/month]
Average (working days):	14	[m/d]

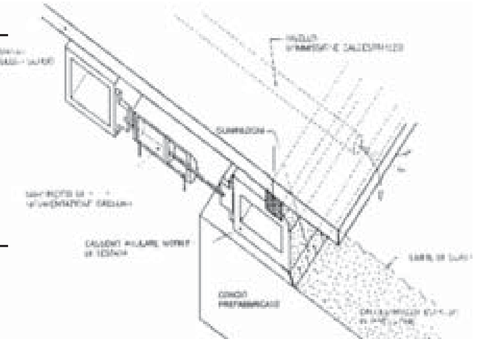


Figure ap.7.6.4 The principle of functioning of extruded concrete.

5 JOB SITE ORGANIZATION AND PERFORMANCES

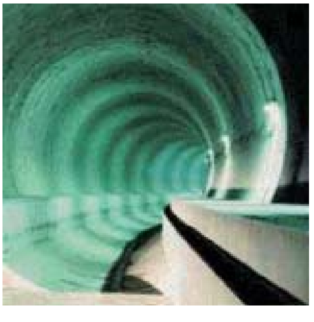


Figure ap.7.6.5 The excavated tunnel.

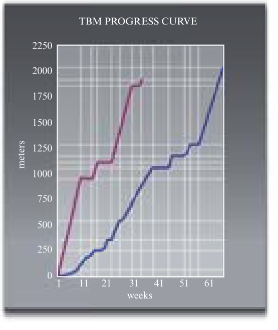


Figure ap.7.6.6 TBM progress curve of Milan crossrail.

Appendix 7.7

Metro of Rome – Line A 2 – Ottaviano – Battistini extension. Battistini – Valle Aurelia stretch

Tunnel construction period 1991–1999

1 GENERAL DESCRIPTION

The Metro of Rome line A Ottaviano – Battistini extension is about 5 km long and includes the construction of 3 stations: "Aurelia Cornelia", "Baldo degli ubaldi" and "Valle Aurelia", and about 900 m of double-track, single-tunnel with an excavation section of 87 m^2, and about 3100 m of single-track, twin-tube tunnels with an excavation section of 35 m^2.

All the tunnels within the "Battistini" – "Valle Aurelia" stretch were constructed using a mechanized tunnelling method. From "Battistini" Station to "Aurelia Cornelia" Station, the 900 m of double-track, single-tunnel section was excavated by a TBM with an excavation diameter of 10,55 m; while the single-track, twin-tube tunnels from "Aurelia Cornelia" Station to "Valle Aurelia" Station were excavated by the same TBM but with an excavation diameter of 6,70 m. The different diameters of excavation forced the choice of a transformable machine capable of excavating the whole stretch.

Tunnels have been excavated starting from "Battistini" Station and from a special man-made structure at "Valle Aurelia".

2 TUNNEL CHARACTERISTICS

Table ap.7.7.1 Project information

Location	Rome
Name	Metro di Roma – Linea A Prolungamento Ottaviano – Battistini
Owner	Comune di Roma (Ente Concedente), Intermetro (Concessionaria)
Designer	Intermetro
Contractor	Condo-Metro JV (Condotte d'acqua – Metroroma)

Figure ap.7.7.1 The TBM approaching the tunnel.

Table ap.7.7.2 Project characteristics

Length	900/3100	[m]
Excavation diameter	10,64/6,60	[m]
Lining type	precasted segments	
Ring Type:	universal	
Lining thickness	0,45/0,30	[m]
N°segments / ring	8 + 1/6 + 1	
Ring length	1,20/1,20	[m]
Ring connections	bolts	

Figure ap.7. 7.2 The excavated tunnel.

3 GEOLOGICAL-ENVIRONMENTAL CONTEXT

The Rome area interested by Ottaviano – Battistini extension is highly urbanized. The cover thickness varies between a minimum of 5 m and a maximum of 30 m.

From "Battistini Station to "Aurelia Cornelia" Station tunnels were excavated mainly in silty clay with local lens of fine sand. From "Aurelia Cornelia" Station to "Valle Aurelia" Station the tunnel crossed sandy-silt and sandy clay, blue clay and recent deposits. Water table level along the whole alignment was variable but in any case it was always above the tunnel crown.

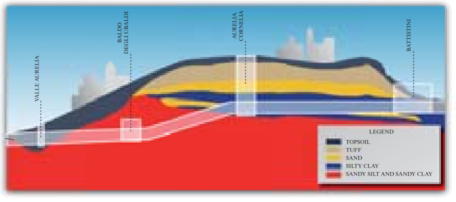


Figure ap.7.7.3 The geological profile of the Line A extension.

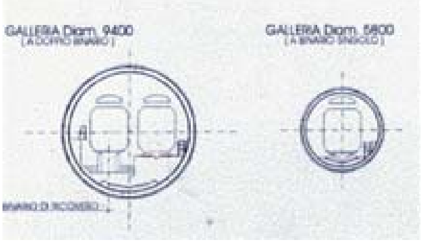


Figure ap.7.7.4 Tunnels with two different diameters were excavated by a transformable TBM.

4 TBM DATA

Table ap.7.7.3 TBM data

Manufacturer	Voest Alpine (Austria)	
Type and model	Hydroshield HDS 1064/660-OS	
Cutting head	HDS 1064: copy cutters: 2 x 0–30 mm crown, 2 x 10 mm invert	
Thrust (max)	80000–30000	[KN]
Torque (max)	4000–3000	[KNm]
Shield length	6,91	[m]

Figure ap.7.7.5 Picture of the back up.

5 JOB SITE ORGANIZATION AND PERFORMANCES

Table ap.7.7.4 TBM performances

Advances speed (best day):	12 [m/d]
Average (work days):	6 [m/d]

Figure ap.7.7.6 Picture of the cutting head.

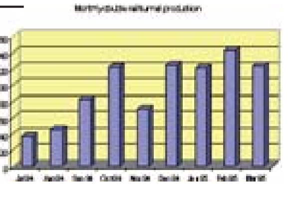


Figure ap.7.7.7 Monthly production of the double-track, single-tunnel construction.

Appendix 7.8

Metro of Genova – Line 1 – Principe–Caricamento–Grazie stretch

Tunnel construction period (1993) 1998–1999

1 GENERAL DESCRIPTION

The Principe–Caricamento–Grazie stretch is one of the most important parts of Line 1 because of the urban-historical importance of the zones interested by the excavation.

On the 14th of January 1998, after a long period of stoppage, Comune of Genova gave Ansaldo Trasporti permission to resume the tunnelling work.

The order consisted in the realization of a metro stretch having a total length of 1787 m, which was added to the 3040 m stretches of Brin–Dinegro and Dinegro–Principe, already in operation from 1990 and from 1992, respectively.

The project included also the realization of “Darsena” and “S. Giorgio” stations, and the completion of “Principe” station.

2 TUNNEL CHARACTERISTICS

Table ap.7.8.1 Project information

Location	Genova
Name	Metro di Genova Linea 1. Tratta Principe–Caricamento–Grazie
Owner	Comune di Genova (Concedente) Ansaldo Trasporti (Concessionaria)
Contractor	Metrogenova JV (Impregilo, Astaldi, Carena, Coopsette, Gepco-Salc, ICLA, Lombardini, SCI costruzioni)

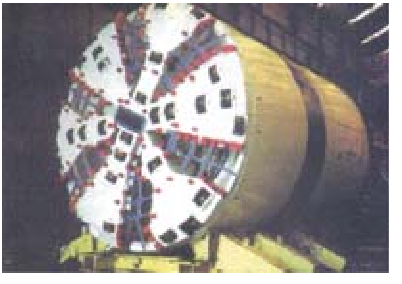


Figure ap.7.8.1 The TBM for excavating the tunnel.

Table ap.7.8.2 Project characteristics

Length	1787	[m]
Excavation diameter	6,29	[m]
Lining type	precasted segments	
Ring Type:	universal	
Lining thickness	0,30	[m]
N°segments / ring	6 + 1	
Ring length	1,20	[m]
Ring connections	plastic dowels	

Figure ap.7.8.2 Metro of Genova tunnel excavation site.

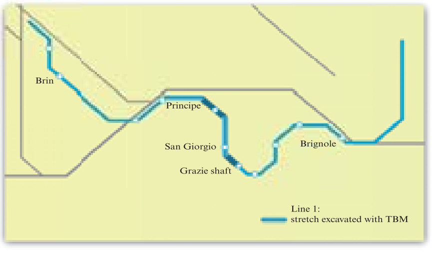


Figure ap.7.8.3 Metro of Genova Line 1 alignment.

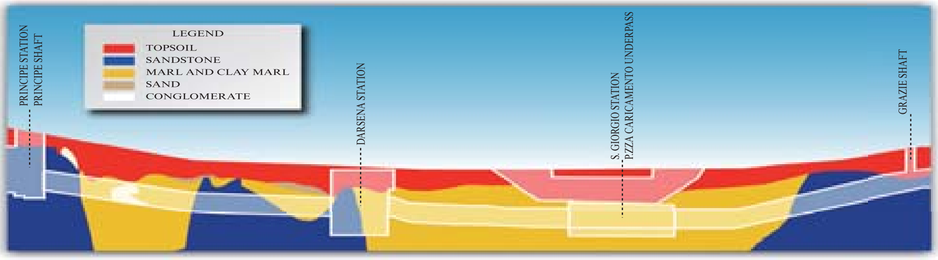


Figure ap.7.8.4 The geological profile of Genova metro.

3 GEOLOGICAL-ENVIRONMENTAL CONTEXT

Tunnels crossed mainly clay-marl ground, below the water table. Limited tunnel stretches were excavated in sandstone and marl, set in a sandy-gravel matrix.

Figure ap.7.8.5 The TBM cutterhead.

4 TBM DATA

Table ap.7.8.3 TBM data

Manufacturer	James Howden – Wirth	
Type and model	EPB	
Cutting head	disk and picks	
Power installed	2150	[KW]
Thrust (max)	3879	[KN]
Torque (max)	n.a.	[KNm]
Shield length	7,95	[m]
Back-up Length	145	[m]

Figure ap.7.8.6 The segmental lining of Genova metro.

Metro of Milan – Line 1 –
Pero – Rho Fiera extension

Tunnel construction period
2003–2005

1 GENERAL DESCRIPTION

With the *Atto Integrativo all'Accordo di Programma per la qualificazione e lo sviluppo del Sistema Fieristico Lombardo*, the municipality of Milan focused the target to realize, within the inauguration of the new Fiera, the Milan Line 1 extension: "Pero–Rho Fiera".

The stretch between "Molino Dorino" and "Rho Fiera" St. (the new Exhibition Pole), starts from the terminal station "Molino Dorino" and arrives in the area destined to the Pole. Connections with future stations of the Regional Railway system and High Capacity Railway line are foreseen.

This metro Line 1 stretch is 2,1 km long and develops in the towns of Pero (1,1 km) and Rho (1,0 km) including the two stations: "Pero" and "Rho Fiera".

2 TUNNEL CHARACTERISTICS

Table ap.7.9.1 Project information

Location	Milan
Name	Metro di Milano Linea 1 Prolungamento Pero – Rho Fiera
Owner	Comune di Milano
Designer	Metropolitana Milanese S.p.A
Contractor	Torno International

Table ap.7.9.2 Project characteristics

Length	1120	[m]
Excavation diameter	6,60	[m]
Lining type	precasted segments	
Ring Type:	universal	

Figure ap.7.9.1 The TBM for excavating the Line 1 extension tunnel.

Figure ap.7.9.2 The layout of the Metro of Milan Line 1.

Figure ap.7.9.3 The profile of the Metro 1.

3 GEOLOGICAL-ENVIRONMENTAL CONTEXT

All the area interested by the Line 1 extension is constituted by glacial-river deposits.

These deposits (geological unit named: Fluvio-glaciale Wurm), are characterized by a thickness of 50–60 m and composed by gravel and sand in silt matrix with local presence of clay. The study of the grain size distribution allowed to subdivide the ground to be excavated in two groups: coarse ground (Type A - blue distribution) and fine ground (Type B - red distribution), see Fig. 4.

In the whole area of the Line 1 extension, the water table level varied from place to place significantly.

4 TBM DATA

Table ap.7.9.3 Geothecnical parameters (coarse ground)

Dry density	17–18	[KN/m3]
Drained cohesion	0	[kPa]
Drained friction angle	32–33	[°]
Young's modulus	75 (z < 10 m)	[MPa]
	100 (z > 10 m)	[MPa]
Poisson ratio	0.25–0.30	–

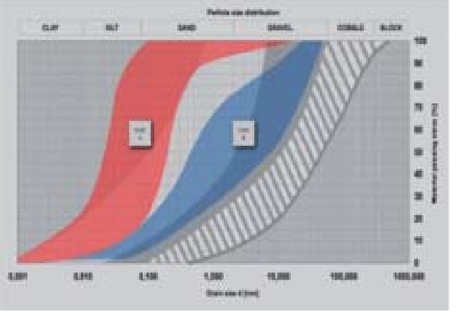


Figure ap.7.9.4 The grain distribution curves of the soil interested by excavation.

Table ap.7.9.4 TBM data

Manufacturer	Lovat	
Type and model	EPB	
Cutting head	40 disk, 38 ripper, 68 scraper	
Thrust (max)	45000	[KN]
Torque (max)	n.a.	[KNm]
Shield length	9.0	[m]
Back up length	67	[m]

5 JOB SITE ORGANIZATION AND PERFORMANCES

Table ap.7.9.5 Job site performances

Average advance speed all-line south (calendar days)	12.5	[m/d]
Average advance speed Pero Pisacane (working days)	12.5	[m/d]
Best daily advance	16	[m]

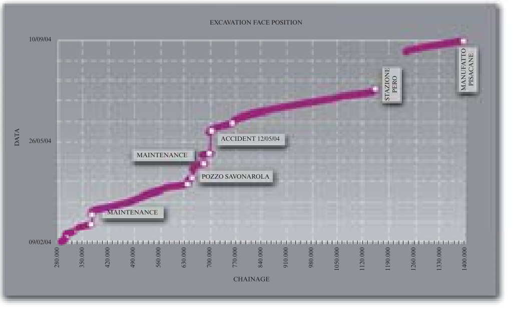


Figure ap.7.9.5 Milan TBM performances.

Rome – Viterbo Railway – "Quattro Venti" tunnel

Tunnel construction period
2002

1 GENERAL DESCRIPTION

"Quattro Venti" tunnel is the most important structure of the "Roma–Viterbo" railway. The tunnel is located in the South – West of the town between "Roma Trastevere" St. and "Roma S.Pietro" St.

The "Quattro Venti" tunnel runs in an heavy urbanized context, between Monteverde and Trastevere districts, characterized by high density of population and by presence of important infrastructures, under or over the surface.

Figure ap.7.10.1 The TBM for excavating the "Quattro Venti" tunnel.

Figure ap.7.10.2 The geological profile of the "Quattro Venti" tunnel.

2 TUNNEL CHARACTERISTICS

Table ap.7.10.1 Project information

Location	Rome
Name	Galleria "Quattro Venti"
Owner	Rete Ferroviarie Italiane (R.F.I.)
Designer	Italferr
Contractor	Quattro Venti JV (Astaldi-Impregilo)

Table ap.7.10.2 Project characteristics

Length	2200	[m]
Excavation diameter	7,97	[m]
Lining type	precasted segments	
Ring Type	universal	
Lining thickness	0,33	[m]
N°segments/ring	6 + 1	
Ring length	1,50	[m]

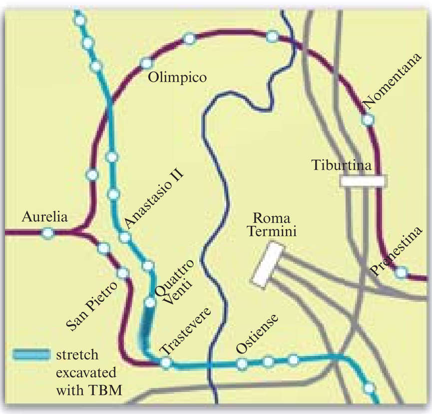


Figure ap.7.10.3 The layout of the railway ring.

Figure ap.7.10.4 A picture of the tunnel.

3 GEOLOGICAL-ENVIRONMENTAL CONTEXT

The geological context of the ground interested by the first 300 m of tunnel excavation is very complex: volcanic deposit (Tuff and Pozzolana), silt-clay alluvial deposit and continental deposit (gravel, sand and clays). After this stretch, the excavated ground is mainly compostled by overconsolidated marine deposit (Vatican Clays), constituted by silt and clay with lens of sand.

The water table is over the tunnel crown for the first 300 m with pressures of 1–1,5 bar, while in the other part of the tunnel water in pressure is present in sand, with pressures until 3–3,5 bar.

Average overburden is 16 m in first 300 m, and about 40 m in the second part of the stretch.

4 TBM DATA

Table ap.7.10.3 TBM data

Manufacturer	Herrenknecht	
Type and model	EPB	
Cutting head	104 picks, 12 cutters	
Power installed	220	[KW]
Thrust (max)	61500	[KN]
Torque (max)	15000	[KNm]

Figure ap.7.10.5 Picture during assembling operation.

5 JOB SITE ORGANIZATION AND PERFORMANCES

The TBM operated in "open mode" in tuff stretches, in "semi-open mode" in "Vatican Clays" stretches, in closed mode in sand and as EPB with polymeric foam in the first stretch of mixed front.

Table ap.7.10.4 Job site performances

Average performances	8,2	[ring/d] (TBM 1)
	9,3	[ring/d] (TBM 2)
Best day production (TBM 1 and 2)	25	[rings] (37,5 m)
Best month production	612	[m] (TBM 1, March 2006),
	635	[m] (TBM 2, April 2006)
Best month production for the two TBMs	1182	[m] (April 2006)

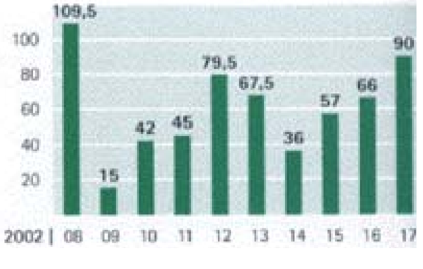


Figure ap.7.10.6 Production recorded in "Quattro Venti" tunnel.

Figure ap.7.10.7 "Quattro Venti" job site area and shaft.

Appendix 7.11

Metro of Naples Line 1– Garibaldi – Dante stretch

Tunnel construction period 2004–2008

1 GENERAL DESCRIPTION

The city transport plan confers to Metro of Naples Line 1 the role of "main axis" of the urban transport, creditable to numerous interchanges, some of them already constructed.

The slope of the Line 1 is about 5,5% along nearly all the stretch "Museo" – "Colli Aminei" Station, with narrow bending radius (even 160 m). The Line develops for the greatest part in underground (nearly always in twin tubes) with the exception of 5 km on viaduct, between "Colli Aminei" and "Piscinola" St. All the stations are underground, until a depth of –47 m. Nowadays the length of the Line 1 is 13,3 km, while the whole ring will be long 25 km.

The so-called "low stretch" between "Garibaldi" and "Dante" will be 3,5 km long with 5 stations (Toledo, Municipio, Università, Duomo, Garibaldi) and will allow the metro connection with the "Napoli Centrale" railway station and with the Circumvesuviana, and through a further interchange with the line 2 (FS).

The works for the "low stretch" began in June 1999 in all the five stations (except for "Garibaldi" Station, which began in December 2000) and completion is expected within the 2008.

2 TUNNEL CHARACTERISTICS

Table ap.7.11.1 Project information

Location	Naples
Name	Metro di Napoli Linea 1 Tratta Garibaldi - Dante
Owner	Comune di Napoli (Concedente) Metronapoli (Concessionaria)
Designer	Metropolitana di Napoli
Contractor	GTB JV (Impregilo, Della Morte)

Figure ap.7.11.1 The TBM for excavating the Naples Line 1.

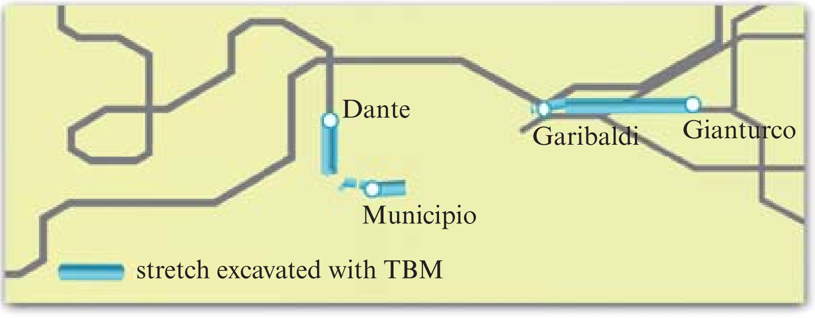


Figure ap.7.11.2 The layout of the stretch excavated by TBM.

3 GEOLOGICAL-ENVIRONMENTAL CONTEXT

Below the topsoil and until 10 meters and beyond, the succession of the natural grounds is a sequence of deposits derived from the volcanic activity of the Campi Flegrei: eruption of volcanos produced the material that is cemented on the top (tuff) or is left in the loose state (pozzolana).

The formation of subvertical fractures that interest the stony part of the formation is known as "scarpine". Groundwater table is at 4.5 m below the Toledo street level.

Tunnel excavation interests for the first 900 m monogranular sand under water table and then compact or fractured tuff, mostly under the water table.

4 TBM DATA

Table ap.7.11.2 Project characteristics

Length	2 x 4000	[m]
Excavation diameter	6,76	[m]
Lining type	precasted segments	
Ring type	universal	
Lining thickness	0,30	[m]
N°segments/ring	6 + 1	
Ring length	1,20	[m]
Ring connections	plastic dowels	

Figure ap.7.11.3 The excavated tunnel.

Table ap.7.11.3 TBM data

Manufacturer	Herrenknecht	
Type and model	EPBS S-238 (Odd Tube)	
	EPBS S-239 (Even Tube)	
Cutting head	136 picks, 2 copycutters	
Power installed	1200	[KW]
Thrust (max)	41000	[KN]
Torque (max)	7300	[KNm]
Shield length	8	[m]
Back up length	77	[m]
Additional informations	backfilling with bi-component	

Figure ap.7.11.4 From the excavating chamber of the TBM: the tuff.

5 JOB SITE ORGANIZATION AND PERFORMANCES

Working cycle: 3 shift per day, 2 for excavation, 1 for maintenance, 5 days per week. Crew in tunnel: 13 people each TBM.

TBM attack: from portal/shaft. Mucking system: lateral tilting wagons.

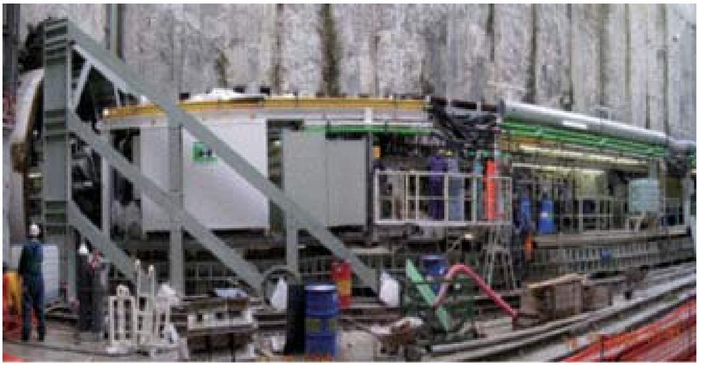


Figure ap.7.11.5 The TBM back-up.

Table ap.7.11.4 Job site performances

Advance speed (best day) S-238:	20,4 [m]
Average Garib.-Duomo (calendar days) S-238:	5,8 [m/d]
Average Garib.-Duomo (work days) S-238:	9,3 [m/d]

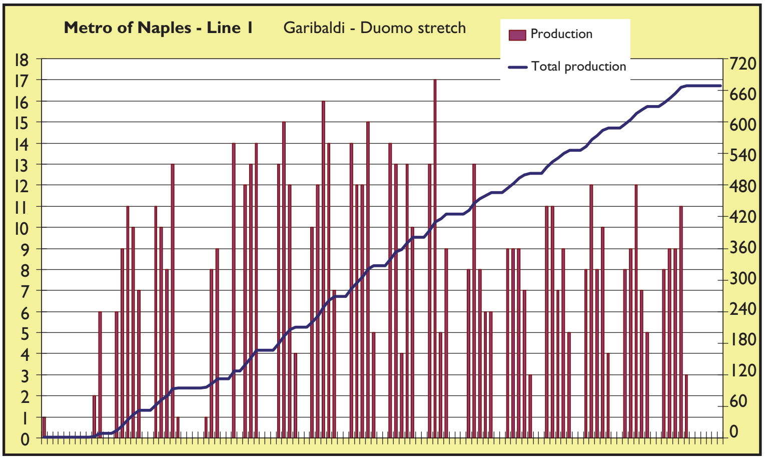


Figure ap.7.11.6 TBM production.

North Milan Railways –
Castellanza tunnel

Tunnel construction period
2004–2006

1 GENERAL DESCRIPTION

The Castellanza twin-tunnel is part of the railway connecting Milano and Malpensa Airport. It consists of two tubes 1.855 m long, with slope of 15‰ and max 16 m of overburden, under the Castellanza Municipality and Olona river.

The excavation was done with a Wirth EPB machine; the 4 by-passes connecting tube "1" and tube "2" were realized with conventional tunnelling method, after ground consolidation. 8 shafs for ventilation and for safety reason were also realized.

Before tunnel excavation it was necessary to consolidate the ground in the foundations of the of Olona bridge piers (railway and road).

2 TUNNEL CHARACTERISTICS

Table ap.7.12.1 Project information

Location	Castellanza (VA)
Name	Galleria Ferroviaria di Castellanza
Owner	Ferrovie Nord Milano
Designer	Metropolitana Milanese
Contractor	Strabag, Torno internazionale, Romagnoli

Table ap.7.12.2 Project characteristics

Length	2 x 1855	[m]
Excavation diameter	8,16	[m]
Lining type	precasted segments	
Ring Type	universal	
Lining thickness	0,30	[m]
N°segments/ring	7 + 1	
Ring connections	bolts	

Figure ap.7.12.1 The TBM for excavating the Castellanza tunnel.

Figure ap.7.12.2 The layout of the Castellanza tunnel.

3 GEOLOGICAL-ENVIRONMENTAL CONTEXT

The excavation concerns sand and gravel alluvial deposits.

Those soils are characterized by a medium-high permeability in the 5–10 m thick superficial zone, and a lower permeability in the more compact zone below..

In correspondence of the underpassing of the Olona river, finer materials are present in the ground to be excavated including silt and sand.

4 TBM DATA

Table ap.7.12.3 TBM data

Manufacturer	Wirth Gmbh	
Type and model	TB 816 H/GS - EPB	
Power installed	1500	[KW]
Thrust (max)	63075	[KN]
Torque (max)	13425	[KNm]
Additional informations	backfilling with bi-component mortar	

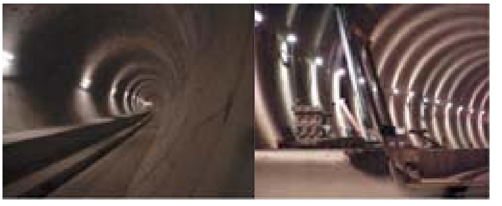


Figure ap.7.12.3 The excavated tunnel.

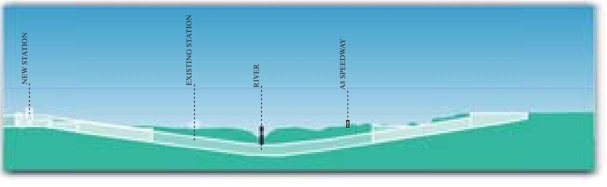


Figure ap.7.12.4 The profile of Castellanza tunnel.

Figure ap.7.12.5 The cutterhead.

5 JOB SITE ORGANIZATION AND PERFORMANCES

For mucking, belt conveyor was used. Muck volume foreseen by the design was about 200.000 m^3.

Figure ap.7.12.6 The TBM into the launching shaft.

Table ap.7.12.4 Job site performances

Average performances (calendar days)	9.7	[m/d] (tube 1)
	13.8	[m/d] (tube 2)
	11.7	[m/d] (both)
Average performances (work days)	14.6	[m/d] (tube 1)
	20.3	[m/d] (tube 2)
	17.0	[m/d] (both)
Best day production	34	[m]

Appendix 7.13

Metro of Brescia – Prealpino – S.Eufemia Lot	Tunnel construction period 2003–2010

1 GENERAL DESCRIPTION

City of Brescia is a deeply urbanized area, where the car traffic is constantly increasing, causing air pollution and problems on efficiency of transports.

The solution is to increase the public service realizing a system to be used for movements in urban area: this is the Metrobus project, that will be the main means of public transportation in Brescia, nowadays utilized only by 20% of the population.

The metro system will be automatically driven, using electric power.

2 TUNNEL CHARACTERISTICS

Table ap.7.13.1 Project information

Location	Brescia
Name	Metrobus Brescia
Owner	Comune di Brescia – Brescia Mobilità
Contractor	Astaldi, Ansaldobreda, Acciona Infraestructures

Table ap.7.13.2 Project characteristics

Length	6000	[m]
Excavation diameter	9,15	[m]
Lining type	precasted segments	
Ring Type	universal	
Lining thickness	0,35	[m]
N°segments/ring	6 + 1	
Ring length	1,50	[m]
Ring connections	plastic dowels	

Figure ap.7.13.3 Segment stored in Metro of Brescia site.

Figure ap.7.13.1 The TBM for excavating the Metro of Brescia tunnel.

Figure ap.7.13.2 The layout of the Metro of Brescia.

3 GEOLOGICAL-ENVIRONMENTAL CONTEXT

Fluvio-glacial deposits, alternations of silty clay and sandy silt, with medium-fine gravel and rare cobbles; sand and gravel with cobbles in silty matrix; lenses of medium-fine sandy silt, with gravel.

Watertable interference from TBM starting shaft to "Stazione FS" Station.

Overburden between 6.5 m and 16.0 m.

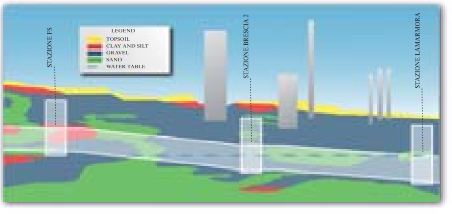


Figure ap.7.13.4 The geological profile of the Metro of Brescia.

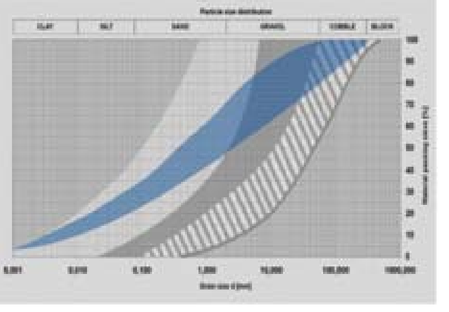


Figure ap.7.13.5 Grain distribution curve of the ground interested by excavation.

4 TBM DATA

Table ap.7.13.3 TBM data

Manufacturer	Herrenknecht	
Type and model	EPB S - 260	
Cutting head	scrapers, rippers, discs, buckets	
Power installed	3000	[KW]
Thrust (max)	81895	[KN]
Torque (max)	12620	[KNm]
Shield length	8,75	[m]
Back up length	114	[m]

Figure ap.7.13.6 Picture of the back-up.

5 JOB SITE ORGANIZATION AND PERFORMANCES

Working cycle: 3 shifts per day, 5 days per week
Crew in tunnel: 13 per TBM, 10 for services
TBM attack: shaft with portal crane
Mucking system: screw conveyor, conveyor belt, muck trains.

Table ap.7.13.4 Job site performances

Average advance (calendar days)	6.9	[m/d]
Average advance (work days)	18	[m/d]
Best daily advance	22.5	[m]

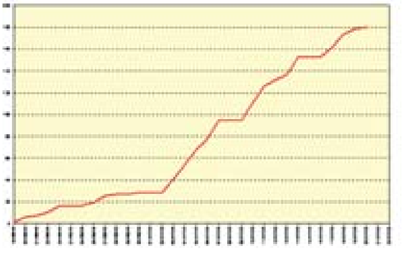


Figure ap.7.13.7 Advancement of the TBM operating at Brescia metro site.

Annex

Contract and construction aspects

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(Englaw/Ladiniabau – Italy)

> "No construction project is risk free. Risk can be managed, minimised, shared, transferred or accepted. It cannot be ignored."
>
> Sir Michael Latham, 1994

A.1 INTRODUCTION

The title of this chapter refers to a subject – "contract" – that, in the mind of the "technical" reader of this book, immediately relates with legalistic aspects and it is thus subconsciously shunned by most "engineers and scientists". But it refers also to "construction aspects", with which then many, if not most, should be decidedly familiar. The author's intention is to refer to specifics of "construction by contract", by which procedure the majority of infrastructure works (including tunnels in urban areas) are actually built, and it purposely avoids the "legalese phraseology" which irks the technical mind.

The principal Actors in any contract are essentially three: Employer (Owner/Promoter), Engineer (Designer), Contractor (Constructor). But also several others come into relevance, at different times with different implications: Government, Administrators, Politicians, Authorities/Agencies (of various kinds), Banks/Financers, Insurers, Suppliers ("in primis" TBM Manufacturers), specialist Sub-contractors.

The "quantum leap" in technical, risk management and interactive and collaborative terms between all actors, advocated by the "technical" substance of this book, will have become evident to the reader who has come thus far. Compared with the present "one-on-one" (often "adversarial" as to individual interests) contractual situation, this ideal procedure is far from being sought and implemented by the types of contract, with which construction is actually achieved at present.

The quotation introducing this chapter will appear to the technical reader as the "summa" of an eminent Engineer with vast experience in construction and its problems. Actually it is not! It will come as a surprise to all those not versed in the contractual world that Sir Michael Latham is indeed one of the most enlightened English judges, to whom the UK government had assigned the task of reviewing the contractual situation and in particular to recommend ways and means to reduce the adversarial atmosphere and the recourse too often to judicial and arbitration procedures. The adjudication process introduced since then has helped to reduce the loads of the courts, by shifting to that procedure the contractual disputes, but did not reduce the levels of the latter. The rest of the world is not in better shape either.

The root of the problem lies in the forms of contract currently in use, not so much for them having an intrinsic 'original sin' of some sort, but by the fact that their use reflects the attitude of the users. Each party tends to shun as much as possible respective risks and tries to gain, depending on the relative strengths of the relationships, maximum advantage from the contractual situation.

In an environment with such strict requirements as tunnelling in urban areas in particular, this has often led to the absurd idea that the level of expenditure for geo-engineering investigations pre-award, essential for the design, rarely surpasses 1% of total costs of project, while the claims situation is commonly running at some 10 to 20% of the total and much worse for some individual cases. It could be sensibly much less, if the "collaborative procedure" intrinsic in the technical management plan advocated in this book could also be implemented in the contract for construction, but so far no such route has been undertaken.

Contractual relationships, contracts and construction by contract have to make a similar "quantum leap" in the light of such appalling and continuing case history. Eminent engineers and experienced law makers have endeavoured to find remedy 'ad hoc' for de-heating the adversarial situation, by devising specific forms of contract and dispute avoidance. There are around the world as many variations of contract forms and types as there are languages, none of which however goes even close to the guidelines ideally sought. Disputes continue to happen, despite different avenues of dispute avoidance being in existence and being implemented.

A wise engineer, Al Mathews introduced in the early 1980s the Dispute Review Board (DRB) to induce the parties to a collaborative attitude: the idea has had tremendous success judging by the exponential growth of cases dealt by that process, but this proves also a parallel growth of argumentation. Other forms of Alternative Dispute Resolution (ADR) such as mediation, conciliation and more recently adjudication have also been implemented. But all these efforts have not solved the root cause intrinsic in the 'adversarial stance' implicit in the current contract forms.

Attempts to change have been made in recent years, by introducing alternative ways to approach contracting, such as "Partnering" and "Alliancing". The effort has been aimed to apply "collaborative" attitudes and in the latter to provide also cost- and time-incentives inbuilt in the contract itself. The draw-back of these forms lies in that the concept is eminently suitable for assuring cooperative and innovative team-work between the actors and for building long-term relationships between Employer and preferred Designers and Contractors, but cannot necessarily secure the Most Economically Advantageous Tender (MEAT), as it implies early choices, which make "competitive" target fixing between alternative contractors practically impossible.

The principles behind the idea however are intrinsically excellent and "Glückauf!" , there has come "light at the end of the tunnel". As recently as February 2006, the Institution of Civil Engineers – UK (ICE) published a Target Version Form of Contract, which appears to incorporate all the positive aspects of Alliancing, while preserving, suitably adapted, the classical ICE format and most importantly the competitive tendering process. So, at last, the long-time dreamed-of "**collaborative contract**" has become a reality and can, in the "construction by contract" process, be the long awaited answer to the technical interaction and risk management co-operation advocated by this book. It is hoped that enlightened players within the industry will understand its potential and that the Fédération Internationale des Ingénieurs – Conseil (FIDIC) also, for the international scene, will follow suit, in a sense 'going back to its origins', but with such innovative ideas.

A.2 CONTRACT TYPES – CURRENT SITUATION

The tunnelling contract environment is characterized internationally by a distinct approach to the question of commercial accessibility to a project. Each major project requires a "prequalification ad hoc", often in conjunction with a discreetly screened "invitation to pre-qualify" by the Employer. Tender documents are then sent only to pre-qualified contractors, with a spread of individual contracts aimed at perceived specific capabilities and know-how of the different contractors. Even when a public call for tender is required, as in the EU, the possibility of pre-screening is still available, effectively stipulating an exacting set of pre-qualifying conditions.

Particular emphasis is given to the extent a tenderer can be considered "responsive". Such assessment is made both at pre-qualification and tender examination stages, with consideration "inter alia" of the following:

- technical competence and experience in the specific type of project;
- assistance and relationship with specialist Designer (e.g. in urban tunnelling);
- current financial position based on the firm's financial statements;
- bonding capacity;
- current amount of work load;
- past history of claims litigation;
- troubles or defaults on previous contracts;
- track record of completion within due time and budget.

Distinction must be made between competitively bid contracts and negotiated contracts. By and large the former category is the most widely used. It is generally the required form of contract for all publicly-funded construction, since it yields a low and competitive price, which ensures taxpayers that their monies are being equitably and cost-effectively disbursed. A trend has however developed, in the most legalistic surroundings (often those previously subject to prominent and distorting political "influence"), where the administration favours awarding to the "lowest tenderer", without consideration of possible different levels of technical and other structural elements making up the "responsiveness" of the various tenderers.

The correct way forward is to evaluate all aspects of the offer, make a thorough scrutiny of technical capabilities, human resources and specific experience, including a face to face Q&A session on all aspects dealing with the way the tenderer intends to face the range of perceived risks and situations: this is the procedure followed to establish what is called in the EU the Most Economically Advantageous Tender (MEAT). The economic advantage for the Employer is evaluated not on offered price alone, but on the assessment of the likely final outcome for the project, technically and cost- and time-wise.

Within the group of competitively bid contracts, there are lump sum (stipulated sum) or admeasurements types, the latter under the form of:

i. schedule of rates only;
ii. schedule of quantities and prices (also called Bill of Quantities – BoQ).

A *lump sum* contract is the oldest form of contract and is still especially popular in the USA. The civil engineering tradition there has been influenced by the necessity of the Engineer/Designer finalizing all the details of the work to be undertaken prior to calling tenders. Main prerequisite is that what is to be constructed must be known exactly at the time of going to tender. It is recommendable not to have to change one's mind, as alterations on a lump sum contract turn out to be rather expensive, more so with the vagaries of underground construction.

A *schedule of rates only* contract is based on a comprehensive list of all the possible items of work to be carried out, without reference to any quantity, as only the final quantities are relevant. The use of such type has turned out hardly recommendable for major projects, since without quantities it is nigh on impossible for the tenderer to plan a correct time schedule and to arrive at a realistic on-cost.

A *schedule of quantities and prices* (or BoQ) contract is the most common form in use. It is based on a list of all items of work to be carried out, with reasonably accurate quantities to reflect the perception held by the Employer and the Designer of the work actually expected to be done. There are provisions to value the contract finally on an actual basis, using the individual rates and also adjustment of rates and quantities for varied, additional or extra work. Some of the advantages recognized in this procedure are:

1. the contractor is paid for the amount of actual work done;
2. while constituting a fair basis for payment there is freedom for alteration of work, if the need arises;
3. adjudication of tenders is relatively simplified, as all tenderers are required to price on a common basis for tender comparisons;
4. the tenderer is given a clear conception of the work involved by way of detailed and fairly accurate bills;
5. most contractors are familiar with this type of contract and its administration and consequently can price the work in a fair and reasonable manner.

The most progressive Administrations appear to depart gradually from the strict competitively-bid contract to a form of *negotiated* contract, where other components, such as for instance alternative design or value engineering, are also taken into account. The origins as mere cost reimbursement contracts (cost plus, i.e. cost + percentage fee or cost + fixed fee) to remedy emergency situations (post-war reconstruction, natural calamities and the like) brought them in disrepute of being an excellent way for contractors to make money, in the shortest possible time. The "new breed" – unfortunately not yet widely accepted, but with one promising perspective, as it will be seen in the last subsection of this Annex – arose from a concept of a target with a profit-sharing clause, with the following successive improvements:

i. cost + fixed fee + profit sharing;
ii. cost + sliding fee:

   a. with single target of cost;
   b. with twin target of cost and time.

The first provides a reward to the contractor who controls costs, keeping them at a minimum. For the overall economy it is essential for the target estimate to be as accurate as possible. If the contractor brings the job in under the target, the savings are shared between Employer and Contractor; over the target there is no extra to be shared. This form is used in "fast track" procedures, by fixing also a Guaranteed Maximum Price (GMP), in conditions where policy (political) decisions or urgency (lack of early planning) require part-concurrent design and construction: given the uncertainties of the underground and the need for proper investigations, this procedure is not to be recommended for tunnels.

The second is an improvement of substance, especially in the twin target configuration. It meets the essential ingredient in any civil engineering contract, that there shall be an incentive for the contractor to carry out the work both cost-effectively and expeditiously. The cost target type offers a linearly proportional monetary incentive to the contractor for completion below target and conversely it applies a similar deduction if the target sum is exceeded. The twin target contract offers distinct monetary incentives (operating independently) for completion below cost target and ahead of time target, while completion above target sum and beyond target date attracts monetary deductions. For both cost and time parameters, it is usual for the rate at which the contractor is rewarded to be greater than the rate at which the contractor is made liable to reimburse the owner.

The "design and build" (D and B) contract is a natural evolution from the negotiated contract, where the Employer ceases to be a direct developer and delegates a large firm or Joint Venture or Consortium to perform on his behalf both design and construction. While such an approach suits the present "hands off" climate of many Administrations, it does have the theoretical merit of bringing in and working together the synergy of the latest "state of the art" Design, suited to the contractor's operating mode and practical expertise, but does not guarantee the most economically advantageous solution for the public, as open competition on such ventures becomes limited. The tendency to limit investigations to a minimum (but to claim later for "unforeseeable" hazards, which proper and time-consuming investigations would have most likely detected) has contributed also to the ballooning of the budget, as the remedial solution is generally more difficult and costly than a preventive plan and care.

The EPC Turnkey contract is a further evolution of the negotiated "performance specification" contract, where an Agency requires facilities of certain defined characteristics and turns to a consortium, who has the know-how, the experience and the resources to build from scratch such facilities with a guaranteed performance: it is generally applied to industrial facilities, where firms are in the market with say patented processing plants, hardly a situation on a par with urban tunnelling.

A relatively recent breed of contracts used for infrastructure facilities, urban areas included, are those called Build-Operate-Transfer (BOT) and the alternative Build-Own-Operate-Transfer (BOOT). Design could be carried out either by the Employer/Developer or delegated to the Contractor (DBOT), if the ultimate owner desires to avoid also initial commitment of capital expenditure and the hassles of design and attendant long lead investigations, even more exacting when dealing with the urban underground. Contractor-supplied financing could be a further requirement (in BOT), the ultimate evolution of which is when the recouping of financing, provided by banks on behalf of the contractor, is achieved by operating the facility via an 'ad hoc' concession for a number of years (BOOT).

The last approach is often integrated in the Project Financing (or PPP – Public Private Partnership) scheme, where the whole project is developed by a group of Promoters including Banks and Insurances, all 'hungry' for *investment*. They have become common in those places where government and politicians have given up the duty of the state to provide, plan and procure funds for needed infrastructures ultimately paid by the taxpayer. Seeing it as an investment opportunity, the promoters have no intention to face risks, nor losses: conversely, at the time of PF proposal, there is very little investigation adequate to ascertain risks, reduce them by appropriate measures and valuate the real residual risk. Thus the pricing of the offer has often, by the nature of the procedure itself, to contain ample margins and watertight contingencies for all potential risks, whether eventuating or not in the end.

The result has been that infrastructure costs, built under such schemes, have soared, in comparison with countries where such practices are not common (Spain, USA). While such a way is tolerable in exceptional cases, e.g. the 4th Elbe tunnel due to the financial over-commitment of German finances in the country reunification after the fall of the Berlin wall, it does not make sense why the public should be goaded to pay for infrastructures two- to three-fold their appropriate cost, only because government has been unable to foster proper procurement. Promoters in Europe, when questioned, admit the economic draw-backs for the end-users of PF-developed projects. So endeavours have been made in some instances, but with "limited lowering of the stakes", to implement "competitive" Project Financing, by having different groups competing. Of course the offers were somewhat lowered.

But the quandry lies in the fact that extensive long-lead (i.e. years) investigations and studies *have to precede* a proper design for a project which endeavours to make risks ALARP – As Low As Reasonably Possible. More so, this is mandatory in urban environment. Further, nowhere money can be borrowed as cheaply as by governments or major public authorities. On the contrary the partners being private PF Promoters (Bankers, Contractors, etc.) are all "in", to make a *safe investment of their money and endeavours* (in today's world awash with cash and so little return for it) and foremost *not to take unnecessary risks*. But as said above, it was not possible to ALARP the risks. The facts from case history are: PF projects cost up to two- to three-fold more than their 'usual' counterparts. People in Europe "enamoured" of the PF should look at the Madrid Metro model (mentioned in next chapter for the merits of its "central coordination applied in advance") of how a *good* public administration can save money for its citizens, as compared with say PPP cases in London and PF in Italy and Europe.

A.3 ENVIRONMENTAL AND PUBLIC IMPLICATIONS

The world population has gradually become aware and very sensitive to environmental aspects: even the most populous nation in the world now emphasizes for its own good the need for the "greening of China". Nowhere like in urban areas the environment appears most fragile, as the space, above- and under-ground, is often so congested and, for the most ancient civilizations, the result of several layers of land use. In such environment the TBM has to be operated very gingerly, to avoid being the proverbial "bull in a china shop".

The urban population, with associated density problems, constitutes the most significant element contributing to the environmental difficulties: as the final "stake-holders" of any tunnelling in their "back yard", the expectation now from the public is that infrastructures of course have to be built, but avoiding *as much as reasonably possible any interference* with the normal life and the individual properties of the citizenship. This concept has to remain paramount in the mind of any developer, designer and constructor.

The big "cut and cover" scars, although temporary, used in the last century as the dominant method to build subways are now unthinkable in the Western world, thanks also to the giant strides made by TBM technology. Part of the public negative response to such monsters as the Boston "Big Dig", a very recent project, stems from, besides excessive cost escalation, the protracted environmental wounds inflicted to the city. Even for much more limited structures, such as subway stations or access sites by shaft to underground diggings, all the Actors have to think in terms of "temporary industrial establishment", subject to severe limitations as to noise, dust, traffic, nuisance abatement, wrapped in a "closed" perimeter structure. This trend, started in congested Hong Kong, has rapidly been adopted around the world.

The reader just has to think how much annoyance is created by the so-called "utilities", seemingly intent to disturb footpaths, thoroughfares and other city surfaces in succession, one utility after another and no apparent coordination. A proper planning for the future requires adoption of common 'utilidors', small tunnels for ALL utilities' use, and most importantly the effort of the relevant authorities to think ahead and coordinate the development together. Fortunately, the technology has given in the last 20 years a substantial helping hand, both in normal and micro-tunnelling, in that mechanised tunnelling makes now possible to achieve what seemed unreachable a few years back, so overcoming the citizenship generalised egoistic attitude of NIMBY ("Not In My Back Yard"). Despite all the good concepts of civic duty, this spirit prevails "within". Knowing this, all actors have to contribute from project inception to a concerted effort so that the solutions chosen for project construction respect the principle guideline of minimizing interference.

Such studies must not be limited to the mechanized tunnelling itself and its potential effect on all overlaying structures, but have to take into account constructability perspectives, such as: safety (within the tunnel and without, i.e. on surface), site establishment, logistics, noise and nuisance abatement or minimization, pollution, muck type and disposal, and any other potential factor, e.g. specific to project location. This is best achieved by adopting from feasibility stage a "collaborative" attitude between Employer, Engineer/Designer and Contractor, where all principal actors gain from sharing the benefits of knowing the respective needs and expertise vis-à-vis the requirements of the project itself, without operating with an 'ivory tower' attitude, nor fear of giving away "family secrets".

As said in the introduction, there are other important actors, besides the principal ones, who play substantial roles in the environmental and public implications. The Government itself, for infrastructures of national relevance, cannot leave, to the "particular" interests of the local politician and politicized (sometimes in opposition) local Authorities (region, province and borough), often final say on matters of apparent minor relevance. These have frequently in the past proven capable of great disruption to the planned development of the project, with consequences in cost and time ultimately paid by the "end users", the local and the general taxpayers. Same public interest considerations have to be applied in dealings with and by the local Authorities and Agencies, whether they represent the province or borough, the "health and safety" aspect, the "fire prevention", the environmental agencies, particular interests or pressure groups.

All these agencies are deemed to represent the public interest. When faced with the problems connected with the urban tunnelling, too often the "public servants" are:

i. either not sufficiently informed "ab initio" and made an integral and responsible part of the Environmental Impact Report (EIR) process;
ii. or, as it too frequently happened, this essential step in the development is carried out in a perfunctory manner.

Only later, at construction time, it occurs that some agencies raise too late, not necessarily together, but often in steps, objections to specific aspects, which properly should have been dealt with much earlier at EIR stage. For the Jubilee Line Extension in London, more time had to be spent to obtain permits, way-leaves and authorizations than in the actual investigations and design (Technical Director – personal communication). For a major HS Rail urban under-crossing in Italy, the progressive "raising of the stakes" by local authorities and agencies, post EIR stage, has meant an initial delay of two years and a complete transformation of site establishment, capped by a tardy requirement for alleged reasons of fire safety during construction, which has forced substantial execution changes, the latter no equal worldwide.

The objection here is not to the fact that such measures had been requested, but to the lateness of such demands: the time for the agencies to raise them was at EIR stage, so that all players would have known the implications, not three years later at beginning of construction. It would have been appropriate for such public servants to consider all these 'last minute' rethinking when properly due, much earlier at EIR time. The general public would have been more appropriately served in cost- and time-savings.

Several cases can be quoted in different parts of the world, where episodes of such nature, catering for "particular" interests, have negatively influenced the regular project development. As a commitment by the Engineer and the Contractor is obtained through a contract, so an equally binding, responsible and enforceable commitment should become an integral part of the process and be obtained in advance from the Agencies and the Authorities. Hence, a substantial rethinking of the construction by contract is required, not as a series of one-on-one negotiations, but as an integrated interaction of ALL the "stake holders", for the ultimate public benefit.

Relevance of the above with the contractual aspect of construction could be queried. However, some of the most disrupting events for any project are those occurring when unforeseeable conditions are met, whether of physical nature or man-made, the latter more frequent in urban areas. ICE includes the latter among "artificial obstructions:" just to quote an example, an archaeological find falls in such category and the sudden stoppage of digging consequent to it and its uncontrollable duration will be familiar to those who met such occurrence. From world experience, the kinds

Figure an.3.1 Nodo di Bologna: the final breaktrough of the two TBMs.

of bureaucratic problems highlighted in the previous paragraphs constitute "artificial obstructions" indeed, which have created cost- and time-effects of much more severe magnitude than the unforeseeable conditions just mentioned. This has led to different outcomes: if the "*central coordination applied in advance*" for ALL stake-holders (the Madrid Metro model) is followed, as compared with others such as e.g. London or Italy or Europe, then the cost per unit length of subway can be halved or even less, an outcome which should make many Agencies and Authorities seriously reflect on their current behaviour and future strategies.

A.4 RISK ALLOCATION AND MANAGEMENT

A.4.1 Risk categories

Types of risk which might be encountered in underground works in urban areas are listed below. Responsibility for design and attendant geo-engineering investigations, for a project designed by (under the control of) the Employer, is in the current contracts customarily allocated to the Employer (if design is "in house"), but in the separate design contract alternative he then has, if and as applicable, redress vis-à-vis the Designer: for this reason the following two categories are now marked E for Employer and D for Designer as applicable (in brackets).

- GEOLOGICAL RISK – connected with the sufficiency of information obtained through the planned investigations:
  - limited investigations and exploration (E and/or D);
  - inappropriate in situ and/or lab tests (E and/or D);

- inappropriate reference model (E and/or D);
- knowledge of rock and soil behaviour (E and/or D);
- knowledge of hydro-geological conditions (E and/or D);
- availability of resources and equipment (E and/or D);
- site accessibility (E);
- lack of systematic face mapping (E and/or D);
- lack of integration of geological data with construction parameters (E and/or D).

• DESIGN RISK – connected with the difficulty of adapting the design to the encountered geo-mechanical conditions, poor construction, experience of the designer and contractual constraints:

  - experience of the designer (E and/or D);
  - uncompleted prediction of the risk scenarios (E and/or D);
  - insufficient definition of countermeasures (E and/or D);
  - constructability of proposed solutions (E and/or D);
  - design flexibility vs. actual ground conditions (E and/or D);
  - loading conditions of the lining (E and/or D);
  - definition of TBM operational parameters (E and/or D);
  - inadequate monitoring controls (E and/or D);
  - inadequate threshold limits (E and/or D);
  - unforeseeable adverse physical conditions and artificial obstructions (E).

• CONSTRUCTION RISK (I) – connected with the choice of an inappropriate or insufficiently industrialized construction technique, occurrence of instability, experience of the contractor and contractual constraints; these risks in contracts are customarily allocated to the Contractor (marked now C in brackets):

  - learning curve (C);
  - incompatibility of the TBM with the ground (C);
  - major mechanical failures (C);
  - inadequate logistics (C);
  - lack of contractor experience (C);
  - lack of personnel training (C);
  - lack of TBM parameters control (C);
  - lack of TBM parameters review (C);
  - insufficient probing ahead (C);
  - inadequate procedures (C).

The above list has been drawn primarily from the design perspective, but there are other risks to be reviewed within the contractual perspective as follows.

• CONSTRUCTION RISK (II) – connected with the potential interaction of the underground excavation with the surrounding, including site interferences and access, and with other responsibilities customarily assigned to one or the other of the main Actors (E for Employer and C for Contractor are affixed in brackets):

  - site accessibility (E);
  - ambiguities and discrepancies in contract documents (E);
  - delay in supply of drawings and instructions (E);

- insufficiency of tender evaluation (C);
- availability of resources and equipment (C);
- problems in labour supply (C);
- quality of materials workmanship tests and samples (C);
- defects of the Works (C);
- safety and security of site operations (C);
- care of the Works (C);
- damage to persons and property (C);
- adverse physical conditions (E);
- artificial obstructions (E);
- use, occupation of Works by others (E);
- force majeur (riot, war, invasion, enemy act, civil war, revolution, insurrection, usurped power, radioactivity) (E);
- interference with traffic (C);
- interference with adjoining properties (C);
- interference with pre-existing utilities (C);
- damage minimization (if unavoidable) (C);
- noise, disturbance, pollution (C);
- damage avoidance to affected roads, bridges, railways, facilities (C);
- finding of articles, structures, things of value or antiquity (E);
- failure to give possession of site (E);
- delays and extension of time (E/C);
- provision for accelerated completion (E);
- variations, alterations, additions and omissions (E).

• COMMERCIAL AND FINANCIAL RISK – connected with the social and political constraints, unclear assumption of responsibility, litigation and security, where responsibilities are customarily assigned to one or the other of the main Actors (E for Employer and C for Contractor are affixed in brackets):

  - failure to provide required security (C);
  - assignment of Works - prohibited (E);
  - assignment and subcontracting of whole Works - prohibited (C);
  - Employer's default (e.g. bankruptcy, receivership, liquidation, etc.) (E);
  - Contractor's default (e.g. bankruptcy, receivership, liquidation, etc.) (C);
  - ambiguities and discrepancies in contract documents (E);
  - removal from site of materials and Contractor's equipment - prohibited (C);
  - required insurances of Works, persons and property, equipment (C);
  - statutes, rules and regulations, licences, patents, notices, etc. (C);
  - consequences of newly introduced (post-tender) rules and regulations (E);
  - failure to give possession of site (E);
  - delay to the Works (E/C);
  - liquidated damages (C);
  - hours of work (C);
  - statutory holidays and leave (C);
  - labour strikes (C);
  - progress payments as specified (E);
  - financing commitments (E/C).

The lists (by all means not exhaustive) provide an overall view of the types of risks most commonly encountered in the construction by contract process. It is in the human nature for each contracting party to try and offload as many risks as possible on someone else. In the "hands-off" present trend, Employers tend to do so on either Designer or Contractor respectively. However the most successful projects have shown on the contrary that the goal has been achieved through a "fair contract". This is characterized by a sensible and realistic basis for the sharing of risks, based on assigning each of them to the party most able to cope with such hazard or occurrence. As stated by one of the most experienced senior judges in the construction field, "*it is easy for the consultant* (to the Employer) *to mistakenly feel that it is in the best interest of his client to try and put every risk on the contractor. He may be tempted to try and justify himself by writing special conditions to reverse every arbitration award or court decision that has gone against him in the past. If he does this, he is merely compounding his client's problems.*" (Sir Ian McKay, IIRC, 1986).

A.4.2 Design risks related to the ground

The question of appropriate risk allocation and management leads to the question of who should bear responsibility for the design and attendant site investigations. In an attempt to minimize Employers' involvement in the project intended to be developed, alternative methods have been devised to implement what are called "performance specification" contracts. There the Contractor accepts responsibility for design, engineering and performance requirements, with total discretion as to how accomplish such tasks. These contracts are either Design and Build contracts or EPC (Engineer, Procure, Construct) Turnkey contracts.

Around the world however the majority of contracts in use are those adopting "design (or method) specifications", which give the Contractor instructions every step of the way as to measurement, materials, tolerances, quality assurance, etc.: the Employer has developed the design and takes responsibility for it, while construction method, workmanship and resources procurement are the responsibility of the Contractor.

This leads to the broad distinctions made previously as to allocation of responsibility, in which Geological and Design risks belong to the Employer (in turn, his Designer), while the Construction risks (I) fall onto the Contractor.

Whoever is responsible for design should carry responsibility for site investigations. Beside logic, the following constitutes a compelling motive. If the "acquisition" of ground for a project is equated to the more normal transaction of purchasing goods, no purchaser would consider buying without knowing the quality and quantity of the goods and their suitability for the purpose. Likewise in the case of ground earmarked for a project, it may not be suitable for the intended purpose: thus, from the proposition of knowing what one purchases, adequate site investigations are required. Of course, if the site investigations reveal that the conditions are not suitable and at the same time there is also no better alternative, one must know what should be done 'a priori' to either make the ground suitable for the available technology or to demand the technology supplier improve the technology, if at all possible, to suit the conditions.

Similarly a designer who purports to produce a design without proper knowledge of the site conditions and material characteristics would be negligent. It is axiomatic that design must be rational. Rational is defined as being based on or derived from reasoning. If a design involving the underground is not based on the adequate knowledge of the site, it then becomes by definition an irrational design: it is not based on reasoning, but rather on suppositions and inadequate guesswork. Additional compelling reason for assigning the owner responsibility for site investigations is that case history pinpoints subsurface conditions as the most important variable in preparing cost estimates for the project, whose accuracy is the Employer's financial responsibility.

In conclusion, it is the owner who chooses the site and prior to its acquisition (and route choice) must take active steps to satisfy himself as to its suitability for the purpose. It is the owner who requires certain works, to be built according to a design appropriate for the conditions of such site, which cannot be ascertained without adequate investigation. The economic benefit of the development is sought by the owner: why then should he gain an unmeritorious windfall at the expense of the contractor, should the conditions worsen beyond the reasonable foreseeability of an experienced contractor? Since the conditions can hardly be said to be under the control of the contractor, while most likely could have been uncovered by further and better site investigations, the owner ends up neither worse nor better off, since in the end he will pay no more than he would have done, had the circumstances become manifest previously.

Such kind of recommendation had been made as early as 1978 in the report "*Tunnelling – Improved Contract Practices*" (CIRIA-UK, 1978), "*which built consideration of risk-sharing and the means of reducing uncertainties between constructing parties onto traditional contracting practices, directly opposed to the trend of retreat into defensive postures.*" (Quoted from Sir Alan Muir Wood, "Tunnelling: Management by *design*", 2000). The USNCTT publication "*Geotechnical Site Investigations for Underground Projects*" (1984) made further recommendations as to minimum requirements for any project, especially considering the different order of magnitude (as percentage of capital cost) between investigation levels (1%) and claims levels (12 – 20% & upwards). Despite slight increases in the former, *the situation is not different over 22 years later*: the industry developers of today have not learnt from the macro-mistakes of their predecessors.

The USNCTT (1984) publication was recommending: "*It is in the owner's best interest to conduct an effective and thorough site investigation and then to make a complete disclosure of it to bidders. …Contracting documents and procedures can provide for resolution of uncertain or unknowable geological processes or conditions before and during construction, rather than afterwards. Adopting a baseline of risks (or geotechnical data) before construction would permit timely recognition of a construction change and cost adjustment during construction, if conditions vary materially*". It took another 13 years and two ASCE publications (on DRB, 1989 and 1991) promoting also such ideas, for the recommendation to take further hold: quoting again Sir Alan Muir Wood, the CIRIA (1978) "*concept has been reinvented in the United States 19 years later under the title 'Geotechnical Baseline Reports* (for Underground Construction)' *(ASCE, 1997)*".

The GBR (Geotechnical Baseline Report) is the most recent evolution of a "Geotechnical Design Summary Report" to be incorporated as integral part of the contract documents. Quoting from the above ASCE publication, "*The primary purpose of the GBR is to establish a contractual statement of the geotechnical conditions anticipated to be encountered during underground and subsurface construction. This contractual statement is referred to as the baseline. Risks associated with conditions consistent with or less adverse than the baseline are allocated to the contractor, and those significantly more adverse than the baseline are accepted by the owner. … The latter conclusion derives from the philosophy that the owner owns the ground. If conditions are more adverse than portrayed in the baseline, the owner should pay the additional cost of overcoming those conditions.*"

The GBR is used by:

- the designer, as a basis for preparing a construction cost estimate, including allowances for specific contingency items, for the owner's budgeting purposes;
- the bidders during the bid phase, for a contractual indication of the anticipated subsurface conditions and geotechnical risks allocated to the contractor;
- the contractor for the selection of construction methods and equipment;
- the contractor and the owner's construction manager during construction, for assessing subsurface conditions and identifying differing site conditions as construction progresses;
- the contracting parties for resolution of disputes related to encountered conditions that are claimed to be more adverse than those indicated in the baseline.

The latter provision should help defuse the continuing debate, in any contractual dispute, as to what constitutes "conditions or obstructions which could not reasonably have been foreseen by an experienced contractor".

An important warning has however to be made at this point. Depending on circumstances, by adopting the necessary measures after an adequate investigation, risk for a defined hazard is brought down to a level *acceptable* to *all* parties concerned, which constitutes the *residual risk* for that hazard. Case history has shown that *even the best investigation* campaign has succeeded in *reducing risk as low as reasonably possible* (ALARP), *not* to *eliminate* it. This has been proven time and again in case of unforeseeable events related to ground conditions and behaviour, even more so in the urban sensitive underground environment. No matter how extended is the knowledge reached through investigations at the final design stage, due to the underground excavation variability, even the most thorough 'assumptions' do not 'guarantee' all the complex reactions of the underground are known 'a priori', fact recognized way back by Terzaghi: "*Unfortunately, soils are made by nature and not by men and the products of nature are always complex.*" (Terzaghi, 1936)

The design task is not finished, in a sensitive urban tunnelling in particular, when the design has reached final stage, even though the experienced Designer has obtained an excellent level of 'practical constructability'. This is just the first, most important step of a continuing "iterative and interactive process", to be continued and continuously refined through the feedback received from the ground re-actions to the actions of man. "*The development of the successful project may be visualized as a convergent helix, illustrating the interactive nature of the process and the constant communications between participants in the design process towards the optimal goal.*"(Muir Wood, 2000) The parameters and ground behaviour *must continuously* be *controlled* and *interpreted* vis-à-vis the original model and the design is to be *adjusted* to suit, to be able to advance safely. As will be seen later, in many instances of 'accidents', human error has also played a substantial part due to lack of recognition of forewarning signals: for this to cause danger in urban area, it is not permissible. So, both for ground and accidental reasons, the tunnel excavation and lining design process does not stop at contract award, but has to be continuously verified through the Risk Management Plan during construction until project completion.

A.4.3 Other design risks

So far the analysis of risks has focused on 'geological' risks: site, ground and site investigations responsibilities, but there are also other aspects listed under 'design' risks. In the former category, on the philosophy that the Employer "owns" the ground and the project development located in it, these risks are customarily assigned to the Owner (Employer), specifically if design is carried out "in house" by his own Designer.

A trend however has started to emerge also for projects 'designed by the Employer', in which the latter tends to engage outside Designers. In such casesh the responsibility and associated risk for adequacy and correctness of investigations and appropriate parameters choice falls on the Designer. There is though a proviso to be raised: the Employer should not impose either limits as to site accessibility or economical and/or time limits (within reason) to the degree of investigation required. If the risk is shifted to the outside Designer, then he cannot be constrained in his judgement, as that could amount to undue interference in his professional tasks. A conflict of responsibilities could also arise, when the Employer decides to have the long-lead investigations carried out independently and not under the control of the Designer: then either the Employer or the site investigations contractor will have to carry ultimate responsibility; in either event, the procedure is not contractually recommendable. If an outside Designer is engaged, he *has to have* control of investigations.

A different situation arises if the Employer decides to embark on a Design and Build package (perhaps also trying "fast-tracking"): then the responsibility also for choices 'connected with the ground' falls entirely on the Contractor and respectively, as relevant, on his Designer. However, given the long lead timing necessary for proper investigations, any Employer would be ill advised to pursue time 'short cuts' and/or put time pressure for these activities. What happens however, when D and B competitive tendering is pursued, is that no contracting organization would have the time (sometimes measured in years) to perform a proper campaign and only limited investigations are performed: because of these limitations, the risk assumption is higher and so are price contingencies. On a 'ALARP risk' philosophy, again not a route to be encouraged.

Coming to the other design risks in the list, with the exception of unforeseeable adverse physical conditions and artificial obstructions, all the other risks properly belong to the Designer in the first instance. They fall back onto the Employer only in case of 'in house' design. In any other situation, when the Designer is independent, engaged either by the Employer or the Contractor, from both there is the tendency to off-load as much risk on the design firm. A warning is in order though: if the Designer makes a mistake (…and it may be a serious one at that!), the reflex redress to the

Professional Indemnity insurance is a poor remedy indeed (nor unlimited either), if meanwhile the design firm collapses.

That is why in tunnel design for urban areas there is no better remedy than Designer experience, experience which comes first and paramount in the list and in ranking. It has to be practical, with senior staff that knows from experience every angle of the design aspects and mechanical and operational parameters of the appropriate TBM excavation technique. Thus, operating either for the Employer (in establishing final design) or for the Contractor (in a homologous activity for a D and B contract or in detailing construction design aspects and controls for the Contractor in a conventional contract), the constructability of the proposed solution is of particular relevance.

But design, as previously emphasized, for mechanized tunnelling in urban areas involves also the "follow-through" of the TBM progress, required from the Designer irrespective of whether he is engaged by the Employer or by the Contractor: design stops only when excavation is finished. So, primarily the Design Consultant engaged by the Contractor has responsibility for advance risk identification and consciousness, prediction of risk scenarios, definition of parameters and thresholds limits, monitoring controls, countermeasures and adaptation of design to actual conditions. The Employer's Designer has similar control tasks too, not the responsibility though, but functioning just as "peer reviewer" for added assurance. Another responsible task for risk prevention by the Design Consultant is to make sure that recommended detailed procedures – a kind of "*Tunnel Manual*" – are established and followed to the letter, so as to reasonably avoid 'human error' by the Contractor's personnel.

Figure an.4.2 Metro Torino Line 1. The connection between a station and the tunnel.

A.4.4 Construction risks contractually defined

The contract defines the responsibilities assigned under the contract to either the Employer or the Contractor and so accordingly the relevant risks. The categories of risk associated with construction have been split in Construction risks (I) & (II) and Commercial and Financial risks. The reason of the former distinction is due to the fact that the contract deals only with the second and third category, but does not specifically refer to any item of the first. It just makes customarily the Contractor responsible for construction method, workmanship and resources, within which the risks listed under the first category fall. A summary analysis of the 'contractually defined' types, with which the 'technical' reader is less familiar, follows in sequence.

As the Employer is the owner of the site, it is his responsibility to provide the site and access to it, when due under the contract, without encumbrances and interferences: failure to give such possession implies recognition of both costs and delay to the Contractor.

As the Employer is the drafter of the contract, its documents should be mutually explanatory: cost and/or delay (if any) consequent to rectification of ambiguities or discrepancies should be reimbursed to Contractor.

As the Employer is (most often) responsible for the design and the supervision of the Works, then any delay in drawings supply or issuing necessary instructions is to the Employer's account. Equally to his account are errors or inadequacies in the design of the Permanent Works (i.e. the Works), while for Temporary Works, usually designed by the Contractor, it is on the latter that responsibility falls.

The risk of incorrect tender evaluation is of course the Contractor's: he is meant to have based his tender on his own inspection and examination of the site and surroundings and on all information, whether obtainable by him or made available by the Employer, and satisfied himself as to correctness and sufficiency, to cover all his obligations under the contract. As construction method, workmanship and resources procurement are customarily the Contractor's responsibility, any risk, connected with procurement of resources (including labour and equipment supply), quality, workmanship, defects, safety and security and attendant care of the Works, damage to persons and property, falls onto the Contractor.

As stated previously, it is the Employer who, as 'owner' of the ground, takes the risk that physical conditions or artificial obstructions could turn out different and more onerous than foreseeable by an experienced contractor. The contract further specifies the "Excepted Risks" for which the Contractor is not liable for loss or damage consequent thereto, including use or occupation of Works by others: the other excepted risks are sometimes collectively defined as due to "force majeure" and comprise riot, war, civil war, revolution, rebellion, etc, radiations.

As safety and damage to persons and property are instead the Contractor's responsibility, then any risk consequent to interference with traffic, adjoining properties and pre-existing utilities (gas, power, water, sewer mains), affected roads, bridges, railways, other facilities and arising from noise, disturbance, pollution has to be to the Contractor's account. A warning is due here. In urban areas more and more the public and the local authorities do not tolerate *any* interference of substance to their environment, their utilities and surface facilities, their properties and themselves. Thus it has become a *must* also for the Employer and his Designer to take into account, at the design stage of any development, these circumstances and such requirements, down to site level and execution details, and not to provide just general Works specifications, leaving then to the Contractor the development of construction design. This practice frequently encounters practical execution difficulties, which *should* have been faced much earlier at EIR stage.

The risk of finding during the course of the works structures and things of value or antiquity is often present in urban environment, especially in case of several layers of land use: the disruption created even by a single archaeological finding is a case in point; the Contractor has to take care, advise and hand over that portion of the site to the Employer, who then carries responsibility for cost- and time-consequences. Any step of possible investigation, at project development and design stage, would be preferable to a stoppage of the works later.

Should delays occur on the progress of the works, these can fall in three categories:

i. "compensable" – those for which the Employer is responsible under the contract, such as e.g. those due to delay in drawings supply, delayed possession of site, unforeseeable conditions, suspensions, variations, etc. The Employer has to assess and grant extension to the contract time (and under the provisions of the individual clauses assesses also extra-costs).
ii. "non compensable" – those for which the Contractor is *solely* responsible, due to defaults on his part, e.g. progress less than estimated and expected or due to TBM underperformance, etc., in other words all the types of risk listed above in Construction Risk (I). The Contractor has the duty under the contract to undertake the necessary steps to recuperate the delay, as no extension of contract time is due (nor extra-costs, of course).
iii. "neutral" – those for which neither Employer nor Contractor bear responsibility: strikes are an example in this category. Extension of time has to be granted by the Employer, but the Contractor has to carry the extra-costs due to the stoppage.

Should delays occur to the works, for which the Contractor is entitled to be awarded an extension of the contract time, in the past it was not permitted to the Employer to request the Contractor to complete the works in less than the extended time, i.e. to request "an acceleration". A few recent contract forms have started, provided the Contractor can and agrees to do so and accepts to be equitably compensated for it, to include specific provisions for accelerated completion.

The Employer is at liberty to order, through the Engineer, "variations", necessary or desirable for the completion and/or improved functioning of the Works. Such variations may include additions, omissions, substitutions and alterations, changes in quality, form, character, kind, position, dimension, level or line and changes in any specified sequence, method or timing of construction. Rather than a risk, it is seen as an opportunity for the Employer to adjust the original concept according to the perceived realities and for the Contractor to submit for approval, even in case of non existence of a specific "value engineering" clause, proposals for job optimization according to his experience, method and equipment used.

A.4.5 Commercial and financial risks contractually defined

Risks referred to in contracts, but infrequent on the part of the Employer are: assignment of the Works (prohibited by contract), default such as bankruptcy, receivership, liquidation, etc. Similar risks exist and are a more common occurrence on the part of the Contractor (hence the need for appropriate prequalification screening prior to tender): assignment of the contract and subcontracting of the whole Works (both prohibited), failure to provide the required security, default such as bankruptcy, receivership, liquidation etc. The contract indicates the remedies allowed to the other party in the event of such fundamental defaults.

The Contractor is prohibited from removing from site materials and equipment provided for the purpose of executing the Works. Appropriate insurances have to be subscribed, also in the name of the Employer, by the Contractor to cover the Works, eventual damages to persons and property, his own personnel and equipment. As the cost of these is included in the tender price submitted and so is ultimately charged to the Employer, a recent trend emerged in complex projects, for the Employer himself to underwrite a "global insurance" to cover the Works, himself, the Contractors and Subcontractors, and Third Party Liability, leaving to the Contractors to insure just their own personnel safety and their equipment.

Compliance with (and obligations and costs related thereto) statutes, rules and regulations, licences, patents, fees, notices, etc. is required from the Contractor, while consequences due to new rules and regulations introduced post-tender are a risk to be met by the Employer. Labour resources procurement is the Contractor's responsibility: so he has to comply with statutory hours of work, pay levels, holidays and leave and has to carry the risk of strikes. Another risk to be met by the Contractor is that of incurring in "non compensable" delays, thus failing to complete the work in due time, in which instance "liquidated damages" become applicable: these correspond to an agreed genuine pre-estimate on the part of the Employer of all the damages he is likely to suffer for any day by which the completion is delayed.

Financing aspects are often a critical aspect of contracting and in many instances have become one of the most relevant risks for Contractors. In contract terms, a payment is certified monthly according to the work's progress and a sum becomes due from the Employer to the Contractor. In submitting his tender, the Contractor has to calculate a project cash-flow and allow for financing the works until the break-even point. Should it occur, as happened many a time has, that the Employer delays payments, the Contractor has the additional burden of 'bankrolling' his operations and the Works, a risk he is not meant to face. As "*cash-flow is the lifeblood of the Contractor*" (Lord Denning), contractual provisions foresee compensation under the form of interest due from the Employer, but often disputes arise as to what is properly due. From case history, especially when claims are substantial and much delay occurs in receiving due consideration, financing costs constitute a paramount risk and have become a real burden for Contractors in many projects.

So far financing aspects of contractual cash-flow have been considered. There is another aspect of cash-flow that both Employer and Contractor often under-estimate. The variability of conditions and other risk factors have a direct influence on work progress, on which most estimates are based. These in turn influence the cash-flow evaluation, both for the Employer to be able to provide the necessary funds in time and for the Contractor to correctly evaluate his rate of expenditure. The 'original sin' is intrinsic in the deterministic way by which project estimates are evaluated, both by the Employer first and the Contractor later when tendering.

Much criticism has been raised, after serious research (most prominently by Flyvbjerg *et al.*, 2002, 2003), about cost overruns and the systemic underestimation of public works (many urban ones included) and the need has been recognized (Reilly *et al.*, 2002, 2004) for enlightened administrations to provide in future an "estimate range", reached via a Cost Estimate Valuation Process (CEVP ©). An allowance for 'contingency' and/or 'risk', expressed as a percentage added to costs, is inadequate, when faced by the present need for an improved risk management, and cannot give any indication as to time evolution of potential costs and time effects, essential to establish reliable cash flow. A tool has been jointly developed since long time by Einstein, MIT, EPFL and Geodata (Einstein *et al.*,1992; Grasso *et al.*, 2002 – referred to in detail elsewhere in the book), useful for the assessment of risks in tunnelling: the Decision Aids for Tunnelling (DAT) system. Through its geological and construction modules and simulation techniques, it allows determining a probabilistic range of costs and times, in function of geologic conditions and variability, possible design solutions, construction parameters and other uncertainties. The use of such a tool provides a better cost and time valuation to the Employer for the whole project and to the individual Contractor for his own tender/contract. Thus both parties can establish a much more reliable, probabilistic estimate and time ranges and consequently cash-flows.

While there are different measures for coping with the various risks discussed, an essential concept to be kept in mind by all actors tunnelling in urban areas is that, due to the specific environment and the human element, any risk has to be kept *"As Low As Reasonably Possible"* (ALARP). Reinforcing what has been discussed elsewhere in the main text, the response has to be through the following key steps:

- choice of the appropriate excavation technique (paramount in ranking);
- risk mitigation aim from project inception (Employer and Designer);
- advance risks identification and consciousness (all parties);
- personnel competence and experience (all parties);
- organization and responsibilities (all parties);
- "ad hoc" procedures development and implementation;
- key indicators identification;
- consistent controls implementation and monitoring;
- preventive measures (to be implemented as and when required);
- "appropriate contract" choice (Employer);
- "team-work" and "common goal" consciousness (all parties).

A.5 "CONSTRUCTION BY CONTRACT" PERSPECTIVES

The brief analysis of the different types of contract indicates a variety of options available in the English speaking world: these are codified in some nations into national standard forms, published by several professional bodies. Authorities may have adopted such forms 'as is', but tinkered versions have generally emerged, with each

Authority keen to 'customize' to suit. Yet this goes against the recommendation of all experienced national drafters, to avoid as much as possible modifications, which risk ultimately subverting the spirit of fairness under which the forms were established and thrived.

In other nations where national forms have not been established as in the English speaking tradition, the proliferation of forms of different authorities and local bodies reaches 'provincial' level. If the scenario is then 'factored' by the language element, then literally there are around the world more contract types than languages, although a form of "schedule of quantities and prices" contract is the most widespread, but with distinct clauses, conditions and applicability. Yet, if all these contracts are considered, hardly any, with the exception of those embodying the "alliancing" process, meets the innovative concept of responsive cooperation and "collaborative" contracting integrating all parties, which this book proves is now becoming a must from the technical aspect, especially for urban areas.

This 'quantum leap' is made necessary by the present "adversarial" state of contracting, worsened by the trend of Owners to "offload" as many risks as possible on others and the attitude to project management as mere administration. Lest such views be considered 'revolutionary', comfort is sought in analogous opinions being expressed by Sir Alan Muir Wood (in "Tunnelling: Management by *design*", 2000) about such attitudes gradually worsening since the early 1980s. It all appears to have started with the 'Thatcherism' of the time, combined with the "hands off" drift of government on the other side of the Atlantic. Under the ICE (and FIDIC, its international counterpart) there had been until then a long standing tradition of a '*fair contract*', with openness for innovation, under the competent administration of an experienced Engineer, with his independence respected and recognized by Employer and Contractor. Although the form of contract could not be called "collaborative", it was the Engineer's attitude to make it so: this is no longer available, as the Engineer role too has been progressively driven by commercial considerations, as noted below.

'Project management' came into fashion instead, with active participation of the legal profession. "*It appeared modish to encourage a drift towards increasingly partisan and complex contract documents with increasingly defensive postures of the Employer reflected in the Contractor, forced to compete at sub-economic rates for a pessimistic outcome, while the Engineer was compressed into a subservient role with overall duties fragmented and diminished… The result, as has been widely recognized but too rarely correctly diagnosed, is that projects, lacking the necessary cooperation across the component parts, are well below optimal achievement…The consequent level of litigation, a direct result of the deliberately confrontational contracts imposed by the influence of the lawyers, should immediately recede.*" (Muir Wood, *ditto*) This exhortation has fallen on deaf ears, despite the earlier findings by the Latham Report (1994) which suggested, unheeded, that, by *fostering cooperation rather than confrontation*, *an economy of some 30%* and more would be readily achievable. Unfortunately no reversal of trends has yet occurred: recent editions of some contracts (e.g. FIDIC) have actually stiffer conditions.

The situation was not improved by the attitude to project management, still current. Management was seen as administration, the manipulation of the tools of management, the understanding of the bureaucratic machine, rather than as the art of blending and synthesis across the diverse contributions to the successful project management. "*Management as administration supposes that the engineering is controlled by directives and undertaken in individual cells, each cell concerned with a particular aspect which is defined and recorded. Administration endeavours to police each aspect to prevent change which might otherwise interfere with other aspects of the project. Administration is remote, avoids technical debate, being incompetent, on account of inadequate technical understanding and an inappropriate structure, to engage in interactive leadership, reacting ineffectually to the consequences of change without active engagement in their anticipation.*"

"(This attitude) *springs essentially from a legalistic approach to project management which derives from the thesis that interests of participants are only guarded by precise definitions of each transaction or undertaking treated separately from any other…On the contrary, the essence of the successful project is the recognition that the parties must share to some degree the benefits of a successful project implicit in the process of optimization, rather than reliance on scoring only by diminishing the benefit of other parties of the project, to the detriment of the project overall…* (The professional engineer) *should ensure that the interests of the client and of the public interest (the 'stakeholders', the social and environmental consequences) should predominate over the reliance on short-term market values of so many politicians and accountants.*"(Muir Wood, *ditto*)

The evolution of contract forms and of contractual stipulations in the last 15 years, under both the 'privatization' and the administrations' "hands off" attitudes, has tended instead to favour the Owners' progressive shedding of risks:

i. by adopting Design and Build contracts, where all the risks connected with the design and ground investigations were off-loaded onto the Contractor and his Designer;
ii. by implementing BOT and DBOT contracts, respectively requiring Contractor-supplied financing and Design also, where need for financing procurement is left to a later stage and where the award to a "General Contractor" allows the Employer also to avoid direct project management and contracts coordination and supervision;
iii. by going to BOOT contracts, where even the capital investment for the works is provided by the Concessionaire, who recoups it through fees levied for the number of years of the concession;
iv. by favouring the recent trend to Project Financing, which delegates to private entrepreneurship all the Owner's previous responsibilities.

Where a conventional form of contract has been adopted, again the tendency of Owners has been to impose over-rigid terms and allocation of risk, in the face of uncertainties which are in part governed by their own transactional and adversarial behaviour. As highlighted also by Muir Wood, characteristics spelling trouble for a project may include:

- *"risk imposed on Contractor in preference to adequate risk analysis and control;*
- *uncertainty of composition of project and performance criteria;*
- *absence of assurance of feasibility* (by critical peer review)*;*
- *absence of prior agreement with regulatory and other authorities;*

- *site investigations inadequately related to construction and not treated as a project resource available to participants;*
- *evidence of intention to rely upon commercial relationships enforced by law in preference to professional relationships built on mutual trust.*

It is also to be noted that international financing agencies and Development Banks are not free from some of the above defects in the conditions that they impose."

There is no risk-free construction and, given the environment and the people, nowhere are the risks higher and more exacting than for tunnelling in urban areas. This book has shown the required technical Risk Management Plan (RMP), which permits the reduction of risks *as low as reasonably possible* (ALARP). All these procedures involve cost and time: allowance in cost and time has to be taken into consideration, both for the RMP and the eventual residual risk. They cannot be ignored by the Owner or by whoever accepts responsibility for them, Designer or Contractor. The financial consequences have to be assessed and compensated for. Faced by a bevy of risks, from technical to financial, those on whose shoulders the burden should properly fall have taken alternative ways to 'off-load' them in turn:

i. one way has been via Insurance, which has been used, both by Owners, Contractors and Designers, as the most readily available and cheapest management tool: this aspect will be dealt with in the next subsection;
ii. the other way (in BOT, BOOT and Project Financing) has been to make sure that sufficient contingencies were allowed in the global price by all actors (Promoters, Contractors, Designers, Banks/Financers, Insurances) so that at least the financial risks, even in event of the worst hazard, would in all circumstances be covered. The situation is worsened by the practical impossibility for the General Contractor to carry out in advance the long-lead geo-investigations essential to ascertain and correctly evaluate risks. In effect this method transfers onto the taxpayers all costs real and perceived, whether they in the end eventuate or not. Thus the public becomes the insurer of last resort, to the economic advantage of the promoting group.

The so much touted "win-win" solution for both public investors and private sectors is in effect so at the detriment of the public taxpayer. In such circumstances, in nations that favour these procedures, it is no wonder that infrastructure costs, built under such schemes, have soared per unit length to levels from two- to three-fold in comparison with countries where such practices are not common.

It is about time that the trend be reversed, even more so for urban environment, where risk must be adequately investigated in advance, so as to be known, quantified and minimized. The Owner and his management need to be deeply embedded in the design and construction process, reverting to the good practice of old: "technical before contractual". There is no place for "hands off" management, which only obstructs the interactive elements of a "collaborative" team work. A special skill has to be favoured by all actors: that is the anticipation of approaching problems, as opposed to the far more expensive practice of reacting on their arrival. As shown in Figure an.5.3 a proactive approach, with constant communication between participants, has to be implemented, given the iterative nature of design and construction. This book has shown that in mechanized tunnelling, especially in urban areas, the

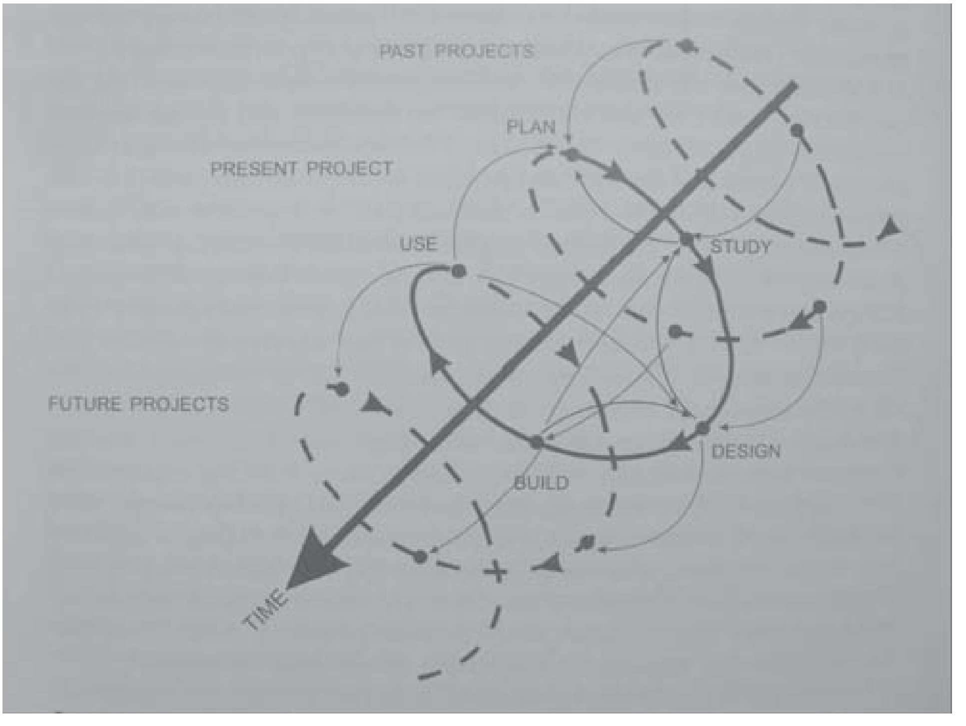

Figure an.5.3 Iterative nature of Design and Construct (After Muir Wood, 2000).

"black box" approach, i.e. to just investigate a risk event 'a posteriori', is no longer admissible.

Unfortunately, as seen before, the forms of contract presently applied favour one-on-one antagonism rather than team collaboration. To try and remedy such attitudes, many efforts have been spent in the industry by eminent engineers, authors and enlightened legal professionals and administrators. Only recently, as a remedy to try and reverse the trend of large cost growth for major projects (epitomized by Channel Tunnel, Great Belt, London Jubilee Line Extension, Boston Central Artery), the Washington State Department of Transport (WSDOT) has promoted instead a "hands on" procedure of a new cost estimating process, including cost validation and risk assessment. This Cost Estimate Validation Process (CEVP ©) goes a long way in the direction of the Risk Management Plan (RMP) advocated by this book. The fact that in USA it has been adopted now at national level by the Federal Transit Administration (FTA) proves the intrinsic merit of such ideas, developed by proactive Administrations implementing the 'evolutionary' ideas of innovators (D. MacDonald, J.J. Reilly, 2002).

Another attempt to reverse the adversarial attitudes has been made by introducing the "Partnering" process, a procedure initially adopted in the USA in the petroleum, processing and power generation industries and gradually now finding some foothold in the US construction industry. The Construction Industry Institution (CII) defined it: "*Partnering is a long-term commitment between two* *or more organizations for achieving specific business objectives by maximizing the effectiveness of each participant's resources. The relationship is based on trust, dedication to common goals, and an understanding of each other's individual expectations and values. Expected benefits include improved efficiency and cost effectiveness, increased opportunities for innovation and continuous improvement of quality products and services."*

In design and construction projects it constitutes an organized effort of all parties to improve communications and to devise more productive ways to work together. The draw-back is that it is established as a voluntary process, although endorsed by key professional societies. The track record has proven it suitable as *integration* into existing forms of contract to *reduce* the confrontational attitudes, but although fostering cooperation it does not constitute, nor does it transform a standard contract into a "collaborative" one, as it is the aim in this book. "*For the time being, partnering for major underground projects will doubtless be confined to those* (mainly private) *Clients who do not depend excessively on finance by commercial banks or dominance by accountancy, with its concomitant distortions of risk allocation, legalistic relationships and management separated from engineering.*"(Muir Wood, *ditto*)

Project "Alliancing" is also a recent idea, introduced by BP in the early 1990s to achieve remarkable performance in the development of offshore oil and gas fields in the North Sea, with both estimate savings and outcome savings in cost (some 22%) and time. As later applied to the construction industry (Australia being at the forefront), the outcomes in time and cost saved have also been positive, but not to such high levels, although always achieved even in the face of great adversity: cost and time overruns have not occurred.

"*A 'project alliancing' is where an Owner and one or more Service Providers (Designer, Contractor, Supplier* – of TBM for instance*) work as an integrated team to deliver a specific project under a contractual framework, where their commercial interests are 'aligned' with actual project outcome*" (Jim Ross, 2003).

Under a traditional contract form, risks and responsibilities are allocated to the parties, with individual commercial and legal consequences, in case of failure to manage their respective duties. Under an "alliance", the participants become like a consortium and assume collective responsibility for project delivery and for all related risks, but share in the 'gain' or 'pain', depending on whether the actual project outcome is lower or higher (in cost) than the pre-agreed target they did jointly commit to achieve. The saving in time is also practically accounted for, as most costs normally run with time and so time saved is reflected in lower outcome cost. The risk allocation in alliancing is still present, not through legal liability, but through the reward/risk arrangement. In essence alliancing espouses a modified form of "target contract".

There are implications in the arrangement. The Target Outturn Cost (TOC) is developed by all parties together, thus there is a need for total transparency; an 'open book' and audited administration is to be conducted during the course of the project with equal transparency. The compensation of all parties is made in three parts:

1. total reimbursement of direct project expenditure including project-specific overheads;
2. a fee to cover corporate overheads and profit;
3. an equitable sharing of gain/pain depending on actual outcome.

The *maximum* risk for non-owner participants is that the 'pain' under item 3 could equate the fee under item 2, i.e. the recovery will be for item 1 only (nil overheads and profit). Alliancing remains open to the criticism that the target value, fixed between participants in absence of price competition, cannot assure the Owner that the most economically advantageous offer has been received: the other participants though would have a lot to lose both in reputation and business future, if the target would have any whiff of being inflated. The participants need also to waive legal rights to pursue each other, which they would normally have, in order to give proper commercial foundation to the alliance.

Because of the collective share of risks, the participants tend to be more aware of the potential risks and to implement more effectively risk management practices. It is a procedure aimed at building long lasting relationships between participants, but requires involvement and commitment at all levels and a substantial cultural shift from traditional adversarial to integration and collaboration. The absence of price competition of the normal tendering process seems to require extra care in the choice of participants and tighter checks and balances of all aspects before and during implementation.

From the above examples, only recently efforts have strived to avoid adversarial attitudes, but the diffusion in the industry has been so far limited. Alliancing appears the closest to the idea of collaborative contracting advocated by this book, but its most serious drawback appears to be the difficulty to implement within the process a form of competitive tendering. As we shall see later, the alliancing philosophy has been adopted by the Institution of Civil Engineers (ICE) under the form of a target contract, which has the benefit however of being couched in the familiar ICE form, tried and tested for more than 50 years and regularly updated, and to allow the usual competitive tendering process. This gives great hopes for proper implementation of the collaborative contracting advocated by the author and will be dealt with in the final section.

A.6 INSURANCES AND OTHER MISCONCEPTIONS

Insurances are essential tools in the tunnel construction by contract, to provide the various actors with a buffer against risk. So in the last half century it has been common for Employers to make the Contractor responsible for the safety and care of the Works, but requiring him as well to insure the Works against possible adverse occurrences of any kind, including of his own making. Faulty design (if any) of the Works was however excepted (but not for Temporary Works constructed by the Contractor), since the Designer (either 'in house' or external to, but commissioned by the Employer) had sole responsibility of it and had to be covered by a Designer's "professional liability" insurance. Damages caused by Employer's agents or personnel were not originally the Contractor's or the Works' insurance responsibility, but for a couple of decades a tendency did surface for the insurance to be made in the joint names of Contractor and Employer, so including the latter damages also.

The Contractor also had to underwrite insurances to cover:

1. "Third Party Liability" (TPL), i.e. potential damages to persons or properties affected by the Works and the Contractor's operations;

2. "social security", against accident, disability and pension to the Contractor's own personnel;
3. "plant and equipment", against damage during their operation.

These insurances were in use for the last half century and more, without any substantial complaint being raised by the Insurers until about the beginning of the 21st century.

Then the tendency of Employers to off-load tunnelling risks on Contractors and Designers became prevalent. Some of the Employer previous design choices in urban areas had already elected to implement the "do nothing" option, for 'minor' (but very annoying to inhabitants and owners) damages to buildings affected by the surface subsidence path caused by the underground tunnel. While such choice could be 'endured' from a major public Authority, once the design responsibility was transferred to the private Contractor in a Design and Build contract, no longer became viable and the trigger threshold was set lower at more sensitive levels. The most serious offloading of responsibility from Employer to Contractor was however in respect of the ground conditions, which potentially entailed the risk of catastrophic consequences. Whether it was serious damage to high rise buildings, a railway embankment, a gas, power or sewer main, a trafficked thoroughfare, a busy commercial shopping centre, having the Contractor covered in turn the risk through his Contractor's All Risks (CAR) insurance, this meant for Insurers a tremendous raising of the stakes compared with before.

It was then noted that the general trends in tunnelling, of particularly high incidence in urban areas, were:

- prevalence of Design and Build arrangements;
- tight construction schedules (some 'fast track');
- inadequate long-lead investigations, due to above two factors;
- one-sided contract conditions;
- restricted pool of works available;
- fierce competition among contractors;
- projects design still based on assumptions and inadequate investigations;
- consequent high risk construction methods;
- 'Third Parties' (i.e. affected public) more claim-conscious;
- exponential increase of TP Liability;
- "life cycle" implications for deferred delivery BOT projects.

From the high incidence of major tunnel losses, the insufficient income from premiums set to historical incidents level, having to indemnify repairs and huge consequential damages, with repair costs alone exceeding the original construction costs, the Insurers came to the conclusion that the Owners' attitude had driven the contracting industry to use the insurance as the "*cheapest risk management tool*". The case history, presented at the International Tunnelling Association (ITA) congress in Seoul in 2006, shows that in the last 12 years, but with a marked jump of occurrences in the last 5 years, 13 (out of 19) major catastrophic losses had occurred in metropolitan areas, for a global value (if those yet to be exactly quantified were assessed and included) of about € 500 million out of a total of € 800 million. Worthy to note that of the 13 cases, 6 were for TBM incidents: the cause of such events, most of which involved tunnel collapses, was due for 8 cases to *faulty workmanship* and for 3 cases to *faulty design*. For all projects, the average individual delay caused by the occurrence amounted to 19 months.

Such delay has prominent relevance, since it shows that the accident caused not just the additional costs of reconstruction and reinstatement, but also huge consequential costs (for the modern private Promoters) due to Delay in Start Up (DSU) and Loss Of Profit (LOP), all required from the bankers participants to be covered by insurance in BO(O)T-Project Financing. Aggravating circumstances are in that "*human error*" is the principal cause for all events, since faulty design is also due to it, whether in the assessment or in the inadequate level of investigations essential for a rational design.

A key operative, on occasion blamed for human error, is the TBM 'operator'. The latter is a misnomer; operator is for a loader or other common device, not for deca-million equipment, so critical for so many aspects and potential consequences, especially in urban areas. It is about time that such figure, with so much responsibility piled-on, receives from a 'professional' point of view utmost attention *from all parties involved*: his *track record* has *to be proven*, before he is allowed to 'operate', like for a train conductor or an air pilot. With so much at stake, there should be a *licence* system to qualify the person as TB Machine Pilot.

This concept of an 'actual licensing' of sorts, particularly for the delicate urban environment, should be extended to *key staff*, for all contracts. The risk management capability is a primary responsibility of the Project Manager and of the Tunnel Manager, who then have to show proven experience record to be able to act as "Risk Managers", applying the RMP procedures and with the know-how to interact with the Design Consultant in case of abnormal events.

Unfortunately 'human related risks' are more random and difficult to control, as case history has proven for the catastrophic tunnel fires events. No matter how much 'procedural control' is applied, men are fallible. The automobile industry has endeavoured to develop 'reactively-safe cars' (since the development is still current for 'pro-active devices'), to obviate the impossibility of individuals' control. The same concept *should* be *pursued by the TBM industry*, to develop, to the extent possible, "*fool-proof*" *machines* and associated equipment. It is not a such outlandish concept, since so many detectors, sensors and video-cameras have now been installed to 'help control'. The problem is that the TBM Pilot has so many tasks to follow and signals to detect, that a little 'automated' help would be welcome. The reference is, for instance, to automated servo-control systems, like automatic detection of over-extraction, slurry injection around the shield to reduce 'physiological' potential volume loss, 'tomography' detection of real tail-void dimensions, self-compensating 'pressurization' of the excavation chamber, etc.

The latter is something already fitted to the EPBM of some manufacturers and is one of the devices suggested elsewhere in this book to compensate pressure drop in case of machine stoppage of any duration; its cost is literally marginal in the contest of the overall machine and comparable with the cost of human repeated intervention for a manual operation. Thus it should become standard fitting on ALL EPBM. As TBM performance no longer appears to be critical for urban tunnels (metro in particular, where stations have instead become the stumbling block), so it becomes appropriate for TBM manufacturers to apply technology R and D to help control and management of human-related risks.

If case history is analyzed further, as remarked also by the main editor of this book, catastrophic development (whether major or minor) has generally been accompanied by “tell-tale” advance warning signs, which went unheeded at the time of the event. This was due either to genuine human error or ‘condoned’ ignorance of the relevance of the sign: a further proof could be that, after a ‘main accident’, usually there has been no further occurrence of the kind, although no change of human resources

Figure an.6.4 Alliados Station in the Porto Metro Line C.

intervened. Against such events the continuing Risk Management Plan advocated in the book is the safest insurance: had this been in force, heeding the warning sign would have avoided later catastrophe. Similar conclusions come from the analogous RMP advocated now by tunnelling Insurers, details of which are mentioned below. In aviation, the "black box" is interrogated 'after the event', i.e. after the 'failure', to ascertain the possible cause. In urban tunnelling, even thinking of the 'flight recorder approach' cannot be tolerated, given the exacting demands and catastrophic consequences there. The RMP is the "pilot box" enabling the steering of the tunnel trough and removing the risks.

For Insurers and their clients, should the trend indicated above continue, tunnelling projects, especially in urban areas, would quickly become *uninsurable*, unless rates were drastically increased, substantial deductibles implemented, actual cover range restricted to various degrees. In the alternative, the "insurability" could be still pursued, but with a complete reversal of approach, especially from the technical point, by implementing a "Joint Code of Practice for Risk Management of Tunnel Works", a move initiated in 2002 by the British Tunnelling Society (BTS) and the Association of British Insurers and endorsed on a world basis in January 2006 by the International Association of Engineering Insurers (IMIA) and the International Tunnelling Association (ITA). Such a code aimed to set minimum standards for "risk assessment" and "risk management procedures" for tunnelling projects and to define clear responsibilities for all parties involved. Had such a code been adopted, it would have meant that proper ground investigations and design checks would have been done, appropriate risk management procedures would be in force, so that 6 out of the 13 major accidents, in urban area tunnels, would have been avoided. IMIA also made a concluding warning: "*Complex organization, a plethora of law-related contractual conditions and spreading responsibilities widely do not compensate for fair contract, fair prices and the involvement of experienced people willing to take on the necessary responsibilities.*" (Boston, 2006)

A.7 ICE TARGET COST VERSION CONTRACT

Perhaps it is no mere coincidence that at the beginning of year 2006 from different participants in the construction process there started to emerge a call for a different approach to the prevailing mode, as of late, to pile 'all risks' on the 'doing party'. In different parts of the world, notably those where adversarial attitudes, risk shifting and legalistic attitudes prevailed, the attempt to transform a construction contract into a more "collaborative" form had been implemented by "partnering" and "alliancing", with the draw-backs highlighted previously. Enlightened personalities, such as Sir Michael Latham and Sir Alan Muir Wood, had expressed their views and exhortations for a change being due.

The crunch came when, in that the commercial world of entrepreneurs, with such an acute eye for the financial aspects, they were faced by the sudden prospect of the Insurers declaring the tunnelling construction, with accent on urban areas, practically "uninsurable". It required the implementation worldwide, on a 'voluntary' basis (which had very little of voluntary, since the option was the "un-insurability"), of a Code of

Practice for Risk Management. In UK such need had been recognized earlier, in year 2002.

It is then perhaps another coincidence that the Institution of Civil Engineers (ICE) felt thereafter the need for innovation required and necessary in the industry, so as to prepare and issue in February 2006 a *different* form of its family of contracts, different in that it did not resemble what existed previously, but in the same mould (so as to result 'familiar'). It was in the author's view a courageous attempt to reverse the prevailing attitude of "off-loading" risks, by going back to its origins of a "fair contract", since the Actors appeared to no longer have regard for such a characteristic, for which ICE and the Engineer was famous.

Although ICE states that the form "*is intended to fulfil an identified need for such a contract to sit within the framework of the current family*", it is felt that this effort goes a long way to become a model for future "collaborative contracts" involving Employer, Designer, Engineer, Contractor, Specialist Supplier (e.g. TBM). It seems an ideal form for mechanized tunnelling in urban environment. Starting from modernizing the "target contracts" concept of old, but avoiding their draw-back of 'cost plus' image, it appears to have conjugated several needs and tasks, previously unavailable together:

- fair risk prevention, allocation and management;
- competitive tendering, even multistage, with prequalification;
- 'familiar' ICE language, format, definitions and clauses (as applicable);
- 'constructive' interaction from early stage of the various Actors;
- pro-active collaboration by sharing expertise and managing risks;
- "risk register" recognition of relevant responsibilities;
- 'advance warning' established procedures;
- certainty of job costs reimbursement;
- 'open book' administration and audit;
- incentive of fair share of 'gains/pains' for the benefit of all;
- minimisation of potential disputes, also with ADR alternative options;
- effective "team work", to reach common success goal, no antagonistic attitudes.

Lest it would give the impression of sympathetic reporting, the author prefers to let the ICE text speak largely for itself (from the ICE Guidance Notes) and provide the reader with the **overall concept**, though paraphrasing and synthesising, with own input where applicable.

Introduction

The ICE Conditions of Contract - Target Cost Version foster collaboration and pro-active team work of all parties, with the aim for all parties to optimise design and construction and reduce costs. For this, a more open type of control and management is required, which allows early and joint approach to identification and management of risks and opportunities. The Contract drafting kept in mind current procurement initiatives and recognized that contractors can add value to any project during the design stages, with reference to construction techniques and sequences as well as choice of materials.

Use of the Contract should promote

- Contractor's early involvement in project development, especially in:
  - understanding the Employer's requirements;
  - design and effective constructability;
  - construction processes and materials selection.
- Employer's and Designer(s) early involvement in:
  - construction techniques and methods;
  - sub-contractors selection and procurement;
  - site planning and operation as an industrial facility, considering specifically site roles and possible team integration.
- Designer incentivization by appropriate share arrangements.
- management and allocation of risks, and
- risk ownership by the party in the best position to control it, defined in a "risk register" accepted by all parties, leading to reduced risk provisions and no need for contingencies in the Contractor's Target Cost.

Principles

- Rates and prices are submitted by the tenderers, together with schedules for identifying:
  - opportunities for value engineering;
  - risk and risk ownership, and
  - the Fee.
- The Employer may adopt a one-stage or a two-stage tender process. One-stage leads to a final offer of a Target Cost, when tenders are submitted. The two-stage allows tendering an initial offer: a period of development and negotiation follows, to reach finally the Target Cost based on the previous schedules.
- The Employer, the Engineer and the Contractor should act as an integrated team. Thus the Employer benefits from the Contractor's "hands-on" experience, whilst the Contractor benefits from the close co-operation needed for value engineering and risk management.
- The Employer and the Contractor will share savings or overexpenditure in the final actual cost of the Works, excepting:
  - risks retained by the Employer;
  - risks involving costs, but not triggering a Target Cost change (e.g. strikes or extreme weather).
- The Contractor is paid the Total Cost, made up of the costs he properly incurs to construct the Works, plus a Fee. Rectification costs of any defective work (provided it does not fall within Disallowed Costs) are also part of Total Cost.
- A Target Cost lower than a 'standard tender' sum should be in the Employer's expectation. For the Contractor it should reflect the reduced uncertainty due to improved cash flow, incentive mechanism, agreed risk ownership and joint

approach to risk management. Both parties have a common aim and incentive to reduce the Total Cost.

- The Contractor's Fee is meant to recover overheads, profit, depreciation, insurances, financing costs, etc. Any resource not used in directly providing the Works is included in the Fee and not in the Total Cost.
- Total Cost items should be attributable to the Works and measurable through contemporaneous contract records, freely available on demand to and audited by the Engineer ('open book' transparency, but privacy safeguards).

Mutual benefits

- Cost outcome much clearer, due to an appropriate Target Cost, to the share arrangement and to recognized risk ownership.
- Incentive for Contractor to keep the Total Cost below the Target Cost, to trigger the Contractor's Share benefit.
- Contractor motivated to self-police, reduce costs and minimize defects.
- Overall project team size (and costs) possibly reduced, by sharing systems and processes.
- Design process to aim at compatible and practical constructability.
- Process for early identification, avoidance and management of risk to be operated jointly and effectively: consequent establishment of agreed procedure to manage changes, regardless of why required. Thus, avoiding nugatory time and costs for claim preparation, assessment and negotiation becomes the common aim, for the project benefit too.

The Contract makes achieving these benefits possible, by encouraging more open and collaborative control and management. Mechanisms are also in-built for the Contractor to reduce provisions for:

- cash flow – since payment is in line with costs (cash flow financing to be included in Fee);
- risk – through early and joint identification, ownership, mitigation and management.

Shared responsibilities

The Target Cost Version enables the Employer, the Designer, the Engineer and the Contractor to contribute in each other's traditional responsibility areas by

- Contractor input into:
  - Employer's requirements;
  - design constructability;
  - construction systems and materials;
  - early project development;
  - Employer's risks co-management.
- Employer, Designer and Engineer integrated input into:
  - construction techniques/methods;
  - procurement strategy and sub-contractors selection;

- site set-up and operation (forming an integrated Employer/Designer/Engineer and Contractor team will also reduce duplications and 'man-marking', thus resulting in further Target Cost savings);
- Contractor's risks management.

Incentives to reduce costs

Incentives for the Contractor to reduce costs are intrinsic in the share arrangement of savings or overexpenditure. Overhead and profit is part of the Fee and is based on Target Cost. Incentives to reduce construction costs for the whole procurement chain too could be introduced, by participating in the share arrangements.

Differences from other contracts (ICE and others)

A target cost contract requires the parties to approach project administration in a fundamentally different way to other forms of contract. In particular, the ICE version provides for and/or encourages

- collaborative working in a spirit of mutual trust;
- joint approach to risk management and improvements opportunities ('value engineering' in effective action);
- Target Cost value optimization, with associated variations provisions;
- open-book costs administration and audit;
- payment of the cost of performing the Works (as defined by Total Cost);
- sharing "gains/pains" relating to Total Cost, below and above Target Cost respectively;
- balanced approach to remedial works (correcting defects);
- greater confidence that final Total Cost will be within and/or below original Target Cost.

The paramount value-added operative aspect of this version, compared with previous 'target' forms, is to have managed to integrate 'competitive tendering' with 'collaborative alliancing'.

Risk management

For the Target Cost to be determined effectively and the Contract managed efficiently through to completion, a '*Risk Register*' must be established, which documents all foreseeable risks. It is essential that risk is owned and managed collaboratively to minimize cost, time and Target Cost adjustments. Though the Contract defines who carries which risks, the emphasis is to openly document and evaluate the potential effects of these on the project. The principles of shared risk management do not change the overall concepts. They merely promote early recognition of the issues and their possible consequences, to allow exploration of mitigation measures. Such opportunities can enable new ideas and work approach to be considered, which can benefit both parties.

Payment to the contractor

Payment to the Contractor is made on the basis of cost incurred plus the Fee stated in the Form of Tender. This is a significant departure from the admeasurement basis of the normal 'schedule of quantities and prices' contract, ICE version included.

For this to work efficiently, the system used for cost recording, control and verification should be jointly agreed to minimize duplication and respect 'open book' transparency. Monthly statements format and content should be agreed at the outset, together with auditing and procedures monitoring Total Cost vis-à-vis Target Cost.

Conclusion

The fundamental differences with the use of this form are:

- Contractor's profit is increased, in total and as a percentage, where the final contract cost (Total cost + the Fee) to the Employer is reduced;
- mutual interest in cost reduction can reduce duplication of effort and will encourage collaboration focused on increasing efficiency;
- transparency of costs incurred in delivering the Works – essential in order to drive down costs, also for subsequent works and projects;
- minimization of Contractor's provisions for risk – Employer should consider taking ownership of risk that Contractor is not in the best position to control.

The principles behind the Target Cost Version mean that the team interest lays in helping each other to minimize cost and waste, in whatever form they arise. *Any inefficiency should be regarded as a failure of the team as a whole.*

A.8 CONCLUSIONS

In the urban areas, the final 'stake-holders', i.e. the public, and the closely-knit environment do not tolerate anything but the 'as low as reasonably possible' (ALARP) level of risks and nuisance. From the risk scenario analyzed, it has become evident that all Actors have to co-operate together in order to obtain such results. The present attitude to 'off-load' all risks, onto the 'doing party', favoured by the one-on-one adversarial forms of contract presently used and the mind-set of some of the players, has resulted in risks reaching extreme levels, to become, if the trend is continued, nearly 'uninsurable', and in soaring of costs, in turn off-loaded onto the taxpayers. This cannot be a 'win-win' situation and the Insurers themselves have sounded for their part a serious alarm. To make the Actors 'collaborate' as a team, the forms of contract in use have proven hardly suitable: more enlightened administrations have started new ways (some with limits) *for all parties to implement jointly the Risk Management Plan* advocated by many, by the insurers and by this book. A new form of contract has been cast in early 2006 by ICE, the professional body of outstanding engineering tradition, which could meet most of what the public and professionals need and require.

The ICE Target Cost Version is not to be considered a 'panacea' solving all difficulties, but it constitutes a first giant step in the right direction of *"collaborative contracting" integrated with competitive tendering*. An added advantage, most suitable for urban tunnelling and metro construction, is that the embedded philosophy favours the establishment of long term relationship and trust between *all Parties*. The contract is not to be seen as a one-time deal, but can be the starting point for one development after the other, though preserving the competition umbrella. This could become even more important for *all the TBM manufacturers* (there are not many around the world), as their contribution and investment in R and D could find the recognition and 'contractual integration into the team' from the equally essential early stages. Thus the foundation is laid for a long-term collaboration among ALL Actors. Questions could be raised as to how the spirit of competition could be preserved, but the answer is in ensuring really competitive tendering and privacy ethics through the process. The results will be measured by the end-users and the taxpayers benefiting from a project at finally low cost.

The ICE form, being for national use of course, has peculiarities to cater for the UK scene, which will have to be modified and adapted to individual national needs. It is hoped it will be endorsed in due course by FIDIC, with adaptation for international and MDB-funded works. For many a nation – China for one, being the biggest urban (and country) infrastructure market of the 21st Century – not imbibed with the law-driven one-on-one adversarial contract, it could constitute the right balance of powers and teamwork more akin to their culture. For the Western world it could become the right antidote to the exponentially-rising level of risks, costs and disputes.

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Glossary

Advance The process of marching forward; sometimes it directly means the distance excavated during a given period of time (generally a shift or a day).
Advance rate, Advance speed Average speed of advancement of the tunnel; can be expressed in rings/day or meters/day.
Air cushion, Air bubble Applicable to Hydroshield only. A part of the excavation chamber, filled fully with compressed air, to provide the face-support pressure to the slurry.
Air lock (See Man Lock).
Anti-roll jack Jack used to contrast the rolling tendency of a TBM.
Annular Gap, Annular Space, Annular Void Space between the excavated ground contour and the lining extrados.
Articulated shield Shield made in two parts, which can be deviated from the longitudinal axis of the machine such that it is possible to better follow curved alignments with limited radius.
Articulation jacks Jacks making possible the articulation of a shield.
Back Chamber Applicable to Hydroshield only. The rear part of the working chamber where air and bentonite slurry are in contact.
Back Filling, Back Grouting The grouting of the Annular Gap.
Back-up Special train wagons containing the main equipment and devices necessary for the functioning of a TBM.
Belt Conveyor A moving belt system used to transport excavated muck from the excavation face to the outside of a tunnel.
Belt filter press Press used to separate solid materials from the liquid part of the muck by means of special belt-conveyors (See separation plant).
Bentonite Slurry A viscous mixture of bentonite and water.
Bleed, Bleeding Separation of water from a slurry.
Brushes, tail seal Special seal, made of steel brushes and mounted on the tail skin of a shield, that slides on the extrados of the segment lining as the machine advances.
Bulkhead The steel partition within a shielded TBM which divides the frontal pressurized zone (the plenum or excavation chamber) from the rest of the machine and thus also the tunnel.
Cake, Slurry Cake Fine slurry membrane formed on the excavation front, providing face stability.
Chamber, Excavation Chamber (See "Plenum").
Chimney A collapse of an underground excavation reaching the ground surface.

Claquage Hydrofracture-grouting.
Clogging A plug formed on the cutterhead by sticky ground.
Closed-Face Shield Any shield machine that can be operated in closed mode, i.e. with a face-support pressure.
Closed Mode A working modality of a machine: the pressure generated in the excavation chamber is higher than that inside the tunnel, providing a face-support pressure.
Collapse, Face Collapse Falling of ground from the tunnel roof or from the tunnel face.
Compressed Air Shield Shield machine operating with the compressed air to support the face. The compressed air could be either limited to the excavation chamber or can be applied to the whole tunnel zone.
Confinement The result of a mechanical or fluid pressure applied at the tunnel face, in order to control ground deformation and stability.
Conicity, Shield Conicity Difference between the cutterhead diameter and the tail-skin diameter.
Connectors: Longitudinal Bolts Longitudinal connection element between rings. It is also used as guide element to facilitate segment positioning in ring-assembling operation. **Dowel – Ring bolt.** Bolt used to connect segments of the same ring.
Control Cabin/Control Panel Cabin equipped with all instrumentation necessary for guiding the advancement of a TBM and for controlling the main operation parameters of the machine.
Copy Cutter Peripherical tool to be extended radially from the cutterhead when overcutting is required.
Counter K-Segment The segment with a longitudinal face in contact with the key segment.
Countermeasures Special measures to be applied to a given situation in order to reduce the corresponding risk level.
Crusher A device, normally installed in Slurry Shields, used to crush excavated boulders which could otherwise paralyze the mucking system.
Curved Bolts Curved steel elements used to connect segments.
Cutters/Cutting tools Tools installed on the cutterhead to penetrate into and remove the ground: picks, pins and disks are the most used tools and the choice among them is a function of the ground characteristics.
Cutterhead, (also Cutting Head, Cutting Wheel) Rotating part of a machine that supports the tools used to excavate the tunnel face through the combined action of thrust force and rotational torque.
Cutterhead displacement Possible displacement of the cutterhead axis with respect to the TBM axis, for better driving of a TBM.
Cutterhead opening ratio The ratio between the opening area and the total area of a cutterhead.
Cutterhead torque The rotating torque necessary to operate the cutterhead during excavation.
Cutterhead thrust The longitudinal force applied to the cutterhead for advancing the machine during excavation; in the case of a telescopic shield it is a part of the measured total thrust.
Cyclone, Hydrocyclone (See Hydrocyclone).

Disc/Roller Cutter Rotating excavation tool for rock excavation.

Double Shield Shield machine equipped with a front telescopic articulated shield and a rear shield provided with a lateral gripping system. The availability of two systems of longitudinal thrust jacks allows for simultaneous excavation and ring assembling, continuously.

Dowel Special type of plastic or steel connector.

Earth pressure Pressure created into the plenum of an EPBS by the accumulation of muck.

Earth Pressure Balanced Shield (EPBS)/Machine (EPBM) Shield machine where mechanical pressure is transferred from a bulkhead to the excavated material within the plenum, counter-balancing the earth pressure from the face during excavation. This face-support pressure helps to prevent or minimize heave or subsidence on the ground surface. A screw conveyor extracts in a controlled manner the material from the plenum, regulating thus the level of pressurization of the muck accumulated in the plenum itself.

Emergency Lock Device which is equipped with the basic and necessary first-aid measures for use in case of an accident involving workers operating inside the pressurized zone.

Emergency Tail Seal Expansive elements that can provide additional sealing of the gap between the shield tail and the extrados of the segment ring in emergency case.

Erector, segment erector Device present inside the TBM shield by which the segments of each new lining-ring are lifted and placed into their respective positions.

EPB, EPBM, EPBS, EPB Machine (See Earth Pressure Balance Machine).

Excavation chamber (See Plenum).

Extrados The external surface of a cylindrical structure.

Extruded concrete Forming of the final lining through pumping directly grout mixture into the annular gap between the ground and a special primary support structure.

Face, tunnel face, excavation face The surface separating the excavated tunnel from the ground to be excavated.

Face-support pressure Pressure applied to the tunnel face for stability reasons.

Filter cake (See "cake").

Filter Press A special filter unit utilized in a slurry treatment plant for the last stage of separation of fine elements.

Foaming Solution Mixture composed of water and surfactant to be injected with compressed air, creating the "foam".

Front-Shield The forward shield.

Full-Face-Boring Machine A machine capable of excavating the entire tunnel face at the same time.

Greenfield settlements Ground movements on the surface induced by tunnelling where no structures are present.

Gripper System capable of providing contrast against surrounding rock, necessary for applying the thrust force to advance the TBM. It consists of gripper cylinders/jacks and gripper shoes/pads.

Gripper Cylinders Jacks used to provide the gripping force/contrast. In a section transversal to the tunnel alignment, they can be radially directioned or they can also have a longitudinal component.

Gripper Pads/Shoes The part in direct contact with the surrounding rock, transferring the gripping force to the rock.

Groove Recess for housing gasket around the segments.

Grout, Grouting Injection or filling with mortar.

Guiding System – Guidance System System by which it is possible to maintain a TBM along a desired alignment. It generally includes a laser guidance system and the differential use of the main thrust jacks and/or the TBM shield/cutterhead articulation.

Hazard Event which can cause damages to persons and/or properties.

Head Opening Ratio (See Cutterhead Opening Ratio).

Hydraulic Mucking/Slurry Mucking System Transport of muck through the circulation of fluids in pipes, realized using pumps.

Hydrocyclone Separation unit used in slurry treatment plant mainly for the fine-sand separation, by centrifugal force.

Hydroshield Special type of slurry shield, equipped with an "air cushion"(see also Slurry Shield).

Hyperbaric chamber (See "Air Lock", "Man Lock").

Intrados Internal part of a cylindrical structure.

K-Segment, Key Segment Segment closing and blocking a ring: it is the last piece to be installed.

Learning curve The initial stretch of a tunnel, where the excavating crew learns the use of the machine.

Lining ring Structure used to line a TBM-excavated tunnel, consisting of pre-casted segments assembled in-situ at the end of each excavation cycle. Each ring is composed by a fixed number of segments, made of pre-cast concrete or cast iron.

Man Lock Device which allows the transfer of persons from a pressurized zone (chamber) to an atmospheric pressure zone and vice versa.

Main Drive, Main Ring Gear Gear connected to the cutterhead, supplying it with rotation torque coming from hydraulic or electric motors.

Main Seal Sealing system used to protect the main drive and the ring gear.

Materials Lock Device which allows the transfer of materials from a pressurized zone (chamber) to an atmospheric pressure zone and vice versa.

Mortar A mixture of cement, water, sand, and additives used normally to fill the annular void.

Muck Excavated materials to be removed.

Mucking Loading and transport of the excavated materials from the excavation zone to outside of the tunnel.

Mucking Bucket Mucking device used to collect and transport the excavated materials from the face to the primary conveyor.

Mucking Trains Trains used to transport muck from inside the tunnel to the outside.

Opening Ratio (See "Cutterhead Opening Ratio").

Open Mode Working modality of a machine: there is no pressure into the plenum, which can be not completely full of slurry and/or excavated material.

Overcut, overcutting Intentional overexcavation of ground outside the required diameter, usually necessary to assist the steering of the TBM. It is created by using copy cutters.

Overbreak Unintentional removal of ground outside the required tunnel excavation profile.

Overburden Ground above the tunnel crown till surface level, measured as "m high".
Overexcavation Excavation of ground outside the required diameter, usually necessary to assist the steering of the TBM or for minimizing the effects of squeezing ground. It can be intentional (for steering), or unintentional, causing overbreak.
Pad / Shoe Gripper part in direct contact with the contrast zone.
Particle size distribution curve Graphic illustration of the distribution of the grain sizes of a granular ground.
Penetration Rate Actual advance speed of excavation, generally expressed in mm/min.
Personnel Lock (See "Man Lock").
Pick A tooth-shaped cutting device for excavating soft ground.
Plenum (Also called "Chamber", "Excavation chamber", "Working chamber") The zone of a shield machine in contact with the tunnel face. This space must be pressurized when the excavation is done in 'closed mode'.
Pressure cells, sensors Sensor installed on the bulkhead of a Closed-Face Shield, that gauge and indicate to the TBM operator the fluctuation of the operating pressure.
Pressure jacks (See "thrust cylinders").
Rear Shield Back part of the shield: it has the role of protecting the zone of ring assembling. In double shielded machine, it is the shield with grippers, where rings are assembled.
Ring (See "lining ring").
Roll The tendency of a machine to rotate along its longitudinal axis.
Rotary Joint, Rotary Distributor Joint placed between the cutterhead and its support, supplying the cutterhead with all fluids necessary for its functioning.
Rotation Speed (rpm) Rotation speed of the cutterhead, expressed in revolution/min; in EPBM, the speed of rotating of the screw conveyor, too.
Rubber Tail Seal Rubber sealing system of the tail void.
Screw Conveyor Archimedean screw trepassing the EPBM bulkhead, allowing for controlled removal of excavated material under pressure from the chamber to 'free air' at the discharge gate.
Segment Pre-casted element; when assembled constitute the tunnel lining (ring).
Segment erector (See "erector").
Separation Plant (See " Slurry treatment plant").
Settlement, surface settlement Vertical ground movement above the tunnel.
Shield Support system composed by a cylindrical steel structure.
Shield Articulation Possible displacement of the shield axis with respect to the TBM axis, in order to obtain lateral overcut for the driving in soft ground.
Shielded TBM Tunnel Boring Machine equipped with a protective shield.
Slurry Viscous suspension of minerals (bentonite, clay, etc,) and/or polymers in water; circulating in feeding and discharging circuits of a slurry shield, it provides face-support pressure and facilitates muck removal.
Slurry cake (See "cake").
Slurry Discharging Pipe Pipe transporting mud and excavated material from excavation chamber front toward treatment plants.
Slurry Feeding Pipe Pipe trasporting fresh or recycled slurry from surface to excavation chamber.

Slurry Shield Shield machine providing face support through the pumping of mud into the excavation chamber and operating muck removal through pumping out the same mud but mixed with the excavated ground.

Slurry Treatment Plant Plant for the separation of the bentonite slurry from the excavated material, making possible the recycle of the bentonite.

Steering Actions on the TBM for following a predefined direction of driving.

Stickyness, Sticky behaviour Particular behaviour of the ground which may cause the clogging of the cutterhead, and even the blockage of the machine.

Straight Bolts Elements connecting segments and/or rings.

Straight Ring Ring whose front and rear faces are parallel.

Stroke Excavation cycle lenght (corresponding to the effective extention of the thrust jacks).

Subsidence Vertical movement of the ground surface, consequent to tunnel excavation; see also "settlements".

Tail Shield, Tail skin Back part of the shield; see also"rear shield".

Tail Seal Sealing system of the gap between the tailskin and the extrados of the lining rings.

Tail seal brushes A kind of tail seal using steel brushes; see also "brushes".

Tail void (See "annular void").

Taking point A point or hook for picking up a segment.

Tapered Ring Ring whose front and rear faces are not parallel, making it possible to excavate through curves along the tunnel alignment (See also "Universal Ring").

Target Target fixed to the machine, with the purpose of controlling the TBM position during excavation.

Telescopic Head Cutterhead capable of advancing some tens of cm independently from the shield body.

Thrust Cylinders/Rams/Jacks Jacks providing the thrust force necessary for advancement.

Thrust per Cutter Part of the thrust force transmitted to a single cutter.

Total Thrust Resulting force of the thrust cylinder forces acting on all segments of a ring.

Tunnel Boring Machine (TBM) Machines which can excavate ground or rock and in some cases directly build the final lining.

Tunnel face (See "Face").

Universal Ring Ring where front and rear faces are not parallel: this makes it possible to excavate through curves along the tunnel alignment by installing the key segment in different positions.

Vibrating screen A sieve on which the muck is separated by vibration, normally used in slurry treatment plants.

Wire-Brush Tail Seal Tail seal device composed by two or three wire brushes feeded by grease (See also "Tail seal").

Working Chamber (see "plenum").

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