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# Tunnel Engineering

Selected Topics

*Edited by Michael Sakellariou*





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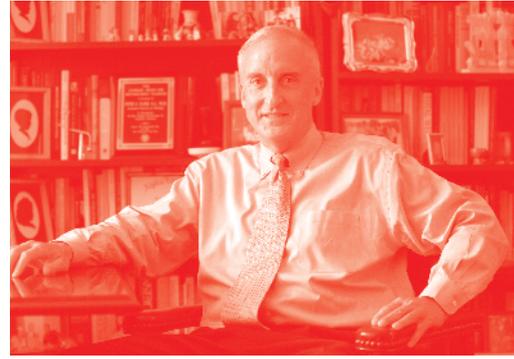
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Edited by Michael Sakellariou

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# Preface

In this volume, a selection of chapters covers a wide range of tunneling engineering topics. The scope was to present reviews of established methods and new approaches in construction practice and digital technology tools like building information modeling (BIM).

Site investigation methods and geological aspects of rock mass behavior to define the critical failure mechanisms are the subjects of two chapters in the first section of the volume. Analysis and design topics of tunnels are the subjects of the second section. Computational methods of analysis and an in-depth presentation of designing tunnels in the whole spectrum of shallow to deep tunnels are covered in three chapters. In the following section, new methods and challenges in tunnel construction are discussed. The first of this section examines tunnel–structure interaction challenges posed by transit-oriented development projects, which are a new topic in designing contemporary and future city and metro systems. The chapter covers in detail all the aspects of designing and analyzing the project through the use of BIM and monitoring techniques. Methods to improve the seismic response of tunnels and innovations of multigallery and multifloor tunnel sections to improve safety and reduce cost are the subjects of the next chapter in the section. The section ends with a chapter on immersed tunnels covering details of design and construction with an emphasis on the presentation of designing submerged floating tunnels. The last section is devoted to new trends in designing a tunnel based on digital construction strategies and advanced computational tools. The first chapter presents digital tools from BIM to artificial intelligence and discusses the major changes in the construction industry with an emphasis on tunneling. BIM is presented in great detail in the last chapter with an insight into future developments.

**Michael Sakellariou**  
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Section 1

# Geological Aspects of Tunnelling



# Engineering Geology and Tunnels

*Vassilis Marinos*

## Abstract

Currently, knowledge and understanding of the role of geological material and its implication in tunnel design is reinforced with advances in site investigation methods, the development of geotechnical classification systems and the consequent quantification of rock masses. However, the contribution of engineering geological information in tunnelling cannot be simply presented solely by a rock mass classification value. What is presented in this chapter is that the first step is not to start performing numerous calculations but to define the potential failure mechanisms. After defining the failure mechanism that is most critical, selection of the suitable design parameters is undertaken. This is then followed by the analysis and performance of the temporary support system based on a more realistic model. The specific failure mechanism is controlled and contained by the support system. A tunnel engineer must early assess all the critical engineering geological characteristics of the rock mass and the relevant mode of failure, for the specific factors of influence, and then decide either he or she will rely on a rock mass classification value to characterise all the site-specific conditions. Experiences from the tunnel behaviour of rock masses in different geological environments in Alpine mountain ridges are presented in this chapter.

**Keywords:** weak rocks, ground types, tunnel behaviour, tunnel design, tunnel behaviour chart, geotechnical classifications

## 1. Introduction

Currently, knowledge and understanding of the role of geological material and its implication in design is reinforced with advances in site investigation methods, the development of geotechnical classification systems and the consequent quantification of rock masses. Rock mass rating (RMR) [1] and Q [2] were developed to provide tunnel support estimates through a rating of rock masses. In addition, the advancement of the geotechnical software that is easier to use led to an increased requirement of data related to mass properties. This kind of data is needed as input for analysis in the numerical solutions for designing tunnels. The onset of numerical tools to handle rock-support interaction and the advancement of concepts related to ground reaction curve permitted issues to be managed well beyond the ultimate extent for application of different tunnel support classification systems such as the RMR and Q systems. Practice picked up from the early application of more modern numerical modelling recommended that there was great correspondence between the rules from these classifications and the displaying outcomes about reality when rock mass behaviour was generally simple; for example, the RMR system does not give good outcomes beyond the range of values between 30 and 70 under moderate stresses. Good results and realistic outcomes may well be produced where the

sliding and rotation of intact rock pieces essentially controlled the overall failure process, comparing to an encounter database on which the early classifications were built. Truly hundreds of kilometres of tunnels were effectively excavated on the sole premise of this application.

Solid appraisals of the strength and deformation characteristics of rock masses are required for nearly any procedure of investigation concerning an underground work. Subsequently, an approach for the estimation of rock mass properties from intact rock properties and joint characteristics is fundamental. The Hoek-Brown failure criterion [3–5] would be of no benefit in the event that it might not be promptly connected with engineering geological perceptions for the nature and fabric of the rock mass. Hoek [6] proposed a methodology for getting estimates of the strength of fractured rock masses based on an appraisal of the interlocking of rock pieces and the condition of the surfaces between these pieces. For such an evaluation, the Geological Strength Index (GSI) was presented. The GSI has been established over a long period [3, 5, 7–13] to meet the desires of practitioners and cases that were not at first realised. The application of the mass properties from the GSI values basically accepts that the rock mass behaves isotropically. It is not factious where there is a clear anisotropic behaviour, e.g. clearly characterised favoured failure surface or discontinuities. The appropriate use of rock mass characterisation systems, notably the GSI (for details, see [14, 24]), allowed the quantification of difficult ground for the evaluation of the geotechnical properties and the selection of the design parameters. An extension of the original GSI application charts for heterogeneous and structurally complex rock masses, such as flysch, was initially introduced by Marinou and Hoek [10] and recently updated and extended by Marinou [11]. Specific GSI charts for molassic formations [12], ophiolites [13], gneiss in its disturbed form [14, 15] and particular cases of limestones [15] and under particularly difficult geological conditions have been developed from experience gained during excavation of 62 tunnels as part of the Egnatia project in Northern Greece, along Alpine mountain belts.

In expansion to the GSI values, it is additionally fundamental to consider the choice of the ‘intact rock’ properties  $\sigma_{ci}$  and  $m_i$  for these rock masses with different mineral composition. The fundamental inputs of the Hoek-Brown failure criterion are assessments or measurements of the uniaxial compressive strength ( $\sigma_{ci}$ ) and the material constant ( $m_i$ ) related to the frictional properties of the rock and of the GSI. Furthermore, to assess the deformation modulus of the rock mass ( $E_m$ ), Hoek and Diederichs [16] proposed a formula based on the values of the intact rock deformation modulus ( $E_i$ ) or the modulus ratio (MR).

The role of the ground characteristics and its effect in tunnel design, strengthened with progresses in site investigation techniques, cannot be exclusively based on the advancement of geotechnical classification frameworks and the following quantification of rock masses. Temporary support measures for rock masses with equivalent classification values can be diverse. The engineering geology appraisal displayed in this chapter cannot bypass the geological and/or in situ characteristics managing or affecting the tunnel behaviour compared with a regular classification that might miss the specifics and particularities of and around a tunnel segment. The likely ground types must be assessed, and after that, combined with the components of the tunnel geometry, the primary in situ stresses and the groundwater regime, the possible failure modes must be considered. These classified behaviour modes, followed by the appropriate mechanical properties that are required for sound tunnel design, are the premise for the numerical design of the appropriate primary support measures to achieve stable tunnel conditions.

There has been a serious effort to develop guidelines and procedures for tunnel design in which the observation of rock mass behaviour is incorporated in

the determination of excavation and support classes [17–21]. The first step of this methodology involves the definition of the possible rock mass type, the second step involves the evaluation of the rock mass behaviour in tunnelling, the third step suggests the type of tunnel excavation-support system, and the final step is the definition of tunnel length with equal support requirements and the appraisal of time and cost for incorporation in the tender documents.

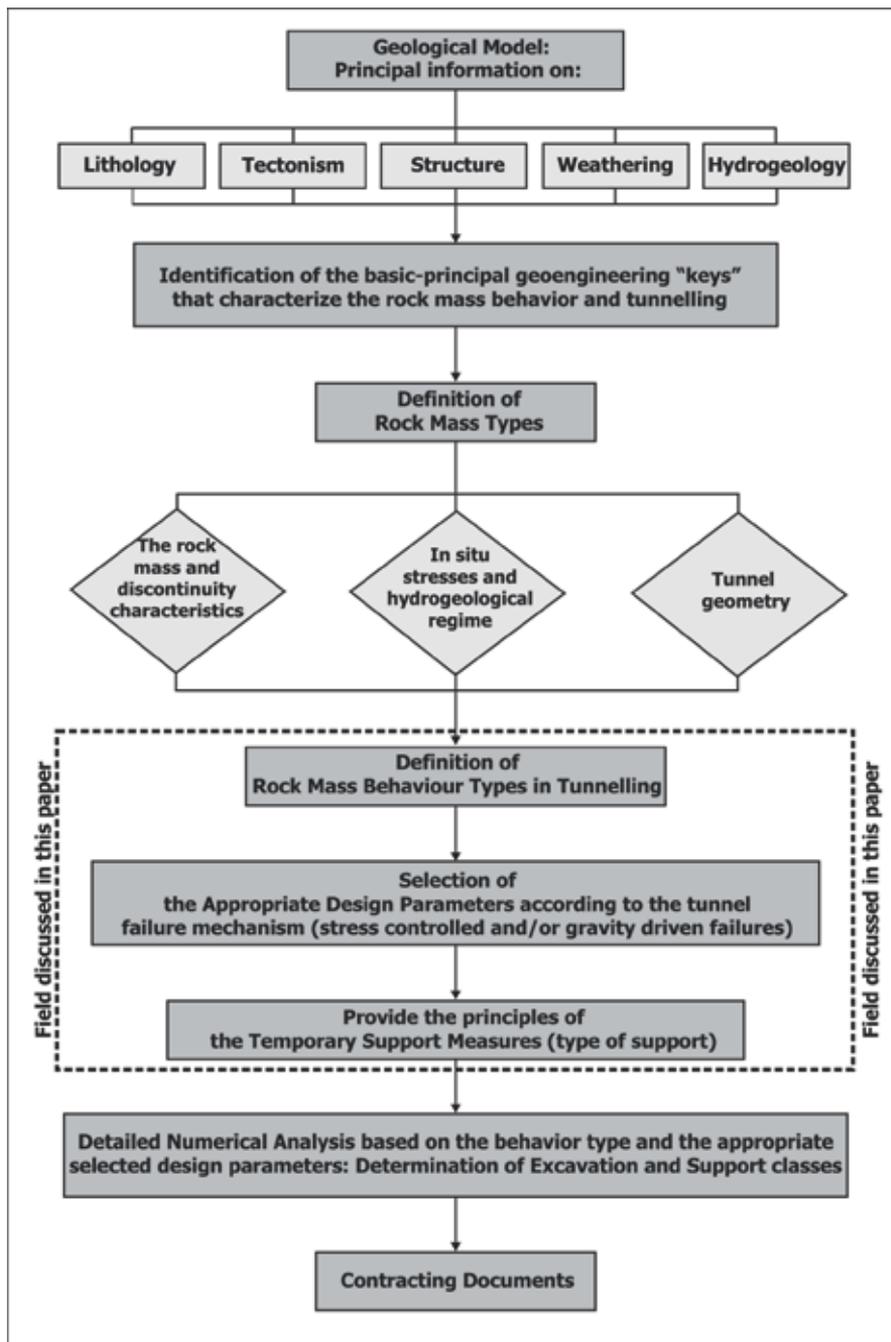
The design methodology discussed here incorporates the assessment of the tunnel behaviour type in the selection of design parameters and the definition of temporary support measures. A flowchart in **Figure 1**, based on Schubert [20] with modifications, presents this design methodology. As shown in this flowchart, the fundamental link between the rock mass model and the excavation and support classes is the definition of the tunnel behaviour type.

Hence, the contribution of engineering geological information for safe and economical tunnelling cannot be simply presented solely by a rock mass classification value (e.g. RMR, Q, GSI, or others). A classification rating, if used, must be accompanied by an understanding of the actual rock mass behaviour in tunnelling [22]. The tunnel behaviour may vary from one rock mass to another, indeed on the off chance that they have the same classification rating within the same stress field and the same groundwater conditions. An illustration of two different ground types with the same classification value but distinctive tunnel failure mode is displayed in **Figure 2** [22]. The two frameworks in **Figure 2** outline that the choice of the immediate support measures cannot be based solely on a classification rating (either GSI or RMR or Q) but that it moreover requires an understanding of the tunnel failure type.

Attention, therefore, should be given to the evaluation of the failure mechanism that ‘fits’ the ground type after its excavation. For instance, it is clear that in the process of design, the structure of the rock mass must be considered together with the classification index. Taking after the assessment of the failure mechanism, one can be more certain either in utilising the rating of the associated classification value or in deciding the particular geological or in situ characteristics—‘keys’ that oversee the tunnel behaviour of the ground type. This procedure assists the designer in the analysis of tunnel behaviour and the selection of support measures and in the establishment of the contractual documents and guidelines for the construction.

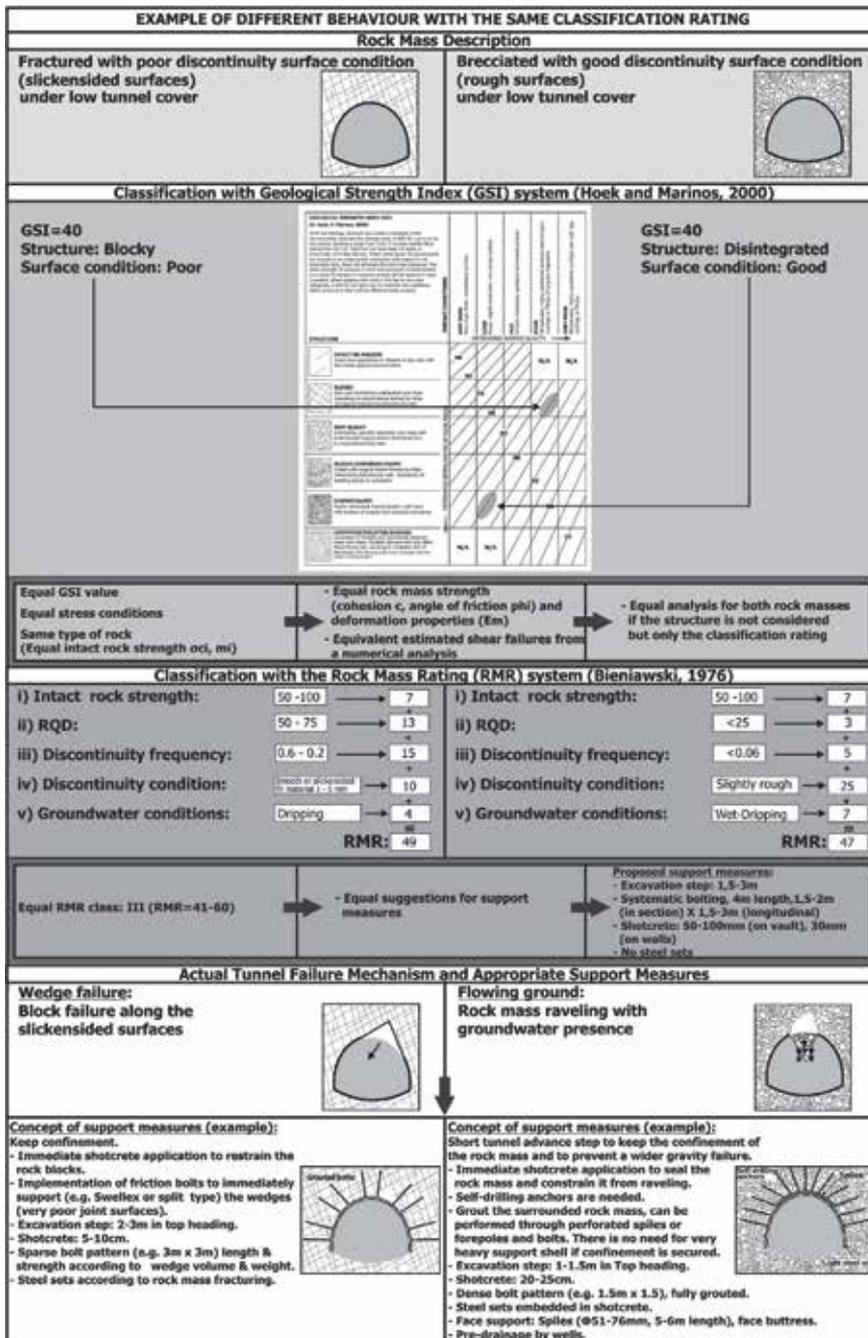
After the appraisal of the tunnel failure mode, the appropriate numerical modelling can be performed, the conditions can be more soundly analysed, and the principles of tunnel support can be more precisely considered. The appropriate and critical design parameters can also be chosen in connection with the standards of the failure mode. If the behaviour of the rock mass can be considered isotropic and governed by induced stress, the tunnel engineer must be focused on the rock mass parameters (e.g. GSI in the Hoek-Brown transfer equations relating intact and rock mass properties with respect to the GSI) [3, 4]. On the other hand, in the event that the main failure mode is gravity-induced instability, the practitioner must focus on parameters related to the joints. In the event that the rock mass is weak but moreover anisotropic (e.g. due to schistosity or well-defined bedding planes), both the rock mass parameters and the persistent discontinuity properties must be considered [23]. Being that most tunnel designs presently incorporate numerical analysis, the issue is whether to utilise rock mass parameters (such as shear strength of the rock mass,  $c_{\text{mass}}$ ,  $\phi_{\text{mass}}$  and  $E_{\text{mass}}$ ) when the rock mass behaves isotropically or to incorporate the joint parameters (orientation, distribution, persistence, shear strength  $c_{\text{joint}}$  and  $\phi_{\text{joint}}$ ) when the behaviour is controlled by the discontinuities or impacted by the resulting anisotropy.

Recent research regarding weak rock masses and their engineering geological behaviour, as well as the experience gained by the recent tunnelling projects in the Greek mountains, offered sound and adequate information for the investigation of



**Figure 1.** Flowchart of the design procedure for tunnelling using conventional drill and blast excavation from the geological model to the definition of rock ground types and from the appreciation of the tunnel behaviour modes to the numerical design. Based on Schubert [21] and modified by the author [22].

the impact of these conditions on the behaviour of the geological material, as well as on the design and construction methods. To make substantial use of the experience accumulated from the design and construction of these tunnels and to correlate this material, a database was built, i.e. ‘Tunnel Data Examination System’ (TIAS), which was outlined and created for 62 bored tunnels within the Greek region along



**Figure 2.** Example of two equally rated rock masses with the GSI or RMR system but with completely different behaviours in tunnelling [22]. The selection of the temporary support measures should not be based only on the classification ratings but also on the understanding of the tunnel failure mechanism, which is greatly dependent on the rock mass structure.

the Egnatia Highway [25] mainly spanned in Alpine mountain ridges under difficult geological conditions in weak rock masses, excavated with conventional methods, in the concept of top heading and bench excavation. This database is built to 'relate' all available data through all the phases of a tunnel project and premises deep knowledge from the geological and geotechnical investigation to the final design

and construction. The data processed by TIAS came from a variety of sources such as geological mapping, boreholes, laboratory and in situ testing, geotechnical classifications, engineering geological behaviour, groundwater, design parameters, information concerning immediate support measures, construction records and cost. The scope of the system is to provide a tool for the evaluation of anticipated and encountered geological and geotechnical conditions, the evaluation of geotechnical classification and design methods and the relations regarding rock mass conditions and behaviour and immediate support methods and types.

The variety of geological formations under different in situ stress conditions, not only in both mildly and heavily tectonised rock masses but also in altered and/or weathered rock masses, provided a significant amount of information regarding the engineering geological behaviour of several rock mass types. The general geological and engineering geological characteristics and the behaviour in tunnelling of specific rock masses, such as heterogeneous rock masses of flysch and molassic formations (tectonically undisturbed heterogeneous sediments) as well as sound, disturbed and altered ophiolites, are briefly presented in the next paragraphs as examples.

## **2. Tunnel behaviour appraisal**

Engineers can analyse reinforced concrete or steel structures utilising certain checks for a particularly predefined failure mechanism. Particularly, analysis is performed against the bending moment, axial force, shear, penetration and deflection (serviceability limit state). In the case of tunnelling, there is no particular methodology to check against a predefined failure mode.

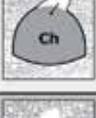
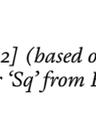
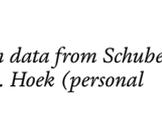
It is pointed out that the primary step is not to begin performing various calculations but to characterise the potential tunnel behaviour modes. After the evaluation of the ground behaviour in tunnelling, the analysis of the temporary support system can be utilised, in two stages: the choice of the appropriate support elements and their detailed analysis. The selection of support measures should be established equally on experience and geotechnical data and on the analytical solutions but must be confirmed or re-evaluated during construction, supported by the monitoring of the tunnel.

Rock mass behaviour evaluation in tunnelling and its relationship with the design process have been significantly researched. Goricki et al. [18], Schubert [20], Potsch et al. [26] and Poschl and Kleberger [19] studied rock mass behaviour from the design and construction experiences of Alpine tunnels and Palmstrom and Stille [27] from other tunnels.

### **2.1 Tunnel behaviour types**

The term ‘failure mechanism-behaviour type’, as alluded here, includes all the components that endanger the tunnel segment when the ground has not yet been supported after excavation.

This paragraph presents the tunnel failure modes as they have been designated by Terzaghi [28] and Schubert [21] and also suggested by the author from the tunnel experience of 62 designed and constructed tunnels along Egnatia Motorway and from other cases in Greece. The tunnel failure modes, apart from stable (St) conditions, are separated into gravity-driven failures (wedge and chimney-type failures and ravelling and ravelling ground) and stress-driven failures (failures, squeezing and swelling, anisotropic deformations and brittle failures). The limits and ranges where each behaviour type is connected are briefly depicted and

TUNNEL BEHAVIOUR TYPES			
St	<b>Stable ground:</b> Stable tunnel section with local gravity failures. Rock mass is compact with limited and isolated discontinuities		
Br	<b>Brittle failures:</b> Brittle failure or rock bursting at great depths		
Wg	<b>Wedge failure:</b> Wedge sliding or gravity driven failures. Insignificant strains. The rock mass is blocky to very blocky, blocks can fall or slide. The stability is controlled by the geometrical and mechanical characteristics of the discontinuities. The ratio of rock mass strength to the in situ stress ( $\sigma_{cm}/p_o$ ) is high (>0.6-0.7) and there are very small strains (<1%)		
Ch	<b>Chimney type failure:</b> Rock mass is highly fractured, maintaining most of the time its structure (or at least that of the surrounded rock mass). Rock mass does not have good interlocking (open structure) and in combination with low confinement (lateral stress) can tend to block falls which develop to larger overbreaks of chimney type. The overbreaks may be stopped and "bridged" by better quality rock masses, depending on the in situ conditions. This type may be applied also in cases of brecciated and disintegrated rock mass in ground with high confinement (high lateral stress)		
Rv	<b>Ravelling ground:</b> The rock mass is brecciated and disintegrated or foliated with practically zero cohesion and depending on the intact rock interlocking (Rv1 case: without infilling) and possible secondary holed geometrical (Rv2 case: with infilling, e.g. clay), rock mass can generate immediate rock mass ravelling in face and tunnel perimeter. The difference with Ch type lies in the block size, which is very small here, the self support timing, which is very limited here and the failure extension, where it is unrestricted due to the lack of better rock mass quality in the surrounding zone		
FI	<b>Flowing ground:</b> The rock mass is disintegrated with practically zero cohesion and intense groundwater presence along the discontinuities. Rock fragments flow with water inside the tunnel		
Sh	<b>Shear failure:</b> Minor to medium strains, with the development of shear failures close to the perimeter around the tunnel. Rock mass is characterized by low strength intact rocks ( $\sigma_c < 15 \text{ MPa}$ ) while the rock mass structure reduces the overall rock mass strength. Strains develop either at a small to medium tunnel cover (around 50-70m) in case of poor sheared rock masses, or in larger cover in case of better quality rock masses. The ratio of rock mass strength to the in situ stress ( $\sigma_{cm}/p_o$ ) is low ( $0.3 < \sigma_{cm}/p_o < 0.45$ ) and strains are measured or expected to be medium (1-2.5 %)		
Sq	<b>Squeezing ground:</b> Large strains, due to overstraining with the development of shear failures in an extended zone around the tunnel. Rock mass consists of low strength intact rocks while the rock mass structure reduces the overall rock mass strength. The ratio of rock mass strength to the in situ stress ( $\sigma_{cm}/p_o$ ) is very low ( $\sigma_{cm}/p_o < 0.3$ ) and strains are measured or expected to be >2.5%, and they can be also take place at the face		
Sw	<b>Swelling ground:</b> Rock mass contains a significant amount of swelling minerals (montmorillonite, smectite, anhydrite) which swell and deform in the presence of groundwater. Swelling often occurs in the tunnel floor when the support ring is not fully closed		
San	<b>Anisotropic strains:</b> The rock mass is stratified or schistose or consists of specific weak zones and develops increased strain characteristics along a direction defined by the schistosity.		

**Figure 3.** Brief descriptions and schematic presentations of tunnel behaviour types [22] (based on data from Schubert [21], Terzaghi [28] and from the author). Photos from the author except for 'Sq' from E. Hoek (personal communication) and for 'San' from Seingre [29].

appeared in **Figure 3**. The failure modes are assembled based on the examination of tens of rock mass types, their rock mass and joint quality properties and their actual behaviour below different stress conditions (from 30 m to 500 m overburden).

**Stress-driven failures:** The advancement of critical strains around a tunnel is characterised by the ratio of  $\sigma_{cm}/p_o$  [30]. Specifically, when  $\sigma_{cm}/p_o$  is between 0.3 and 0.6, shear failures can develop in a shallow zone around the tunnel perimeter (Sh failure mode). Such cases include rock masses with poor to very poor fabric and low intact rock strength (< 10–15 MPa) under medium overburden or with

more competent structure and low intact rock strength below high tunnel cover. Squeezing conditions (Sq failure mode) with severe tunnel strains can be induced when  $\sigma_{cm}/p_o < 0.3$ .

Gravity-driven failures: They are generally differentiated with relation to the rock mass fabric (original conditions and tectonic deformation) and to the conditions of being kept in confinement or not. These gravity-controlled failures occur in rock masses that are clearly characterised by the joints. When the rock masses are just excavated, wedges may fall or slide, depending on the tunnel geometry, the orientation and the shear strength characteristics of the discontinuity planes. Wedge (Wg), chimney (Ch) or ravelling (Rv) failure types can take place in rock masses with poor interlocking of rock blocks due to fracturing degree and/or low confinement. The rock mass cannot arch after the falling, and the crown failure may be significant and irregular. The volume and recurrence of these sorts of tunnel behaviour depend on the structure of the rock mass ('blocky-disturbed' and 'disintegrated'), its relaxation ('open structure') and the tunnel cover/lateral confinement conditions. With an increase in the depth of the tunnel, the rock mass quality is generally improved, and the confinement pressure 'tightens' the structure of the mass.

Of course, there are cases where both stress and gravity-driven failures can be met in a rock mass. In such cases, particular consideration ought to be given to the principal failure mode for the choice of suitable support measures.

A tunnel behaviour chart (TBC) [22], illustrated in **Figure 4**, has been proposed for assessing the rock mass behaviour in tunnelling and covers a wide range of rock mass conditions. This assessment is based on the structure of the rock mass, the strength of the intact rock and the overburden depth.

This classification frame, the TBC, joins the rock mass characteristics straightforwardly with the design and the tunnel support standards and covers a wide extent of conditions. The TBC could be a classification for the estimation of tunnel behaviour and requires three parameters: the rock mass structure, the overburden (H) and the intact strength of the rock ( $\sigma_{ci}$ ). This is an integrated classification based on the TIAS database and the data from the design and construction of 62 tunnels in Greece [25]. The purpose of this chart is to foresee the basic failure modes of several rock mass qualities and conditions. The cases that were investigated elaborated intact rock strengths up to 100 MPa and depths not more than 500 m, while many tunnels were less than 300 m deep. It is noted that the values of the uniaxial compressive strength of the intact rock ( $\sigma_{ci}$ ) and the overburden thickness (H) utilised within the chart are reasonable trends but should only be considered as indicative.

This chart can be applied in a wide range of geological and geotechnical conditions, since numerous geological formations with various tectonic, weathering and alteration intensity, commonly found worldwide, have been excavated and effectively supported, under a large range of tunnel covers (up to 500 m). The chart does not refer to very high overburden (e.g. many hundreds of m or > 1000 m) and very large intact rock strengths, where brittle failures (spalling or rock burst) can be developed. Hence, TBC can be really useful in any mountain formations in a tunnel excavated with the conventional principles within this wide application range.

The rock mass structure is an essential parameter to appraise its prompt reaction in underground opening in the TBC chart. From the structure of the rock mass, one can 'read' the tectonic disturbance, the blockiness of the mass, the probable size of blocks, the shape of rock elements (massive, blocky, foliated or sheared) or the ability of the rock blocks to rotate. Rock mass fabrics were categorised after the GSI system [8].

For gravity-controlled failures, the tunnel depth impacts the degree of a failure since the degree of interlocking between the rock blocks changes and the confinement

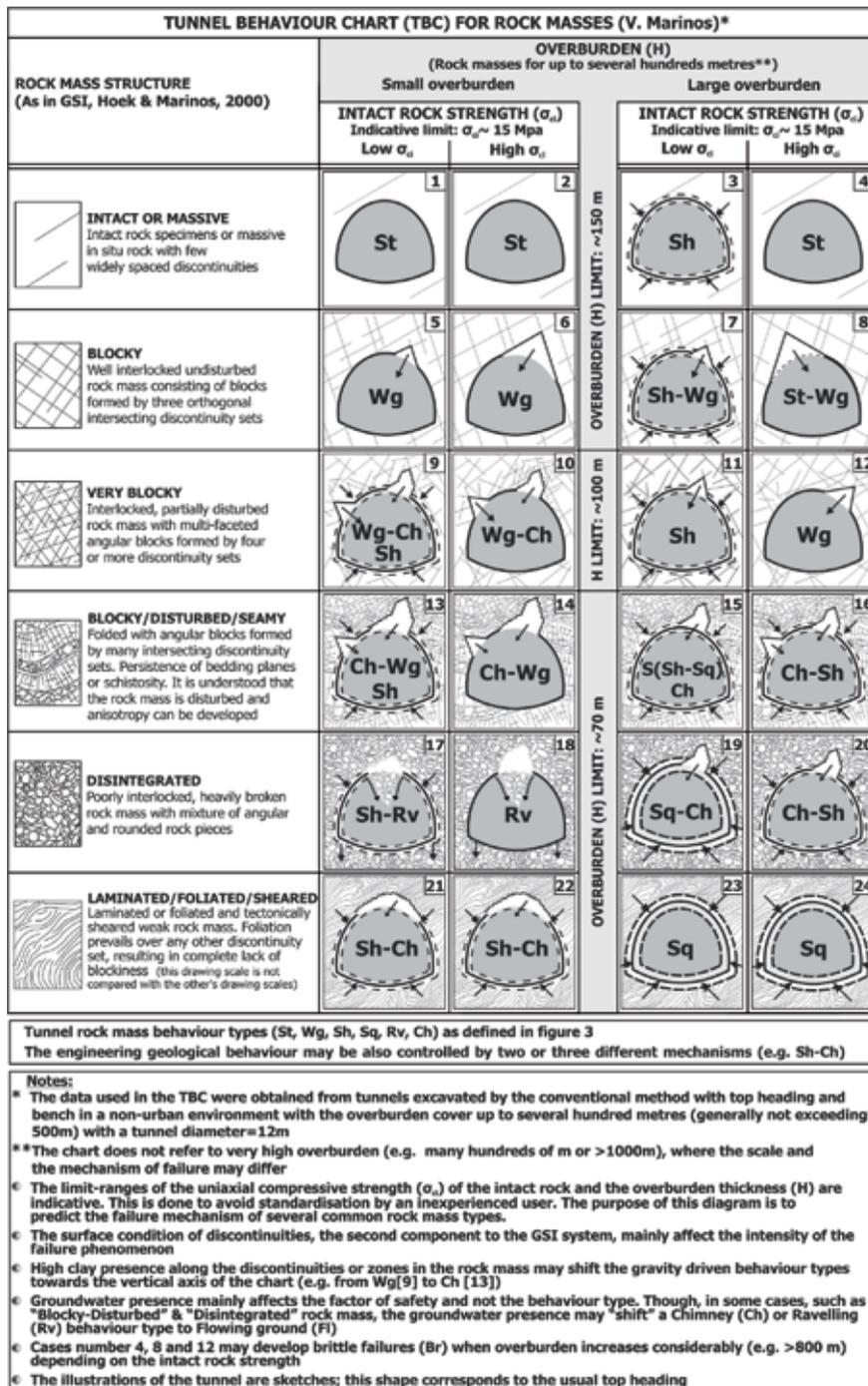


Figure 4. Tunnel behaviour chart: an assessment of rock mass behaviour in tunnelling [22].

stress varies with depth. For instance, the rock mass may ravel (Rv) near the ground surface, but under higher overburden, a chimney-type (Ch) failure may be developed.

As far as the stress-driven modes are concerned, tunnel cover H characterises when shear failures and deformations are formed. These limits are appraised in the following manner: 150 m for competent structure (intact and blocky-seamy undisturbed),

100 m for very blocky structure and 70 m for the very poor fabric (seamy-disturbed, disintegrated or laminated-sheared). These values are basically evaluated by back analysis and by the calculated values of the ratio  $\sigma_{cm}/p_o$ , with  $p_o$ , the in situ stress, considered isotropic. For example, in the event that  $\sigma_{cm}/p_o < 0.3$ , squeezing conditions are likely; in case  $0.3 < \sigma_{cm}/p_o < 0.6$ , minor to medium strains may happen; and on the off chance that  $\sigma_{cm}/p_o > 0.6$ , minor or no deformations are expected.

The limit of intact rock strength ( $\sigma_{ci}$ ), i.e. 'low' vs. 'high', considered to characterise the rock mass behaviour in tunnelling, in the TBC chart, is based on the value when shear failures and deformations initiate. This limit is assessed at 15 MPa. The  $\sigma_{ci}$  values that were analysed in the design of the investigated tunnels are extended between 5 and 100 MPa.

### **3. Examples of engineering geological appreciation and behaviour in tunnelling of particular rock masses**

The general geological and engineering geological characteristics and the behaviour in tunnelling of specific rock masses, such as heterogeneous rock masses of flysch (tectonically disturbed heterogeneous formation) and molassic formations (non-tectonically disturbed heterogeneous sediments) as well as sound, disturbed and altered ophiolites, are briefly presented in the next paragraphs as examples. For more details on each of the specific rock types, their engineering geological characteristics, their specific GSI characterisation and their tunnel behaviour, the interested reader is referred to the original publications presenting the individual charts [9–13, 15, 22, 23, 30, 31].

#### **3.1 Flysch formations: tectonically disturbed heterogeneous rock masses**

Due to the generally poor characteristics and uncertainties with respect to its geotechnical characterisation, flysch frequently causes problems or challenges towards the design and construction of engineering projects. Flysch is composed of variable alternations of clastic sediments that are related with orogenesis, since it closes the cycle of sedimentation prior to the paroxysm folding process. It is characterised basically by rhythmic alternations of sandstone and pelitic layers (siltstones and silty or clayey shales). The thickness of the sandstone or siltstone beds ranges from centimetres to metres. Conglomerate beds may also be included. Heavy folding and highly shearing with various overthrusts characterize the environment in areas of flysch formations. The main thrust movement is associated with smaller satellite reverse faults within the thrust body. The overall rock mass is profoundly heterogeneous and anisotropic and moreover may be influenced by extensional faulting creating mylonites. The structural deformation due to tectonism radically debases the quality of the rock mass. Hence, flysch rock mass types are associated with undisturbed, fractured, heavily sheared or even chaotic structures. Such flysch qualities are classified into 11 rock mass types (I to XI) [11] according to the siltstone-sandstone proportion and their tectonic disturbance (**Figure 5**).

The design of tunnels in poor rock masses such as folded and sheared flysch presents a major challenge to geologists and engineers. The complex structure of these materials, coming about from their depositional and structural history, implies that they cannot effectively be classified in terms of the broadly used characterisation systems. The range of geological conditions under varied in situ stresses, in both mild and heavy tectonism investigated here, offered valuable data with respect to the engineering geological conditions and geotechnical behaviour of several flysch rock mass types.

ROCK MASS TYPE	STRUCTURE	ROCK MASS TYPE	STRUCTURE	ROCK MASS TYPE	STRUCTURE
<b>TYPE I.</b> Undisturbed, with thick to medium thickness sandstone beds with sporadic thin films of siltstone.		<b>TYPE II.</b> Undisturbed massive siltstone with sporadic thin interlayers of sandstone.		<b>TYPE III.</b> Moderately disturbed sandstone with thin films of interlayered siltstone.	
<b>TYPE IV.</b> Moderately disturbed rockmass with sandstone and siltstone interbedded.		<b>TYPE V.</b> Moderately disturbed siltstones with sandstone interbeds.		<b>TYPE VI.</b> Moderately disturbed siltstones with sparse sandstone interbeds.	
<b>TYPE VII.</b> Strongly disturbed, folded rockmass that retains its structure, with sandstone and siltstone in similar weathers.		<b>TYPE VIII.</b> Strongly disturbed, folded rockmass, with siltstones and sandstone interbeds. The structure is receded and deformation - shearing is not strong.		<b>TYPE IX.</b> Disintegrated rockmass.	
<b>TYPE X.</b> Intensely deformed intensely foliated siltstone or clay shale with 'broken' and deformed sandstone specks forming an almost chaotic structure.		<b>TYPE XI.</b> Intensely strongly sheared siltstone or clay shale forming a chaotic structure with pockets of clay.			

**Figure 5.** Rock mass types in tectonically disturbed heterogeneous formations such as flysch [11].

A classification of flysch rock masses depending on their geotechnical behaviour (strain due to oversteering, overbreaks or wedge failure, 'chimney' type failure, ravelling and their corresponding scale) is displayed from now on. Depending on its type, flysch can show a range of behaviours: be stable even under significant overburden, display wedge sliding and more extensive chimney type-crown failures, or show large deformations even under low to medium overburden.

In a general sense, the behaviour of flysch arrangements amid tunnelling depends on three major parameters: (i) the structure, (ii) the intact strength of the governing rock type and (iii) the depth of the tunnel. The anticipated behaviour types (stable, wedge failure, chimney type failure, ravelling ground, shear failures and squeezing ground) can be outlined within the tunnel behaviour chart [22]. A detailed introduction of the range of geotechnical behaviours in tunnelling for each flysch rock mass type (I–IX), which is based on engineering geological characteristics, is displayed in **Figure 6**.

The rock mass is frequently taken as a 'mean isotropic geomaterial', in the case that rock mass properties are quantified through a classification system. This presumption is normally accepted in conditions of a uniformly jointed, highly tectonised rock mass without persistent joints of certain unfavourable orientation. This condition can be quite true for types VII to IX. Where bedded rock masses are involved, at a scale of the tunnel segment, the engineering geological behaviour during tunnel construction is controlled by the properties of the bedding planes. This case may be applied to the flysch rock mass type III to VI.

A reliable first estimate of potential problems of tunnel strain can be given by the ratio of the rock mass strength to the in situ stress  $\sigma_{cm}/p_0$  [30]. This is usually followed by a detailed numerical analysis of the tunnel's response to sequential excavation and support stages. Minor squeezing (1–2.5%) can be developed in the very poor flysch rock mass types X and XI from 50 to 100 m tunnel cover, while severe (2.5–5.0%) to very severe squeezing (5–10%) can be developed from 100 m to 200 m cover. Undisturbed rock mass types of sandstone or conglomerate (types I and III) do not exhibit significant deformations under 500 m.

Regarding the rheological characteristics of flysch formations, the creep potential of sandstone formations is considered to be negligible. However, in the case of tunnel excavation in siltstone or shale formations, especially under high overburden, time-dependent displacements or loads may be developed.

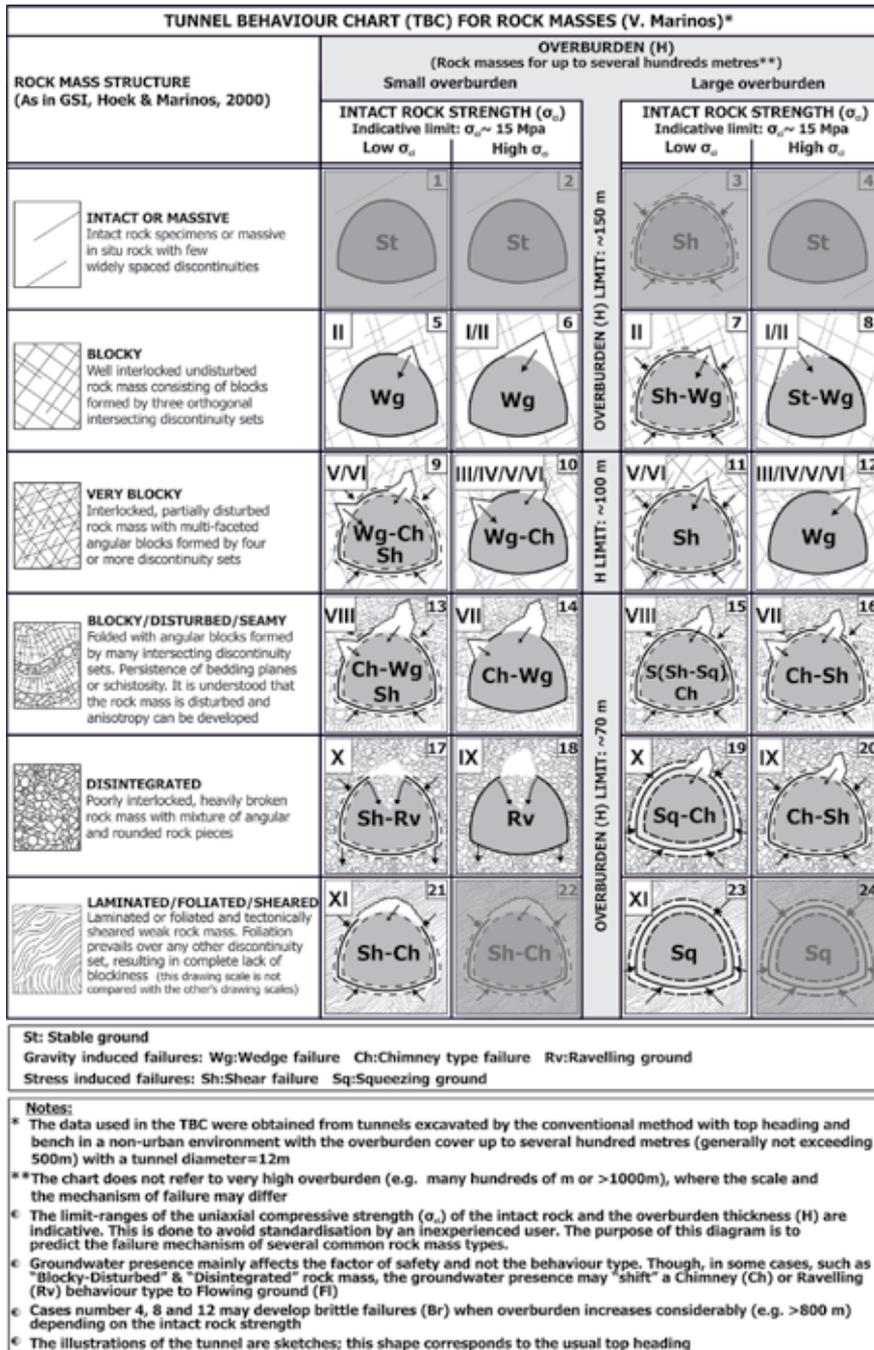


Figure 6. Tunnel behaviour chart with projections of the principal failure mechanisms for the rock mass types of flysch (I–XI) [31].

The influence of groundwater on the rock mass behaviour in tunnelling is very important and has to be taken into great consideration in the estimation of potential tunnelling problems. The most basic impact of groundwater is on the mechanical properties of the intact rock components, particularly on shales and siltstones that are susceptible to changes in moisture content.

The evaluation of tunnel behaviour and the conceptual assessment of the support measures must be also based on detailed ground characterisation. This detailed characterisation cannot bypass the geological and/or in situ characteristics managing or affecting the tunnel behaviour compared with a standardised classification. This classification, named ‘Ground Characterization, Behaviour and Support for Tunnels’ [22], urge the practitioner to assess the information in detail, to appraise the tunnel behaviour and to select the appropriate design parameters and the suitable support measures. An illustration of this characterisation in tunnelling through tectonically disturbed flysch type is displayed in **Figure 7** [31].

GROUND CHARACTERIZATION, BEHAVIOUR AND SUPPORT FOR TUNNELS (1/2) (V. Marinos, 2012)	
Location:	Malyssou area (Northern Greece)
Classification phase (primary, evaluation, construction):	Final design phase
Date:	
<b>I. GEOLOGICAL CONDITIONS</b>	
<b>a) Lithology</b>	
<ul style="list-style-type: none"> <li><input type="checkbox"/> Geotectonic unit:</li> <li><input type="checkbox"/> General formation to which it belongs (e.g. Flysch):</li> <li><input type="checkbox"/> Rock mass name:</li> </ul>	<ul style="list-style-type: none"> <li><input type="checkbox"/> "Pindos" geotectonic unit-zone</li> <li><input type="checkbox"/> Flysch</li> <li>Type X Sandstone - claystone oolitic structure with sandstone fragments</li> <li>Claystone oolitic structure with sandstone fragments</li> <li><small>(Note: Information concerns significant alteration of the rock mass, intact rock or surface weathering, presence of folded - clayey portions and bedding thickness if it is stratified)</small></li> </ul>
<b>b) Tectonism</b>	
<ul style="list-style-type: none"> <li><input type="checkbox"/> Tectonic zones:                             <ul style="list-style-type: none"> <li>Major thrust zones which affect the project in great scale:</li> <li>Localized fault or disturbed zones:</li> </ul> </li> <li><input type="checkbox"/> Fracturing or Shearing:                             <ul style="list-style-type: none"> <li>Fracturing degree:</li> <li>Continuation - persistence of fracturing with depth:</li> <li>Shearing or foliation across the rock mass:</li> </ul> </li> <li><input type="checkbox"/> Folding:                             <ul style="list-style-type: none"> <li>Type:</li> <li>Geometry:</li> </ul> </li> </ul>	<ul style="list-style-type: none"> <li><input type="checkbox"/> The area is disturbed by several thrusts. The rock mass is sheared in a wider area (tens of m)</li> <li><input type="checkbox"/> The disturbance is not localized in a specific fault zone (several m)</li> <li><input type="checkbox"/> Slightly fractured    <input type="checkbox"/> Fractured    <input type="checkbox"/> Very fractured    <input checked="" type="checkbox"/> Brecciated    <input type="checkbox"/></li> <li>The tectonic disturbance continues in depth for tens of m. The sandstone beds are broken, ground is transformed into small pieces.</li> <li>Soluble bedding is highly sheared and foliation planes of very low strength are present. Intact rock of siltstone has been intensively sheared</li> <li><input type="checkbox"/> Several folds (recumbent type)</li> </ul>
<b>c) Weathering</b>	
<ul style="list-style-type: none"> <li><input type="checkbox"/> Discontinuities:</li> <li><input type="checkbox"/> Intact rock:</li> <li><input type="checkbox"/> Persistence with depth:</li> </ul>	<ul style="list-style-type: none"> <li><input type="checkbox"/> Weathering is strongly favored along the siltstone planes. Clay minerals are formed</li> <li><input type="checkbox"/> The rock is disintegrated in small pieces due to staking</li> <li><input type="checkbox"/> Weathering is limited only close to the surface</li> </ul>
<b>d) Permeability</b>	
<ul style="list-style-type: none"> <li><input type="checkbox"/> Qualitative appraisal:</li> <li><input type="checkbox"/> Quantitative appraisal:</li> </ul>	<ul style="list-style-type: none"> <li><input type="checkbox"/> High (<math>k &gt; 10^{-3}</math> m/sec)    <input type="checkbox"/> Low (<math>k &lt; 10^{-10}</math> m/sec)    <input type="checkbox"/> Practically impermeable (<math>k &lt; 10^{-12}</math> m/sec)</li> <li><input type="checkbox"/> Medium (<math>k: 10^{-4}-10^{-9}</math> m/sec)    <input type="checkbox"/> Very low (<math>k: 10^{-11}-10^{-10}</math> m/sec)</li> <li><input type="checkbox"/> <math>k =</math> _____ m/sec</li> </ul>
<b>II. IN SITU CONDITIONS AND TUNNEL CHARACTERISTICS</b>	
<b>a) Tunnel Geometry</b>	
<ul style="list-style-type: none"> <li><input type="checkbox"/> Tunnel Size:</li> <li><input type="checkbox"/> Shape:</li> <li><input type="checkbox"/> Tunnel Direction:</li> </ul>	<ul style="list-style-type: none"> <li><input type="checkbox"/> 12m</li> <li><input type="checkbox"/> Horseshoe</li> <li><input type="checkbox"/> Horizontal in E-W direction</li> </ul>
<b>b) Overburden</b>	
<ul style="list-style-type: none"> <li><input type="checkbox"/> Overburden range with similar behaviour:</li> <li><input type="checkbox"/> Insitu stresses (<math>P_1 = \gamma H_{top}</math> to <math>\gamma H_{base}</math>):</li> </ul>	<ul style="list-style-type: none"> <li><input type="checkbox"/> H: 150-200 m</li> <li><input type="checkbox"/> <math>P_1 = 5</math> MPa</li> </ul>
<b>c) Stress field particularities</b>	
<input type="checkbox"/> Particular presence of lateral pressures ( $k_1$ ):	
<b>d) Adjacent zone close to tunnel perimeter</b>	
<input type="checkbox"/> Weak zone close to tunnel perimeter:	Dip: _____ / _____ Dip Direction Thickness: _____ m Geological characteristics: _____
<input type="checkbox"/> Competent zone close to tunnel perimeter:	Dip: _____ / _____ Dip Direction Thickness: 3 m Sandstone beds overlying the weak sheared rock mass. If these beds are close to the tunnel perimeter, strains could be reduced Geological characteristics: _____
<b>e) Hydrogeological conditions (location of aquifer according to the tunnel axis)</b>	<input type="checkbox"/> Aquifer is located above the tunnel axis
<b>f) Other boundaries</b>	<input type="checkbox"/>
<b>III. CHARACTERISTIC "KEYS" FOR TUNNEL BEHAVIOUR OR INSTABILITY</b>	
<input type="checkbox"/> Intact rock strength:	<input type="checkbox"/> Solition strength has been considerably reduced due to shearing. Sandstone strength does not significantly contribute to the so weighted intact rock
<input type="checkbox"/> Rock mass strength to insitu stress ratio ( $\sigma_{int}/P_1$ ):	<input type="checkbox"/> $\sigma_{int}/P_1 > 0.6$ <input type="checkbox"/> $0.3 < \sigma_{int}/P_1 < 0.6$ <input checked="" type="checkbox"/> $\sigma_{int}/P_1 < 0.3$
<input type="checkbox"/> Structure "interlocking":	<input type="checkbox"/> Sandstone beds marginally "follows" the folding of the siltstone beds
<input type="checkbox"/> Presence of low strength minerals:	<input type="checkbox"/> Clayey minerals maybe present. Possible swelling minerals
<input type="checkbox"/> Intact rock weathering, clay zones:	<input type="checkbox"/> Clay zones due to intensive shearing
<input type="checkbox"/> Groundwater presence:	<input type="checkbox"/> Groundwater is present but cannot be drained easily
<input type="checkbox"/> Block geometry - bed thickness:	<input type="checkbox"/> Only small sandstone blocks may be present
<input type="checkbox"/> Rock mass structure (based to GSI classification)	<input type="checkbox"/> Blocky <input type="checkbox"/> Very blocky <input type="checkbox"/> Blocky/Disturbed/Seamy
<input type="checkbox"/> Discontinuity geometry:	<input type="checkbox"/> Disintegrated <input type="checkbox"/> Laminated/Sheared <input checked="" type="checkbox"/>
<input type="checkbox"/> Discontinuity persistence:	<input type="checkbox"/> The geometry "shifts" within 1-2m
<input type="checkbox"/> Discontinuity quality (based to GSI classification)	<input type="checkbox"/> The discontinuities, if possible to be measured in some points, are not persistent
<input type="checkbox"/> Rock Quality Index (RQD):	<input type="checkbox"/> Very good <input type="checkbox"/> Good <input type="checkbox"/> Fair <input type="checkbox"/> Poor <input checked="" type="checkbox"/> Very poor <input checked="" type="checkbox"/>
<input type="checkbox"/> Other characteristic:	<input type="checkbox"/> RQD: N/A
<b>Fundamental engineering geological characteristics - "Keys":</b> Low intact rock strength ( $\sigma < 10$ MPa), Very disturbed-sheared structure (GSI=15-20), Very low compo. Water pressure	
<input checked="" type="checkbox"/> The behaviour is controlled by the overall rock mass: It is essential to enter the relevant rock mass parameters in field "Vc"	<input type="checkbox"/> The behaviour is controlled by the discontinuities: It is essential to enter the relevant parameters of the discontinuities in field "Vd"

(a)

**GROUND CHARACTERIZATION, BEHAVIOUR AND SUPPORT FOR TUNNELS (2/2) (V. Marinos, 2012)**

**IV. ROCK MASS BEHAVIOUR IN TUNNEL EXCAVATION** - The user may consult the Tunnel Behaviour Types Table  
- The user may consult the TBC classification in order to fill the section IV.

a) Isotropy - Anisotropy (Stress or gravity driven failures):  
 Isotropic  
 Anisotropic:  Yes  No The behaviour of the rock mass is clearly isotropic

b) Behaviour type of unsupported tunnel section:  
 Qualitative:  
 The behaviour is controlled by its size strength and local deformability that are susceptible for the development of important deformations, even under low to medium overburden. If there is a consistent "package" of competent sandstone beds close to the tunnel roof, the deformations could be less. In greater depths, squeezing conditions can be adverse causing sometimes failure of rigid support actions due to overloading of the steel.  
 Particular care is needed close to the surface, where important overexcavation can occur, due to overburden and the relative height structure.



c) Design philosophy:  
 Structural dependent instability analysis (e.g. Unwedge Programs)  
 Structural and stress dependent instability analysis (Wedge and Numerical Analysis)  
 Stress dependent instability - Deformation analysis (Numerical Analysis)  
 Empirical design  
 Three dimensional numerical models to analyze adequately issues of face stability and sequential excavation and installation of support

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**V. DETAIL CHARACTERISTICS AND DESIGN PARAMETERS** - Focus on  $V_{a1}$  and/or  $V_{a2}$  according to the rock mass behavior

a) Rock mass parameters (Hoek & Brown):  
 GSI classification value: 19-23  
  
 Intact rock strength:  $\sigma_{ci}$ : 15 MPa  
 Constant  $m_i$ : 7  
 $\gamma$ : 0.025 MN/m<sup>3</sup>  
 Modulus Ratio (MR) or ( $E_i$ ): 3300 MPa  
 Disturbance factor (D): 0

b) Discontinuity parameters:  
 Number of discontinuities: 3  
 Geometry ( $D_i$ /dip direction):  $3_1$ ,  $3_2$ ,  $3_3$   
 Persistence: m, m, m  
 Distance apart or Spacing: m, m, m  
 Aperture: mm, mm, mm  
 Filling material:  
 Hard < 5mm, Hard > 5mm, Soft < 5mm, Soft > 5mm, None  
 Weathering:  
 Unweathered, Slightly, Moderately, Highly, Decomposed  
 Ground water conditions:  
 Dry, Sub-wet, Wet, In drops, Flow  
 Joint Roughness Condition (JRC):  
 Joint Compression Strength (JCS): MPa, MPa, MPa

Shear strength properties of rock mass:  
 Friction angle ( $\phi$ ): 22°  
 Cohesion (c): 0.25 MPa  
 Deformation modulus ( $E_{m1}$ ): 150 MPa  
 Rock mass strength ( $\sigma_{cm}$ ): 1 MPa  
 Hoek & Brown parameters ( $m_b$ ,  $s$ ):

c) Other rock mass classification value  
 RMR: Q: RQD:  $Q_i$   
 Discontinuity parameters can be assessed from  $V_b$

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**VI. TUNNEL SUPPORT PHILOSOPHY** - The specific support measures and loads must be calculated through detailed design analysis  
- The user may also consult the Tunnel Support Measures for Each Tunnel Behaviour Type Table in order to complete section VI.

Qualitative:  
 - Excavation phases:  
 - Excavation step:  
 - Shotcrete/bolts:  
 - Steel sets:  
 - Light face support for structurally dependent instability (e.g. spiles)  
 - Face support against stress dependent instability (e.g. fibreglass, forepoling, invert)  
 - Water drainage:  
 - Other (e.g. grouting):

- At 3 phases (Top Heading, Bench and final Invert)  
 - Small excavation step (~1m)  
 - Dense bolt pattern to control the deformation  
 - Steel sets in order to increase the rigidity and strength of the support shell  
 - Face retaining measures: Depending on excavation depth (fibreglass nails or/and forepoling)  
 - Permanent and temporary invert to improve the shell rigidity  
 - Special support requirements should be considered in case of swelling rockmasses  
 - (e.g. possible in type VI, VIII, X, XI)  
 - If possible, drainage relief holes are required  
 - In case of very high overburden (>100-150m) the construction of a flexible support system using yielding elements may be required

---

**VII. REMAINING RISK**  
 Special support requirements should be considered in case of swelling rockmasses (e.g. possible in type VI, VIII, X, XI)

(b)

Page 2 from 2

Figure 7.

(a) Modified example of the ground characterisation, behaviour and support for tunnels illustrated [31], in light characters by an example of tunnelling in a tectonically deformed intensively folded siltstone (flysch rock mass type X) [for page 2 from 2 see (b)].

Apart from a few cases of simple tunnelling conditions in areas of good rock mass types of flysch (sorts I–V), most of the investigated tunnels were excavated under challenging geological conditions (sorts VII–XI). These tunnels have been excavated utilising top heading and bench methods. Particular measures were applied to stabilise the face, such as forepoling or/and establishment of long

grouted fibreglass dowels in the face. Furthermore, immediate shotcreting and face buttressing have been utilised in several combinations for face stabilisation. After the stabilisation of the tunnel face, the application of the immediate support shell, consisting of shotcrete layers, rockbolts and steel sets embedded within the shotcrete in different combinations, was essential to ensure the stability of the tunnel. Elephant's foot and, in uncommon cases, micropiles were utilised to help the establishment of the top heading foundation zone and to secure stability when benching. Temporary and final invert closure was applied to meet the squeezing conditions.

Under severe squeezing, the application of yielding systems is an alternative solution [e.g. in Schubert, 1996, 20]. In the case of tectonically sheared siltstone rock masses under high cover (e.g. up to 250 m), where tunnel squeezing is a significant problem, the pillar stability in these twin tunnels requires careful evaluation.

### **3.2 Molassic formations (non-tectonically disturbed heterogeneous rock masses)**

The term molasse comes from a Swiss local title at first allotted to soft sandstones related to marls and conglomerates belonging to the tertiary that had an extraordinary advancement within the lowland parts of Switzerland. They are as a result of debris of weathering and erosion of the Alpine mountains. The term is currently used to describe the deposits from the erosion of a mountain belt after the ultimate stage of orogenesis behind the mountain building zone. Molasse comprises of a sequence of tectonically undisturbed sediments of sandstones, conglomerates, siltstones and marls. Molassic rock masses may have exceptionally distinctive structures near to the surface compared to those restricted at depth, where bedding strata do not show up as clearly characterised joint surfaces that separate the rock mass into blocks [12].

Tunnelling through molassic rocks is based on the experience picked up from the design and construction of 12 tunnels along the Egnatia Highway in northern Greece. A context is displayed here concerning the distinctive rock masses of molassic rocks, the geotechnical behaviour of each type in tunnelling and the temporary support philosophy, both for underground construction and portal zones. The major characteristics of the investigated geomaterial that cause its specific tunnel behaviour are (a) the lithological heterogeneity, as the series comprises of a nearly continuous units of sandstones, siltstones, marls or claystones and conglomerates, with alternations of layers from some centimetres to some metres thick; (b) the low to moderate strength of the intact rock of these units; (c) the compact, nearly intact structure at depth, indeed when sandstone strata alternate with siltstones; and (d) the problematic behaviour of the siltstone-marly units near to the surface due to slaking and weathering.

Molassic rocks display noteworthy differences between the surface and at depth. These contrasts lie within the rock mass fabric, weathering and permeability and thus are exceptionally critical for the rock mass quality and behaviour in tunnelling. Molassic rocks, especially sandstone and well-cemented conglomerates, tend to be profoundly frictional. Due to the narrow deformation to which they have been subjected in deposition, the discontinuity in these rocks is by and large free from the impacts of shear development (slickensides).

Siltstone or claystone beds, being restricted shortly beneath the surface, are compact enough to create a nearly unbroken medium. Their presence may, be that as it may, diminish the quality of the whole rock mass, due to its nature. In any case, there are occasions where siltstones are fairly competent and below low stress; their behaviour does not essentially contrast from that of sandstones. The bedding is the basic joint set in a molassic rock mass but is only communicated on and close to the surface.

At depth, the bedding is mostly concealed. For the cases examined in this chapter, rock quality designation (RQD) values close the surface run from 0 to 50%. At low depths (~5 m), the rock masses ended up medium broken and weathered, whereas bedding planes are still apparent. At depths greater than 10–15 m, the rock masses are as a rule homogeneous in structure and continuous, with RQD values >60%.

Weathering usually transforms the rock mass strength. Siltstone (or marly) members are susceptible to weathering, and fissility may be built parallel to the bedding when these rocks are uncovered to the surface or are close to it. Siltstone (or marly) members in outcrops show up thin layered or even schistosed, and when they alternate with sandstones, the appearance of the rock mass takes after that of flysch. This appearance in outcrops can be deceiving when considering the behaviour of molassic rocks in a limited underground environment, in which the slaking is confined and the rock mass is massive. There are conditions where sandstones are loose and may be treated as dense sands. In such poor molasses, clays and silts also present, and the fabric can be treated like a soil. It is not the goal of this chapter to address these soil-like molasses that have constrained spatial dissemination. In any case, it ought to be underlined that molasses close to the surface may make a cover with such soil characteristics.

Based on the outcomes of numerous in situ permeability tests (Marinos et al. [9]), the overall permeability of the molassic series is rapidly reduced with depth. Though, the permeability of the sandstone members within the molasse is altogether higher than that of the siltstone ones. Within the case of variations of the two types of rocks, the permeability approaches the value of the siltstone since the siltstone layers do not permit the water flow through the rock mass and decrease the overall permeability. Besides, the frequent horizontal transitions do not allow the development of a uniform aquifer. Fault zones, in spite of the fact that they are more permeable, are neither frequent nor extensive. Thus, in spite of the fact that the water table will ordinarily be over the tunnel, only minor water inflows are expected, in spite of the fact that in a few circumstances it may be essential to relieve water pressures by drilling.

The high strength of the molassic rock mass in relation to the in situ stresses at shallow to medium depths does not qualify stress-driven failures. The prevailing failure mode in tunnels is the gravity-induced falls and slides of rock blocks and wedges characterised by intersecting joints and bedding planes. It ought to be stressed, however, that this behaviour has been confirmed with tunnel construction in depths up to 110 m and should not be reflected for much larger depths.

These types of behaviours are differentiated in two regions (see **Figure 4**):

- Stable (St) within the case of massive structure and shallow to medium tunnel covers. As the tunnel depths increases, stable behaviour with no deformation can be assumed for sandstone- or conglomerate-dominated series (zones #1, #2 and #4 in **Figure 4**).
- Stable (St) with limited strains (Sh), particularly in cases of siltstone-dominated rock mass types, under notable tunnel cover. The size of the resulting deformation depends on the strength of the siltstone and the overburden. Serious strains have not been experienced along the Egnatia tunnels, as the greatest depth was restricted to 110 m (range #3 in **Figure 4**).
- Wedge failure (Wg) in cases of blocky rock masses and shallow to medium depths (ranges #5 and #6 in **Figure 4**). Similar characteristics with minor deformations are witnessed for sandstone or conglomerate formations with the increase in tunnel depth. The developing confinement with depth may result in

less wedge sliding events (St-Wg) (range #8 in **Figure 4**). A marginally different failure mode can be presented within the case of thin-bedded series with nearly horizontal bedding planes. Failure of rock blocks due to self-weight from the crown section may be occasional and extensive once their base is exposed due to deconfinement which might cause subvertical tension joints. Such unfavourable conditions must be controlled to face systematic crown failures.

- Wedge failure with limited deformation (Sh-Wg), within the case of siltstone governed formations, beneath critical tunnel cover (region #7 in **Figure 4**). The size of the deformation depends on the intact rock strength of the siltstone and the depth of the tunnel.
- Broad wedge failures that can advance into chimney-type failure (Ch-Wg) within the case of weathered and disturbed fabric near to the surface (portal zones or under shallow depths beneath streams or gullies) due to slaking and loosening of the siltstone parts (ranges #13 and #14 in **Figure 4**).
- Repeated wedge failure that can slowly transform into chimney-type failure (Wg-Ch) along the case of exceptionally blocky rock masses due to faulting. In the case of siltstone-dominated rock mass types, at medium to large depths, limited deformations (Sh) can be developed (region #13 in **Figure 4**). However, considerable strains have not been experienced within the tunnels of the Egnatia Highway since the greatest depth of the tunnels, within these rock masses, was 110 m.

With regard to water inflows, minor occurrence of water has been met along the 12 tunnel projects, which develop basically within the form of increased moisture to drips. In a few uncommon cases, periodic or continuous low flow at different areas, primarily in sandstone-siltstone contact layers and along major discontinuities, has been experienced. However, this presence degrades the characteristics of the discontinuities and ought to be taken under consideration when evaluating the geotechnical characteristics of the rock mass types. The low geotechnical properties of the molassic formations, near to the surface, have driven to numerous failures in the portal areas. These instabilities were not directed by pre-existing discontinuities, such as the bedding planes, but they were related to the advancement of a new circular-shaped failure surface across the weathered poor rock mass.

The tunnel support concept in molassic rock mass types must take into consideration the rock mass fabric and the expected failure modes in connection to the depth as depicted above. These approaches for the philosophy of temporary support measures have been formed based on the geotechnical behaviour of molassic series as well as on construction data. Along the tunnels of the Egnatia Highway through molassic formations, 54% of the whole length was excavated employing a support category with shotcrete shells, anchors, steel sets and light spilling. A really light support category containing a thin shotcrete shell and a sparse pattern of bolts was received for 38% of the entire length. At long last, a heavy support category with thick shotcrete shells, steel sets, forepoling and fibreglass nails was executed in only 6.5% of the full length and basically within the region of the portals. Hence, absent from the ground surface, where the rock mass is subjected to surface weathering conditions and any credible fault zones, there are two basic types of immediate support systems that could be implemented.

The first type concerns stable conditions with solely gravity-controlled failures and minor to zero deformation. This is often the foremost common case for all molasses at depth and ought to be connected for low to medium overburden or

indeed under higher overburden in cases of sandstone or conglomerate mastery. The immediate support comprises of a thin shotcrete layer and a pattern of rock bolts, while the advance step can be 3–4 m. The primary 3–5-cm-thick layer of shotcrete, implemented, as soon as possible, on the uncovered rock mass surfaces, seals and secures the siltstone layers from slaking. The rock bolt design reinforces the rock mass, keeps it restricted and prevents likely gravity-controlled falls of loose, fundamentally defined blocks or wedges due to decompression of the otherwise sealed bedding planes. The introduction of another layer of shotcrete, strengthened either by wire mesh or by fibres, makes a complementary shell, engaging the heads of the rock bolts and guaranteeing the stability of the tunnel. The next type of support system for competent molasses absent from portals or faults alludes (a) to conditions with frequent wedge failures due to the geometry of major joint systems and the conditions already displayed (horizontal bedding planes) and/or (b) to cases of weak rock (e.g. siltstone) governed by molassic formations below considerable to large overburden. In expansion to the shotcrete and the rock bolts, light steel sets may be necessary, while the advance step must be restricted (around 2 m) to prevent any wedge formation or critical strains in the case of large depths.

For weathered molassic series near to the tunnel portals or intensely jointed and poor molassic rock masses along fault zones, stiffer support is required by the use of heavier steel sets and a thicker shotcrete shell. Consideration ought to be given to limiting disturbance to the encompassing geomaterial by reducing the excavation step (~1 m). Furthermore, it may be essential to stabilise the tunnel face utilising face support measures (e.g. fibreglass nails) or face protection schemes (e.g. spiles or forepole umbrella) to avoid progressive detachment, deconfinement and creation of chimney-type failures.

### **3.3 Ophiolitic complex**

The term ophiolite was at first given to a series of basic and ultrabasic rocks, more or less serpentinitised and transformed, appearing within the Alpine chains. Ophiolites are presently considered as pieces of the oceanic crust produced at an oceanic ridge and the upper mantle of an ancient ocean, thrust up on the continental crust during mountain building [32].

The ophiolitic complex is in a general sense characterised by underlying peridotitic rocks that are overlaid by gabbroic/peridotitic rocks, which, in turn, are covered by basalts or spilites. The basal peridotites are laminated ('tectonites'). The subsequent alternations of peridotites and gabbros frequently have a layered structure of cumulates and are taken after by enormous gabbros, norites or other basic rocks richer in SiO<sub>2</sub>. The overlying basalts are either continuous or within the frame of pillow lavas. In between these rocks, sedimentary rocks of deep sea may be stored. This geometry is exceedingly exasperated since the ophiolitic complexes happen primarily in tectonic zones with superposition of various overthrusts. Metamorphism, which is additionally displayed, changes the initial nature of the materials. The high degree of serpentinitisation and the intensity of shearing can make it difficult to distinguish any lattice mineral of either fibrous or laminar shape. This unordinary alteration is a phenomenon of autohydration that occurs amid the final phases of the crystallisation of magma where there's an abundance of water. In other scenarios, serpentinitisation compares to a low initial cumulate texture [33].

Serpentinitisation is the change of ferromagnesian minerals, specifically olivine, to serpentine—a grade metamorphism of peridotites. In all these cases, the peridotites can be changed into serpentinite. This new rock is initially compact, moderately soft and more naturally sheared by tectonic processes. Serpentinitisation can moreover be created due to exogenic conditions with meteoric water under regular

weathering conditions. In this case, the alteration deteriorates the parent peridotite to a schistosed mass and later to clayey soil-like mass. The development at depth of weathered peridotites is less generalised up and clearly restricted compared with the endogenic serpentinisation portrayed already [13].

Rock masses in an ophiolitic complex display a wide variety of engineering behaviour in tunnelling. Typically, this is true due to their petrographic variety and structural complexity. An advanced degree of serpentinisation together with the increased shearing may result in a mass in which it is hard to recognise any initial surface or texture. Thus, behaviour can change from stable to severe squeezing conditions in cases where ophiolites are related with overthrusts. The main rock mass types are peridotites, gabbros, pillow lavas, peridotites that are more or less serpentinised, serpentinites, schisto-serpentinites, sheared serpentinites and chaotic masses in melanges. Peridotites are sound and behave as typical brittle materials. Serpentinisation can be found along the discontinuity surfaces, and the conditions of the joints are significantly reduced to very poor with coatings of 'slippery' minerals such as serpentine or talc. In a disturbed ophiolitic mass, the serpentinisation procedure regularly loosens and disintegrates parts of the rock matrix itself, contributing to lower GSI values and reducing the intact strength [13].

The extraordinary assortment of numerous rock mass types, the unpredictable changes and the alteration mark the ophiolites a formation where great care is required in the tunnel design. This is often true for tunnel projects due to their linearity and their depth that increase the possibility of experiencing the unfavourable zones related with the ophiolites, whereas the uncertainty as to their exact location and extent impairs the difficulty.

In sound and competent rock masses of peridotite, simple and straightforward tunnelling conditions can be anticipated, where consideration has got to be concentrated on maintaining wedge failures. Within the case of a more broken peridotite, schistose or great serpentinite, the behaviour is controlled by sliding and rotation on joint surfaces with generally little failure of the intact rock pieces. In this case, the control of stability can be amended during tunnel excavation by keeping the rock mass confined. In poor quality serpentinite, due to alteration or shearing, blockiness may be totally missing, and clayey areas with swelling materials may be present. Tunnel instability will at that point be due to stress-dependent rock mass failure with severe squeezing at depths [13].

Peridotites: In great quality masses of peridotite, straightforward tunnelling conditions can be anticipated. Consideration must be concentrated on controlling gravity-driven instabilities from wedges. For these failures comprising some joints, the issue is basically one of three-dimensional geometry and stereographic tools or numerical analyses such as UnWedge (see <http://www.roscience.com>) ought to be utilised for an investigation of design of support measures.

However, compared with other rock masses of comparative structures, the peridotites in a general sense have smoother joints with poor frictional properties. As clarified previously, it's due to the existence of serpentinised material, which is regularly present even if the serpentinisation has not affected the rock itself. This makes the gravity-driven failures more challenging and for the most part requests heavier rock bolting patterns and/or thicker shotcrete (zones #2, #4, #6 and #8 in **Figure 4** depending on the depth and intact rock strength). In exceptionally hard rock masses at large depths, spalling, slabbing and rockbursting are the mechanisms of failure which will be developed and controlled by brittle fracture propagation in the intact rock with the joints having as it were a minor influence. In these cases, the utilisation of the brittle rock failure models must be considered, such as that proposed by Kaiser et al. [34].

Disturbed peridotites or schistose serpentinites: Within the case of a more disturbed peridotite, schistose or weaker serpentinite, the behaviour is controlled

by sliding and rotation on joint surfaces with generally minor failure of the intact rock fragments (ranges #10, #12, #14 and #16 in **Figure 4** depending on the depth and intact rock strength). In this extent of GSI values, the RQD values can be exceptionally low. This is typical, given the structure of the rock masses, but some of the frictional behaviour of the unaltered fragments of the mass is reserved. In such cases, the control of the stability can be effectively achieved during tunnel excavation by maintaining the rock mass confined.

Sheared serpentinite, squeezing behaviour: In low-quality serpentinite, as a result of alteration or shearing, blockiness may be nearly totally lost, and clayey areas with swelling materials may be available. Tunnel stability will at that point be controlled by stress-dependent rock mass failure with significant squeezing at depths (regions #21 to #24 in **Figure 4** depending on the depth and intact rock strength). In these cases, a detailed numerical analysis must be performed that permits progressive failure and support interaction analysis to be demonstrated. In any case, it is exceptionally instructive to carry out a closed form analysis of the tunnel behaviour to get an indication of the significance and value of deformation [13]. The 'strain' can be evaluated from the proportion of the rock mass strength to the in situ stress [30]. This plot is valid to single circular-shaped tunnels.

#### **4. Conclusions**

In general, the application of well-known classification systems has the drawback of not necessarily displaying information concerning rock mass behaviour in tunnels. Consequently, there are many cases in which the geological 'identity' of the geomaterial is lost since it is not involved in the analysis, and in that way, it is possible that its special characteristics are mislaid. Despite the capabilities offered by the rapid advance of the numerical tools in the geotechnical design, the outcomes can still include uncertainties when parameters are utilised straightforwardly without considering the real failure mode of the rock mass in tunnelling. This chapter points out that the assessment of the principle tunnel failure mode is an essential information for the temporary support measure definition. The work presented in this chapter was based on a large set of data, incorporated in a tunnel information and analysis system (TIAS), from the design and construction of 62 tunnels through a wide variety of geological conditions.

Two classifications and characterisation schemes have been presented to assess tunnel behaviour based on the engineering geological identity of the rock masses. The primary, called the tunnel behaviour chart, is a classification framework for predicting the rock mass behaviour in tunnelling and covers a wide extent of rock mass conditions. This evaluation is based on the fabric of the rock mass, the strength of the intact rock and the tunnel cover. The moment, called Ground Characterisation, Behaviour and Support for Tunnels, is a step-by step appreciation of a rock mass quality, with detailed engineering geological and geotechnical characteristics, towards the evaluation of the foremost tunnel behaviour and its support requirements.

After defining the most possible failure types for every kind of the predicted rock mass, the most appropriate design parameters are identified, either of the rock mass, if it displays an isotropical behaviour, or characteristics of discontinuities if it behaves in an anisotropic manner. These proposals allow an early assessment of the principles for the choice of appropriate support measures and their basic dimensioning, as dictated by the ground behaviour and the associated mode of failure. The accuracy of the classifications and the support system can be managed directly from direct tunnel observation and monitoring.

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# Advanced Geological Prediction

*Shaoshuai Shi, Xiaokun Xie, Siming Tian, Zhijie Wen, Lin Bu, Zongqing Zhou, Shuguang Song and Ruijie Zhao*

## Abstract

Due to the particularity of the tunnel project, it is difficult to find out the exact geological conditions of the tunnel body during the survey stage. Once it encounters unfavorable geological bodies such as faults, fracture zones, and karst, it will bring great challenges to the construction and will easily cause major problems, economic losses, and casualties. Therefore, it is necessary to carry out geological forecast work in the tunnel construction process, which is of great significance for tunnel safety construction and avoiding major disaster accident losses. This lecture mainly introduces the commonly used methods of geological forecast in tunnel construction, the design principles, and contents of geological forecast and combines typical cases to show the implementation process of comprehensive geological forecast. Finally, the development direction of geological forecast theory, method, and technology is carried out. Prospects provide a useful reference for promoting the development of geological forecast of tunnels.

**Keywords:** advanced geological prediction design, content of advanced geological prediction, method of advanced geological prediction, geological hazard detection, engineering application

## 1. Introduction

### 1.1 Main contents and common methods of geological forecast

The advanced geological prediction of the tunnel includes geological analysis and macroscopic geological forecast of the tunnel area, advanced prediction of tunnel geological disasters, and warning of major construction geological disasters [1, 2]. The main forecast content [3–6] is as follows:

1. Stratigraphic lithology forecast, the focus is on prediction of soft interlayers, broken formation, coal seams, and special rock.
2. Geological structure prediction, the focus is on the prediction of the tectonic development of the rock mass integrity, such as faults, concentrated joint band, and fold axis.
3. Unfavorable geological prediction, the focus is on the prediction of karst, man-made tunnels, gas, etc.
4. Groundwater prediction focuses on the prediction of karst conduit water and water-rich faults, water-rich fold axis, and fissure water in water-rich strata.

In response to the abovementioned exploration targets, the researchers have developed geological forecast techniques for various types of construction tunnels. The common methods for geological forecast are geological researching, advance drilling geological prediction, geophysical prospecting prediction in tunnel, advance heading, etc. [7–13], as shown in **Figure 1**.

### 1.1.1 Hydrogeological survey

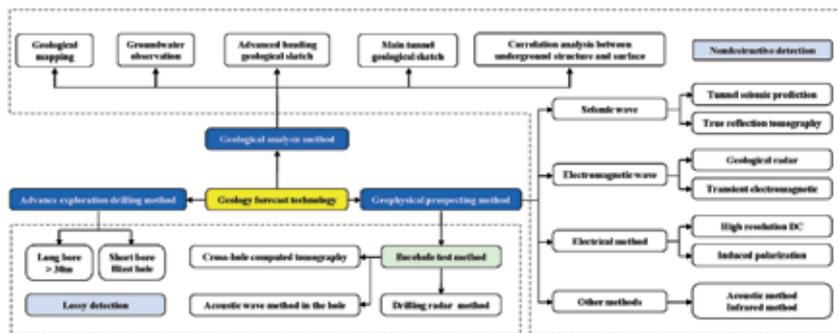
The hydrogeological survey method mainly includes the supplementary geological survey of the tunnel surface, the geological sketch of the working face in the tunnel and the geological sketch of the tunnel wall, the underground and surface correlation analysis of the stratigraphic boundary line and the structural line, and the geological mapping.

The hydrogeological survey method is the earliest and most basic method used in various tunnel geological advance prediction methods. The interpretation and use of other tunnel advance prediction methods are based on geological data analysis and judgment. The hydrogeological survey method is based on the existing survey data, the geological survey data supplemented by the surface, and the geological sketch in the tunnel, through the sequence comparison of the geological layers, the stratigraphic boundary line, and the correlation analysis of the sub-surface and surface of the stratigraphic tectonic line, the fault elements, and the tunnel geometry. Correlation analysis of parameters, possible precursor analysis of adjacent geological bodies in the tunnel, etc. use conventional geological theory, geological mapping, and geological development trend analysis, etc. to speculate the possible geological conditions ahead of the excavation face. The method has high accuracy in the case where the tunnel has a shallow depth and the structure is not too complicated, but in the case of deep depth and complicated structure, the method is difficult, and the accuracy is poor.

### 1.1.2 Probe drilling

**Probe** drilling methods mainly include advanced geological drilling, deepened shot hole detection, and borehole photography.

The **probe** horizontal drilling method is a kind of advanced geological prediction method for obtaining geological information by drilling with drilling equipment or directly using blasting holes to drill ahead in the tunnel excavation working face. The method can directly reveal the lithology, rock structure, groundwater, karst cave and its properties, rock integrity degree, etc., from tens of meters to hundreds of meters in front of the tunnel working face, and can also



**Figure 1.** Common methods for geology forecast of tunnels.

be obtained through core test. Quantitative indicators such as rock strength are applicable to the main unfavorable geological sections that have been basically identified. For undetermined unfavorable geological sections, the unsatisfactory leakage of unfavorable geological bodies is often caused by the problem of “one hole seeing.”

### *1.1.3 Geophysical prospecting*

Geophysical methods mainly include elastic wave reflection method (seismic wave reflection, horizontal acoustic wave profile method, negative-vision velocity method, and very small offset high-frequency reflection continuous profile method), electromagnetic wave method (geological radar, transient electromagnetic), and electrical method (high-resolution DC method, induced polarization method, etc.).

The geophysical prospecting is based on the physical difference between the target geological body and the surrounding medium, such as electrical, magnetic, density, wave velocity, temperature, radioactivity, etc., and the spatial distribution of the underground geological body is determined by observing changes in natural or artificial physics. The scope is a physical exploration technology that solves geological problems. The method is fast, comprehensive, accurate, and economical. It is a nondestructive testing method, which mainly includes seismic wave reflection method, electromagnetic wave method, and electric method.

1. Seismic wave reflection: The basic principle of seismic wave reflection method is to use the characteristics of reflected waves generated by seismic waves in uneven geological bodies to predict the geological conditions in front of and around the tunneling face. At present, the main methods include tunnel seismic prediction (TSP), tunnel reflection tomography (TRT), and terrestrial sonar.
2. Electromagnetic wave method: The electromagnetic wave method is a detection method for detecting the distribution of underground medium by using ultrahigh frequency electromagnetic waves, including geological radar method and transient electromagnetic method.
3. Electrical: The electrical method uses the distribution characteristics and laws of the DC electric field to detect the surface of the face and the surrounding underground medium. The common methods used in tunnel advance prediction include induced polarization method and high-resolution direct current method.
4. Drilling test: The borehole test method combined with **leading** horizontal drilling and geophysical methods can make the geophysical approach closer to the exploration target and away from the tunnel interference source, thus achieving good detection results. Drilling test methods include cross-hole CT, acoustic wave method in borehole, and borehole radar method.

### *1.1.4 Advance heading method*

The advance heading prediction method mainly includes the parallel advance heading method, the positive hole advance heading method, etc.

The advance heading prediction method is used to excavate a parallel heading in the tunnel or on the side of the tunnel, through geological conditions revealed in

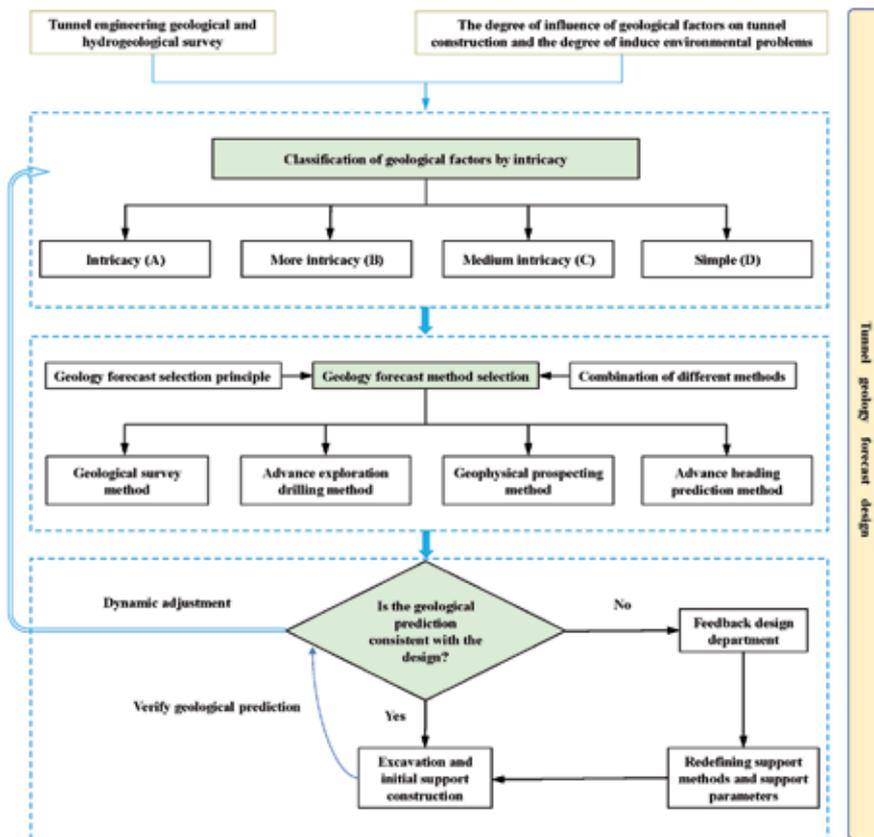
advance heading, geological theory, and mapping method to predict the geological conditions of the main tunnel. The advance heading prediction method includes parallel advance heading method and positive hole advance heading method. The two tunnels with small line spacing can be parallel heading to each other; the tunnel excavated first can predict the geological conditions of the tunnel excavated after. Because of its large cross section, the advance heading method can reveal the geological conditions in front of the positive hole more comprehensively and accurately, but it takes a long time and has high economic cost.

## 2. Geology forecast design

### 2.1 Geology forecast design

The tunnel engineering should carry out corresponding geology forecast design at each design stage, and the selection of prediction methods should be compatible with the construction method. The geology forecast design can be implemented by referring to the following steps, as shown in **Figure 2**:

1. The geological survey method is adopted to investigate the engineering geological and hydrogeological conditions in the area and obtain unfavorable geological structures, special geotechnical areas, and possible geological problems.



**Figure 2.** Flow chart of tunnel geology forecast design.

2. Based on the results of geological surveys, obtaining the classification of geological factors by intricacy.
3. According to the condition of classification of geological factors by intricacy and the selection principle of the geology forecast method, the corresponding advanced prediction methods are selected.
4. In the process of tunnel excavation, according to the results of geology forecast and actual geological condition dynamic, adjust the tunnel classification of geological factors by intricacy and geology forecast design.

## 2.2 Classification of geological factors by intricacy (hazard)

Considering comprehensively the geological and hydrogeological conditions of the tunnel, the influence degree of the possible geological hazards on the tunnel construction and environment classifies the geological factors by intricacy. Tunnel classification of geological factors by intricacy (hazard) is shown in **Table 1** [3]. The purpose of the tunnel classification of geological factors by intricacy (hazard) is to determine the depth (accuracy) of geological prediction and exploration; to select different exploration methods and means (and their combination); determining the relevant technical requirements, workload, and so on; to complete the geology forecast design of tunnel construction; and to realize scientific planning and controllable management of geological prediction in tunnel construction.

## 2.3 Advanced geological prediction method selection

The advanced geological prediction can generally adopt long-distance forecast, medium and long-distance forecast, and short-distance forecast. The choice of forecast length and forecast method should meet the following requirements:

1. Long-distance forecast: If the forecast length is more than 100 m, geological survey methods, elastic wave reflection methods, and **probe** drilling of more than 100 m can be used.
2. Medium and long-distance forecast: If the forecast length is 30–100 m, geological survey methods, elastic wave reflection methods, transient electromagnetic methods, and **probe** drilling of 30–100 m can be used.
3. Short-range forecast: If the forecast length is <30 m, geological survey methods, elastic wave reflection methods, electromagnetic wave methods (geological radar method), electrical methods (high-resolution direct current method and tunnel-induced polarization method), and **probe** drilling <30 m can be used.

## 2.4 Forecast method in typical bad geological body prediction

### 2.4.1 Fault

1. The geological survey method can further verify the nature, occurrence, location, and scale of the fault.
2. The seismic wave reflection method can determine the approximate position and width of the fault within the tunnel.

Influencing factor	Degree of intricacy			
	Intricacy (A)	More intricacy (B)	Medium intricacy (C)	Simple (D)
Geological intricacy	Karst development	Strong development	Medium development	Weak development
	Water and mud gushing	Extra-large water inrush	Larger water inrush	Medium water inflow
	Fault stability	Large fault fragmentation zone, poor self-stabilization capacity, rich in water	Medium-sized fault zone, weak, medium-to weak rich in water may cause medium-sized collapse	Small- and medium-sized faults, weakly rich in water, may cause small collapses
	Ground stress	Extremely high stress ( $R_c/\sigma_{max} < 4$ )	High stress ( $R_c/\sigma_{max} = 4-7$ )	—
Gas influence	Gas outburst: gas pressure $P \geq 0.74$ MPa	High gas: gas emission in the whole work area $\geq 0.5$ m <sup>3</sup> /min	Low gas: gas emission in the whole work area $< 0.5$ m <sup>3</sup> /min	No
Geological factors on tunnel construction	Endangering construction safety	Security risks	May have security issues	Local security issues may exist
Environmental problems	May cause major environmental disasters	Improper construction	General environmental problems	No

where  $R_c$  is the rock uniaxial saturated compressive strength (MPa) and  $\sigma_{max}$  is the maximum geostress value.

**Table 1.** Classification of geological factors by intricacy (hazard).

3. High-resolution direct current method, electromagnetic wave method, and tunnel-induced polarization method can detect the development of groundwater in fault zone.
4. **Probe** drilling can predict the exact location and scale of the fault, the material composition of the fracture zone, and the development of groundwater.

#### *2.4.2 Karst*

1. The geological survey method can analyze the law of karst development and grasp the regional geological conditions.
2. The seismic wave reflection method can be used to ascertain the structural planes such as faults and the karst morphology that is large enough to be detected.
3. High-resolution direct current method, electromagnetic wave method, etc. can qualitatively detect karst water.
4. Tunnel-induced polarization method to find out the three-dimensional location, scale, and shape of karst and the size of static reserves.
5. **Probe** drilling can combine the short-range fine detection results to find out the karst scale, development characteristics, water pressure, and so on.

#### *2.4.3 Coal seam gas*

1. The geological survey method can further verify the position and thickness of the coal seam and analyze and determine the mileage position of the coal seam.
2. The geophysical method can determine the approximate location and thickness of the coal seam within the tunnel.
3. **Probe** drilling can calibrate the exact position of each coal seam and grasp its occurrence and gas condition.

### **2.5 Advanced geological prediction design content**

The advanced geological prediction design of the tunnel firstly evaluates the complexity of the geological complex (hazard) of the tunnel and clarifies the risk events and risk levels. The assessment of the complexity of the geological complexity of the tunnel can initially determine the causes, possibilities, and consequences of various risks. Secondly, according to the complexity of the tunnel and the risk assessment after the prediction plan is made, the prediction grading can be reasonably determined. In the case of complex high-level sections, several geophysical methods with complementary physical parameters should be comprehensively implemented. Targeted advance drilling can be carried out purposefully according to the results of geophysical exploration, and we should give full play to the advantages of geophysical exploration and drilling so that we can achieve accurate and resource-saving objectives. Finally, the forecasters were requested to adopt advanced data collection and processing methods, strive to improve the level and accuracy of the prediction, further investigate the engineering geological and hydrogeological conditions in front

of the tunnel excavation face, guide the smooth progress of the project construction, and reduce the probability of a geological disaster occurring. The advanced geological prediction design relies on the assessment of the complexity of the tunnel geological (hazard) and can be divided into the following four levels, as shown below:

A-level forecast: Based on the geological survey method, integrated seismic wave reflection method (TSP, TRT, etc.), electromagnetic wave method (GPR, GPR, etc.), electrical method (high-resolution direct current method), and other methods conduct comprehensive prediction. According to the comprehensive prediction conclusion, the advanced prediction method is used to verify the comprehensive prediction conclusion. For water-rich layer, detection methods such as transient electromagnetic method (TEM) and tunnel-induced polarization (TIP) should be added to qualitatively locate and estimate the water-bearing structure and supplemented with information such as targeted drilling to detect water pressure to guide the design and construction.

B-level forecast: Based on the geological survey method, mainly seismic wave reflection method (TSP, TRT, etc.), and supplemented by electromagnetic wave method (GPR), electric method (high-resolution direct current method), etc., conduct comprehensive prediction. Adopting **probe** drilling method verifies the forecast conclusions. When it is found that the engineering geological conditions of the local section are complicated and rich in water, it is implemented according to the requirements of class A.

C-level forecast: Based on the geological survey method, the seismic wave reflection method is mainly used to detect the important geological (layer) interface, fault fracture zone, karst cave, or geophysical anomaly section by electromagnetic wave method and high-resolution direct current method. The drilling method is adopted to verify the prediction conclusion.

D-level forecast: Geological survey method is the main method, supplemented by seismic wave reflection method, and if necessary, geological radar and high-resolution direct current method can be used for detection, and **probe** drilling method can be used to verify the prediction conclusion.

In addition, for the coal seam gas, hydrogen sulfide and other harmful gas forecasts according to the tunnel situation select special equipment to carry out the forecast work.

Each type of advance forecasting technology has different characteristics in terms of scope of application, detection distance, and recognition accuracy. According to the geological and geophysical characteristics of the unfavorable geological bodies, the comprehensive advanced geological prediction technical system of the whole process of bad geology can be adopted, which is guided by geological analysis, combined with geology and geophysical exploration, drilling, combining inside and outside the cave, and combining different geophysical methods [8, 9].

The design documents for advanced geological forecast shall be prepared and shall include the following main contents [3]:

1. Tunnel engineering geology and hydrogeological conditions, highlighting the bad geology and special geotechnical, possible major engineering geological problems, and geological risks.
2. Classification of geological complexity.
3. The purpose of advanced geological prediction.
4. The design principle of advanced geological forecast and forecast scheme should (piecewise) forecast the content, method selection, and combination

of the different methods, technical requirements (the same kind of forecast method or overlap between the different forecast method length, hole angle and length, etc.), and in advance, when need should be of meteorological springs, important points and main water hole (flow rate is >1 L/s water point), such as rivers flow observations plan, technical requirement, etc.

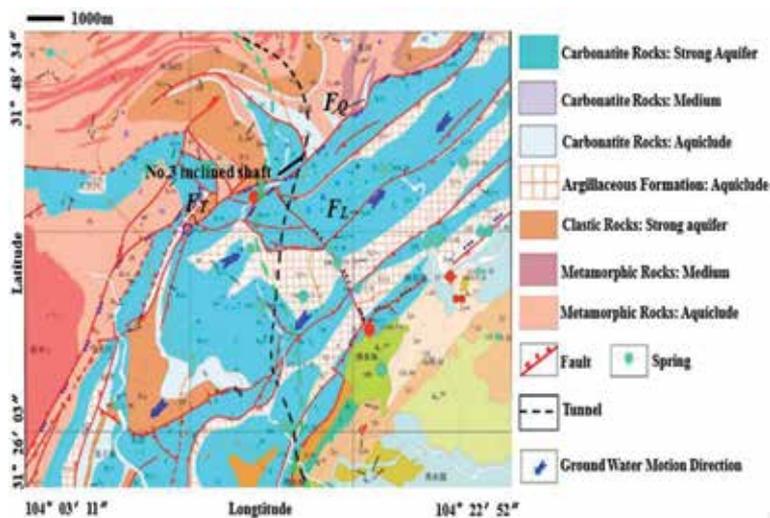
5. Technological requirements for the implementation of advanced geological forecast (if necessary).
6. Safety measures for geological forecast work in advance.
7. The workload of geological forecast in advance and the time occupied by the working face.
8. Budget estimates for advanced geological forecast.
9. Other issues requiring explanation.

### 3. Comprehensive advanced geological prediction design example

#### 3.1 Project overview and hydrogeological analysis

The typical geological disaster prone area of Chenglan railway in western Sichuan is an important part of China's railway "five vertical and five horizontal" planning network. The Yuelongmen tunnel, one of the landmark projects of the Chenglan railway, spans two areas of Anxian and Maoxian. The geological structure at the Longmenshan fault is particularly complex. The maximum depth of the tunnel is 1445 m, the length of the right line is 20,042 m, and the length of the left line is 19,981 m [14–16].

The Yuelongmen tunnel crosses the Longmenshan Central Fault Zone. The tunnel crossing section intersects the mountain range at about 60°. The Yingxiu-Beichuan fault develops multiple secondary faults in the tunnel area. The Guangtongba fault and Gaochuanping fault that the tunnel passes through belong



**Figure 3.**  
*Hydrogeological plan of the tunnel area of Yuelongmen tunnel.*

to its secondary faults. The tunnel also passes through the Qianfoshan fault, the Qianfoshan No. 1 fault, and the Tujiamiao fault. A number of intermountain rivers are developed in the survey area, which is crossing three watersheds. The main surface water in the survey area is intermountain trench water, which is mainly replenished by atmospheric precipitation and partly by bedrock fracture water. The hydrogeological plan of the tunnel area is shown in **Figure 3**.

### **3.2 Tunnel advanced prediction design**

According to the geological survey analysis and comparison of the geological hazard degree classification (**Table 1**), it can be determined that the XJ3K0 + 000–XJ3K0 + 396 segment of the Yuelongmen tunnel should be detected by the class A method of the advanced forecast design, and the seismic wave reflection method (TSP, TRT, etc.) and electromagnetic waves are comprehensively adopted. Methods such as geological radar (GPR, etc.) and electric method (high-resolution direct current method) are used for prediction, and **leading** target drilling is performed to verify the geophysical results.

### **3.3 Implementation of comprehensive geological prediction in typical water-rich section**

#### *3.3.1 Geological analysis of 3# inclined shaft in Yuelongmen tunnel*

The 3# inclined shaft of Yuelongmen tunnel is located at the interface of D2K97 + 700 on the left side of Yuelongmen tunnel. The whole field is 2025 m, and the maximum buried depth is 872 m. The location is a more weathered hilly landform, overlying the Quaternary Holocene alluvial pebble soil, silt-splitting layer silty clay, slope residual coarse breccia, breccia, and block stone. In the lower Devonian system, the Wushan group is a dolomitic limestone, the first subgroup of the Silurian group of the Silurian, the Carboniferous phyllite limestone, and the black carbonaceous slab of the lower Longmaxi group. There are also interbedded with thin-layer siliceous rocks, Ordovician Zhongtong Baota Formation marl, crystalline limestone, Cambrian Qingqing Formation limestone, siltstone, apatite, Sinian Lower Juejiahe Formation Siliceous rocks, shale, carbonaceous shale, limestone, dolomite, and fault breccia. Among them, the XJ3K0 + 396–XJ3K0 + 273 segment is dominated by the Cambrian Qingqing Formation limestone, and the rock is hard, but the karst is moderately strong, the joints are developed, and the surrounding rock is broken. Therefore, when the 3# inclined shaft of Yuelongmen tunnel digs into XJ3K0 + 396, water inrush occurs, and the amount of water inflow is about 1000 m<sup>3</sup>/h.

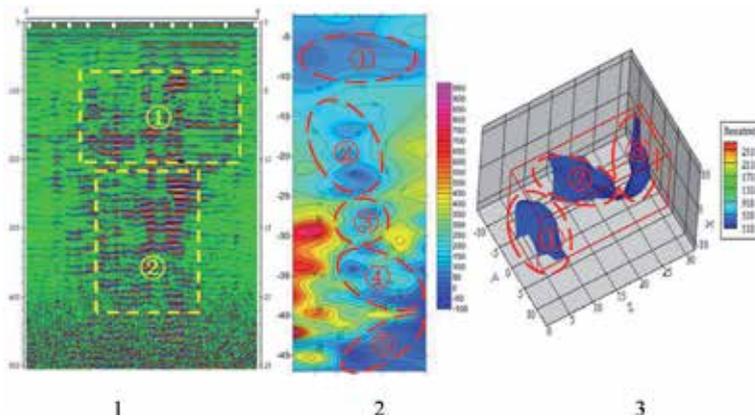
#### *3.3.2 Comprehensive advanced geological prediction analysis*

In order to understand the geological conditions in front of the tunnel face, the TSP method is used to make a large-scale preliminary judgment on the geological body in front of XJ3K0 + 393. The prediction conclusion is that the whole section of XJ3K0 + 393– + 273 is hard but is formed by karst development. The rock is relatively broken, and there is more water between the cracks of XJ3K0 + 393– + 379, XJ3K0 + 374– + 354, and XJ3K0 + 344– + 330. Based on the TSP prediction conclusion and the geological analysis (**Figure 4**), the 3# inclined well fracture of Yuelongmen tunnel is developed and is water-rich. The geological radar method is used to focus on the location, scale, and development of the fracture. The induced polarization method and transient electromagnetic are used. The method carries out detailed detection of the water-rich position and scale of the rock formation.

Comprehensive analysis of TSP, geological radar, transient electromagnetic, and tunnel-induced polarization detection results combined with the geological conditions of the tunnel can be an accurate quantitative judgment of the geologic body in front of the XJ3K0 + 393 face: in general, the front of the face. The surrounding rock within 47 m is generally poor, and the fracture is developed and rich in water, but the water-rich area is uneven. The full section of 0–15 m in front of the face is almost rich in water, while the main middle left side of 16–30 m is rich in water; the 31–47 m side is rich in water in the middle right side. Using the different data of the induced polarization half-life, the estimated static water reserve within 70 m in front of the face is 700 m<sup>3</sup>. In order to further verify the geological and water-rich conditions in front of the face, further advance drilling operations were carried out. Comparing the three detection results, it is found that the three detection methods are consistent with the prediction results of the water body. In particular, the polarization method not only determines the spatial position of the water body but also determines its distribution pattern, which can guide the drilling operation well [17–19].

### 3.3.3 Advanced targeted drilling detection

According to the comprehensive prediction conclusion, four **probe** drill holes were applied at the face of the face to verify the comprehensive geophysical findings. The **probe** drilling position is shown in **Figure 5**.



**Figure 4.** Comparison of detection results. (1) Geological radar detection results; (2) transient electromagnetic detection results; (3) excited excitation detection result (hole body range extraction map).



**Figure 5.** Position of the lead drilling hole.

The verification of the on-site lead drilling indicates that the forecast results are in good agreement with the drilling results and the water-bearing structures predicted in the forecast conclusions are also verified by the disclosure. The forecast results also optimize the location and quantity of targeted drilling, which not only reduces the number of boreholes for the construction unit but also effectively covers the detection of unfavorable geology.

### *3.3.4 Excavation result verification*

During the excavation of the tunnel, XJ3K0 + 393–XJ3K0 + 378 paragraph, the water surge appeared in the middle and lower part of the face, and a certain depth of water appeared in the bottom plate. In the XJ3K0 + 377–XJ3K0 + 368 paragraph, there is water in the left and middle of the face, and there is water in the XJ3K0 + 367–XJ3K0 + 363 section. The water inflow has an increasing trend (**Figures 6 and 7**). The distribution of water content in rock mass is almost always consistent with comprehensive advanced geological prediction, which confirms the accuracy of comprehensive advanced geological prediction.

### **3.4 Tunnel construction measures**

The results of comprehensive geophysical exploration combined with geological analysis and advanced targeted drilling show that there are no large caves or karst pipelines within the effective geophysical range and the main unfavorable



**Figure 6.**  
*Water inrush in the tunnel after the advanced drilling.*



**Figure 7.**  
*Water inrush after tunnel excavation.*

geology is dominated by bedrock fissure water. The geological integrity of the surrounding rock in front of the face is poor; the bedrock fissure is relatively developed and rich in water. Therefore, according to the requirements of tunnel construction safety, quality, schedule, and environmental protection, the Yuelongmen tunnel 3#, the sloped limestone water-rich section changed the original “full-section curtain grouting” to “advanced peripheral grouting” treatment measures, which not only saves time and effort but also speeds up the progress of the project.

Through the implementation of **leading** peripheral grouting measures for the 3# inclined well water-rich section of Yuelongmen tunnel, the risk of inrush water was effectively reduced, and remarkable results were obtained, ensuring the safe and orderly construction of the tunnel.

#### **4. Development and prospect of advanced geological prediction technology**

At present, the location, scale, and distribution of most unfavorable geological bodies have been detected by corresponding technologies, but it is still impossible to determine the rock mechanic properties of unfavorable geological bodies, the detection range is mostly limited to two-dimensional plane, the detection effect is not ideal in complex environment with large disturbance, the construction area is occupied and the time-consuming is longer, etc., which are still urgent problems to be solved in the field of advanced forecasting and detection. In addition, the development direction of future advanced geological prediction technology is the ultra-long **leading** horizontal drilling technology, the advanced geological prediction method while drilling, the fine imaging technology of multi-physical field information joint inversion, the quantitative technology of advanced prediction, the real-time advanced prediction technology, and the intelligent interpretation technology of advanced prediction [20–23]. At the same time, in order to obtain more accurate quantitative information in the tunnel, it is necessary to combine advanced geological prediction and surface exploration technology; use aviation electromagnetic exploration technology and system and intelligent geologic deep drilling; adapt to complex environmental geophysical techniques as breakthrough points; and form the comprehensive exploration and prediction technology that includes space-borne remote sensing, airborne exploration, and surface geophysical exploration and intelligent drilling. These could improve the ability of obtaining geological information and provide guidance for safe and efficient tunnel construction.

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Section 2

# Analysis and Design of Tunnels

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# Topics of Analytical and Computational Methods in Tunnel Engineering

*Michael G. Sakellariou*

## Abstract

In this chapter, a selection of tunneling topics is presented, following the evolution of methods and tools from analytical to computational era. After an introductory discussion of the importance of elasticity and plasticity in tunneling, some practical topics are presented as paradigms to show the successful application of them in achieving a solution. The circular and horseshoe tunnel sections served as the basis of the elastic analysis of deep tunnels. Practical aspects such as influence zone and elastic convergences in both cases are examined. In the case of circular tunnels, the estimation of plastic zone formation is discussed for a selection of strength criteria. After a detailed discussion of the influence of surface proximity, the elastic and plastic analysis of shallow tunnels is examined in some detail. The presentation is completed by a short presentation of computational methods. An overview of recent developments and a classification of the methods are presented, and then some problems for the case of anisotropic rocks have been presented using finite element method (FEM). The last topic is the application of artificial intelligence (AI) tools in interpreting data and in estimating the relative importance of parameters involved in the problem of tunneling-induced surface settlements. In the conclusions a short discussion of the main topics presented follows.

**Keywords:** elasticity, plasticity, deep tunnels, shallow tunnels, influence zone, plastic zone, circular tunnels, horseshoe tunnels, computational methods, ANN, surface settlements

## 1. Introduction

Underground work history goes back to prehistoric times [1]. The oldest known tunnel is the one underpassing the River Euphrates in Babylon constructed 4000 years ago. Hezekiah, King of Judea, built a tunnel 2700 years ago, whereas Eupalinos built the Eupalinion tunnel constructed in Samos Island, Greece, 2600 years ago, both for water supply purposes. For further information see [2–5]. Shelters, underground works for warfare purposes, mineral resource exploitation, traffic tunnels and conveyance tunnels like water supply tunnels, etc. are the main categories of underground structures [3].

Tunnel engineering could be characterized as an art, not fine art of course, with the meaning of know-how based on past experience, lessons learned from tunnel disasters [6], knowledge of geological conditions and behaviour of the ground,

innovations to overcome encountered difficulties, scientific knowledge from applied mechanics and advances in mechanized excavation tools and methods. In recent years, the experience from South Africa and North Europe rock engineering practice resulted in the introduction of comprehensive classification systems, namely, RMR [7, 8] and Q system [9]. In theory, Lamé in 1852 and Kirsch in 1898 [10] paved the road of scientific development of solutions to the problem of stresses and strains around underground holes [10]. Terzaghi [11] examined shallow tunneling through sands and cohesive soils. For a comprehensive presentation of the fundamentals of theoretical rock mechanics, see Jaeger et al. [12]. For a detailed coverage of underground excavations in rock, see Hoek and Brown [13]. Experimental methods such as photoelasticity [14, 15] and Moiré [16] and computational methods such as finite elements [17], finite difference, direct and indirect boundary elements [13, 18], distinct elements [19] and meshfree methods [20, 21] have been developed and extensively applied successfully. On the other hand, the progress regarding the material behaviour resulted in advanced constitutive laws [22] and in the more realistic modeling of continua and discontinua media [23]. Finally, the information systems and in particular the artificial intelligence offered new tools like artificial neural networks (ANN) and other machine learning methods.

## 2. Analytical methods: elastic problems

### 2.1 The case of deep tunnels

Deep tunnels can be considered as the limiting case of a cylinder with thick walls when the radius of the exterior wall tends to infinity. In that case the ground surface has no any influence on the stress field around the opening.

Jeffery [24] solved the problem of stress distribution inside the walls of a cylinder bounded by two non-concentric circles, with the distance of their centres denoted by  $d$ . In this way, Jeffery's approach solves the problems of an infinite plate containing a circular hole as the limit of a thick cylinder with concentric boundaries ( $d = 0$ ) and the problem of a semi-infinite plate containing a circular hole as a particular case of two eccentric boundaries. So, he gave the general solution of deep and shallow tunnels, respectively, as particular cases of a general problem.

For a historical account of the mathematical solutions of stress distribution around underground openings, see Gerçek [25].

#### 2.1.1 Cyclic section

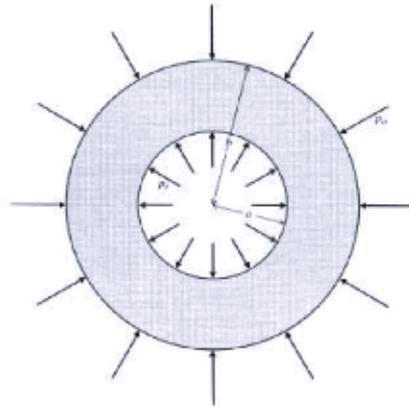
Assuming a homogeneous isotropic linear elastic medium (HILE medium) and considering the geometry of the problem and the equations of equilibrium, we obtain the following set of equations (Eqs. (1)–(4)) (**Figure 1**):

$$\sigma_r = -\frac{2G}{1-2\nu} \left\{ \frac{A}{2} - (1-2\nu) \frac{B}{r^2} \right\}, \Rightarrow \sigma_r = C - \frac{D}{r^2}, \quad (1)$$

$$\sigma_\theta = -\frac{2G}{1-2\nu} \left\{ \frac{A}{2} + (1-2\nu) \frac{B}{r^2} \right\}, \Rightarrow \sigma_\theta = C + \frac{D}{r^2}, \quad (2)$$

$$\tau_{r\theta} = 0 \quad (3)$$

$$u_r = -\frac{1}{2G} \left\{ (1-2\nu)Cr + \frac{D}{r} \right\} \quad (4)$$



**Figure 1.**  
 The cylinder with thick walls.

The constants C and D are defined as:

$$C = -\frac{GA}{1-2\nu},$$

$$D = -2GB$$

Considering the boundary conditions:

$$\text{for } r = a, \sigma_r = p_a.$$

$$r = b, \sigma_r = p_b.$$

therefore:

$$p_a = C - \frac{D}{a^2},$$

$$p_b = C - \frac{D}{b^2}$$

$$D = \frac{(p_b - p_a)a^2b^2}{(b^2 - a^2)},$$

$$C = \frac{(b^2p_b - a^2p_a)}{(b^2 - a^2)}$$

When b tends to infinity, we obtain the special case of a cyclic opening excavated in an infinite medium under hydrostatic stress field p, by superposing the initial stresses induced by the excavation:

$$\sigma_r = p \left( 1 - \frac{a^2}{r^2} \right),$$

$$\sigma_\theta = p \left( 1 + \frac{a^2}{r^2} \right), \tag{5}$$

$$\tau_{r\theta} = 0,$$

$$u_r = -\frac{1}{2G} \frac{a^2}{r} p$$

For the general case of stress field, denoting the ratio of horizontal to vertical in situ principal stress with k, Kirsch [10] obtained the following equations:

$$\begin{aligned}
 \sigma_r &= \frac{p}{2} \left[ (1+k) \left( 1 - \frac{a^2}{r^2} \right) - (1-k) \left( 1 - 4 \cdot \frac{a^2}{r^2} + 3 \cdot \frac{a^4}{r^4} \right) \cos 2\theta \right], \\
 \sigma_\theta &= \frac{p}{2} \left[ (1+k) \left( 1 + \frac{a^2}{r^2} \right) + (1-k) \left( 1 + 3 \cdot \frac{a^4}{r^4} \right) \cos 2\theta \right], \\
 \tau_{r\theta} &= \frac{p}{2} (1-k) \left( 1 + 2 \cdot \frac{a^2}{r^2} - 3 \cdot \frac{a^4}{r^4} \right) \sin 2\theta, \\
 u_r &= -\frac{p a^2}{4Gr} \left[ (1+k) - (1-k) \left\{ 4(1-\nu) + \frac{a^2}{r^2} \right\} \right] \cos 2\theta, \\
 u_\theta &= -\frac{p a^2}{4Gr} (1-k) \left\{ 2(1-2\nu) + \frac{a^2}{r^2} \right\} \sin 2\theta,
 \end{aligned} \tag{6}$$

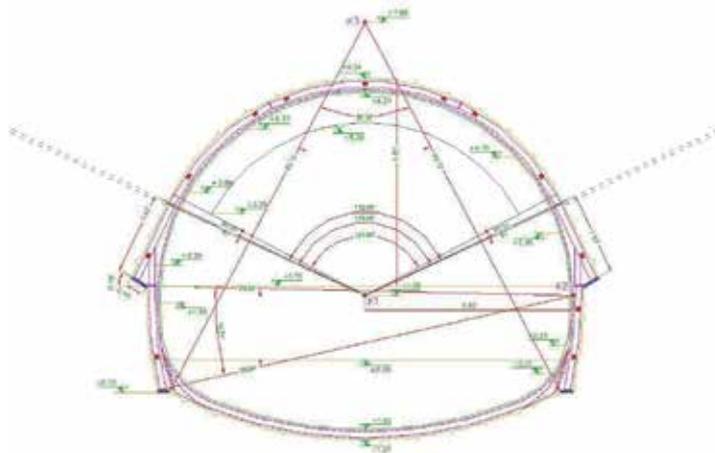
It is interesting to note that in the case of hydrostatic or isotropic stress field ( $k = 1$ ), we can estimate an influence zone accepting a  $\pm 0.04\%$  deviation from the initial stress field. Then by introducing this condition in the above equations, we obtain the radius of influence zone as  $5\alpha$ , where  $\alpha$  is the radius of the excavation.

### 2.1.2 The case of horseshoe section

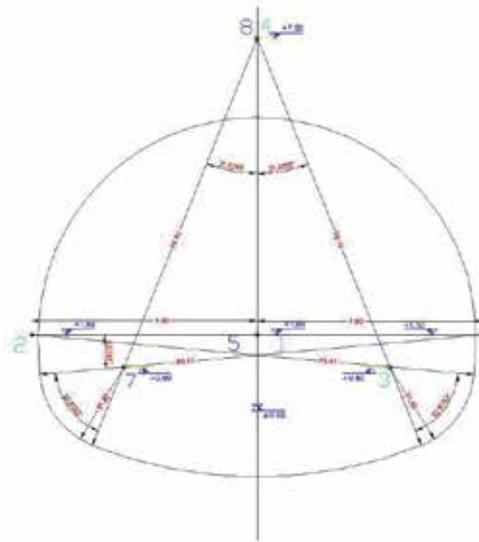
Let us examine now the case of horseshoe section (**Figures 2 and 3**). This problem is very useful in the case of excavations without using a tunnel boring machine (TBM). **Figure 2** shows a typical metro section, whereas **Figure 3** shows the simplified section used in the analysis.

First, the maximum principal stress  $\sigma_1$  contours obtained by a numerical analysis are shown **Figure 4**. A slight influence of the asymmetric shape of the tunnel's section can be observed. Schürch and Anagnostou [26] examined the influence of rotational symmetry violation on the applicability of the ground response curve.

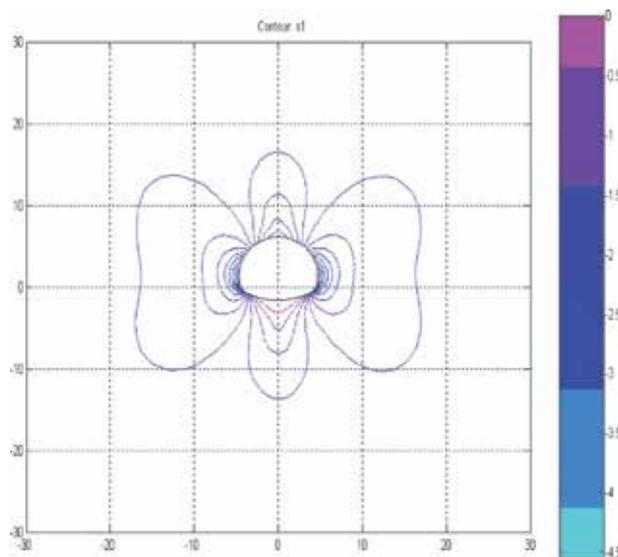
To obtain a solution to this problem, we adopted Muskhelishvili approach, using functions of complex variables and conformal mapping techniques [27].



**Figure 2.**  
Typical horseshoe tunnel section.



**Figure 3.**  
 A simplified horseshoe tunnel section.



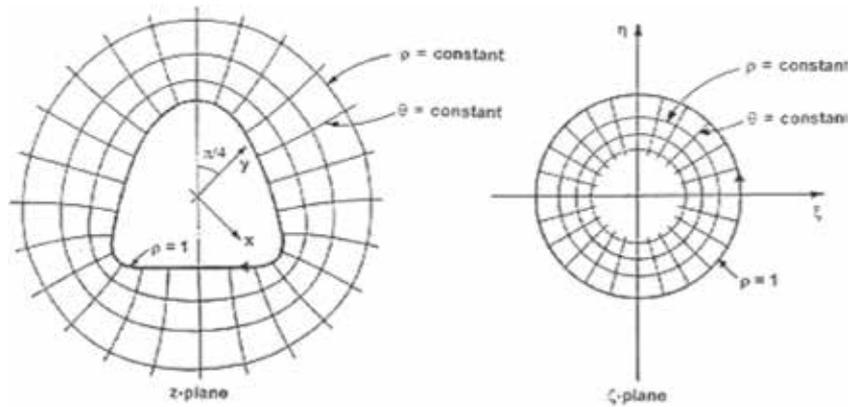
**Figure 4.**  
 Maximum principal stress contours around a deep horseshoe tunnel section.

Adopting Gerçek solution [28], we introduce the following mapping function, which transforms the infinite region surrounding the opening onto the interior of the unit circle (Figure 5):

$$z = \omega(\zeta) = R \left( \frac{1}{\zeta} + \sum_{k=1}^3 a_k \cdot \zeta^k \right) \quad (7)$$

In Eq. (7) R is a real constant, and the complex coefficients  $\alpha_k$  are defined as:

$$a_k = \alpha_k + b_k i, \text{ for } k = 1, 2, 3 \quad (8)$$



**Figure 5.** Conformal mapping of an infinite region surrounding a hole onto the unit circle [28].



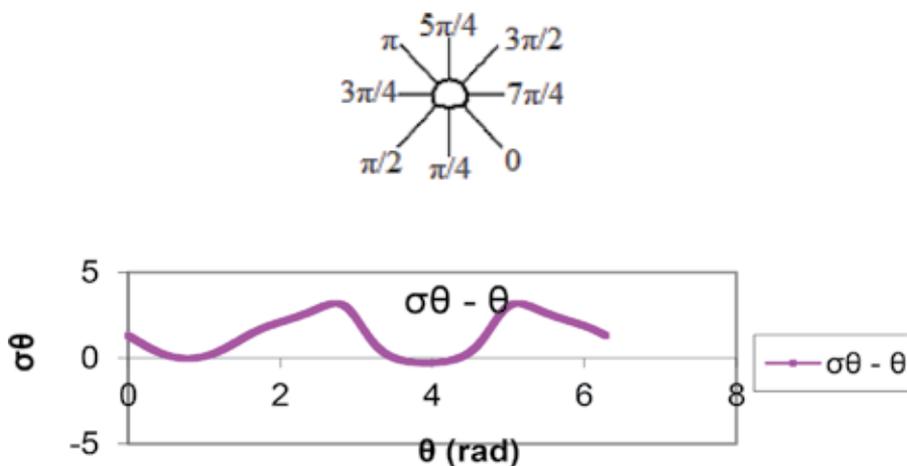
**Figure 6.** The final section that best fits the initial horseshoe section [29, 30].

In order to find the optimum values of the above coefficients to fit best the given horseshoe section, a (123x3) system of equations has been solved [29, 30]. The values of the coefficients are:

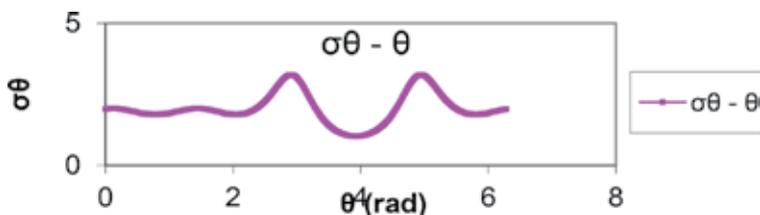
$$\begin{aligned}
 \alpha_1 &= b_3 = 0 \\
 \alpha_2 &= b_2 = 0.047, \\
 \alpha_3 &= 0.029, \\
 b_1 &= 0.104, \\
 R &= 3.756
 \end{aligned}
 \tag{9}$$

In [29, 30] an extended investigation of this problem is presented resulting in the calculation of the set of coefficients for 49 sections. In **Figure 6**, the final section, which is the best for our problem, is presented superimposed to the original horseshoe shape.

Now, we can calculate the stresses around the opening as well as the strains for initial stress fields with  $k = 0, 0.333, 1$  and  $3$  [29, 30]. In **Figures 7** and **8**, the variation of  $\sigma_\theta$  at the boundary of the excavation is shown for  $k = 0.333$  (**Figure 7**) and  $k = 1$  (**Figure 8**).



**Figure 7.**  
 Hoop stress  $\sigma_\theta$  variation along the boundary of the excavation for  $k = 0.333$  [29].



**Figure 8.**  
 Hoop stress  $\sigma_\theta$  variation along the boundary of the excavation for  $k = 1$  [30].

Initial stress field	Convergences of circular section [mm]	Horseshoe-induced roof convergences [mm]	Horseshoe-induced bottom convergences [mm]
$k = 1$	7.8	7.6	8.2
$k = 0.333$	7.1	7.0	7.9
$k = 1$	5.6	5.9	7.2
$k = 3$	1.1	2.5	5.0

**Table 1.**  
 Convergences of circular and horseshoe sections for different stress fields [29, 30].

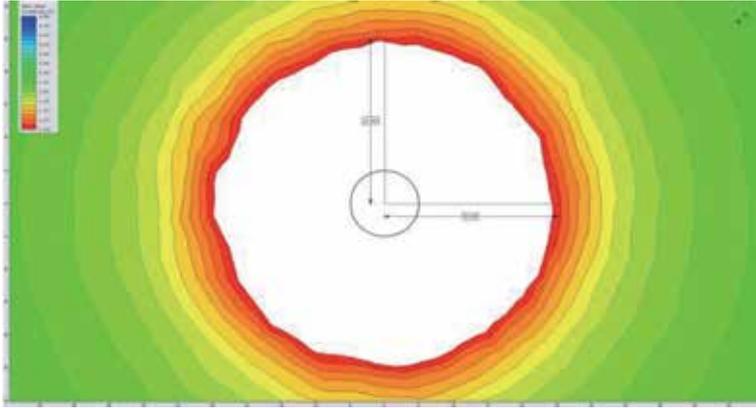
A further result of practical significance obtained by this method is the calculation of the convergence of the excavation boundaries and the extent of the zone of influence. In **Tables 1** and **2**, these values are presented in comparison with the corresponding values for the circular section. We can observe the greater deviation at the bottom of the horseshoe section because of the greater radius of this segment compared to the circle.

In all cases presented in this paragraph, we assumed the initial stress field  $p = 1$  MPa, modulus of elasticity  $E = 1$  GPa and Poisson ratio  $\nu = 0.3$ . Using these values we are able to calculate the corresponding values for any  $p$  value and every kind of rock. The only constraint is that the value of Poisson ratio is equal to 0.3. In the following paragraph, we shall discuss the influence of  $\nu$  upon the stresses.

Initial stress field	Influence zone (horseshoe section)	Influence zone (circular section)
$k = 0$	8.6R	7.5R
$k = 0.333$	7.8R	7R
$k = 1$	5.6R	5R
$k = 3$	7.4R	7R

*R is the radius of equivalent circular section [29, 30].*

**Table 2.**  
Influence zone extends around circular and horseshoe sections for different stress fields.



**Figure 9.**  
Stress distribution around a circular deep tunnel ( $Z = 25R, k = 1$ ) [32].

## 2.2 The case of shallow tunnels: the circular section

Assuming that the tunnel is close to the ground surface, we can observe an influence of the boundary as the vertical stresses depend on the depth. So, the initial stresses at the roof and the bottom levels of the tunnel section are not equal. Bray [31] presented an extensive study on this problem. Some practical rules are summarized below.

By assuming an acceptable deviation, as in the case of the influence zone estimation, we may distinct three cases.

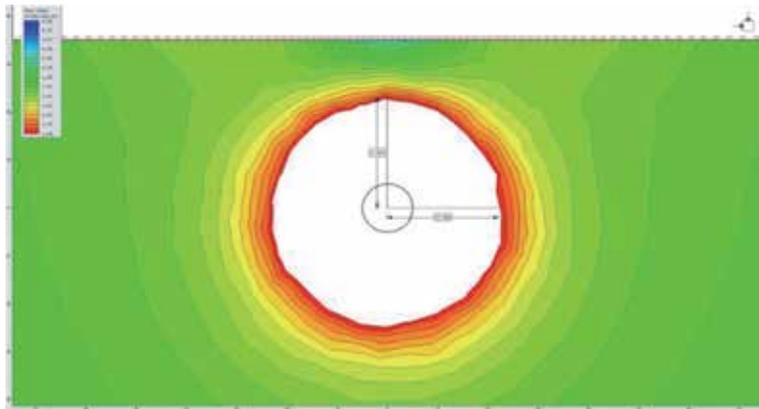
### 2.2.1 Case I

The tunnel depth, measured from the section's centre, is  $Z > 25R$ , where R is the tunnel radius. In this case Kirsch equations apply. In **Figure 9**, the distribution of stresses around a circular deep tunnel is shown [32]. There is no influence of the boundaries. Note that only a window of the stress field is shown.

### 2.2.2 Case II

The tunnel depth is  $7R < Z < 25R$ . In this case we must add a correction term to the Kirsch equations depending on the Poisson ratio. That term is given by Bray as [31]:

$$\sigma_{\theta} = \gamma h [1 + k + 2(1 - k) \cos 2\theta] - \gamma \alpha \sin \theta \left[ \frac{3 - 4\nu}{2(1 - \nu)} + 2(1 - k) \cos 2\theta \right] \quad (10)$$



**Figure 10.**  
 Stress distribution around a circular shallow tunnel ( $Z = 7R$ ,  $k = 1$ ) [32].

where  $\alpha$  is the tunnel's radius. This expression agrees with an equation given by Savin [33].

In **Figure 10**, the distribution of stresses around a circular shallow tunnel is shown. The influence of the surface boundary starts to be noticeable [32].

### 2.2.3 Case III

The tunnel depth is  $Z < 7R$ . This case is more difficult because the influence of the boundary becomes greater. Then Mindlin's closed-form solutions apply [34, 35]. Mindlin obtained solutions to this problem for three cases of in situ stress fields at depth  $z$  (remote from the tunnel):

#### 2.2.3.1 Case I

$$p_z = wz.$$

$$p_h = wz.$$

i.e. isotropic, or hydrostatic, gravitational pressure and  $w$  being the unit weight of mass.

#### 2.2.3.2 Case II

$$p_z = wz.$$

$$p_h = \{\nu/(1-\nu)\}wz.$$

This is the case where the lateral deformation is a constraint remote from the tunnel.

#### 2.2.3.3 Case III

$$p_z = wz.$$

$$p_h = 0.$$

This is the case of nonlateral constraint of the mass remote from the tunnel.

The solutions of Mindlin were in terms of bipolar coordinates  $\alpha$  and  $\beta$ . The expression giving the stress on the circular boundary for the first case is [35]:

$$[\sigma_\beta]_{\alpha=\alpha_1} = \frac{2wA(\cosh \alpha_1 - \cos \beta)}{\sinh \alpha_1} \left\{ \frac{1 - \cosh \alpha_1 \cos \beta}{(\cosh \alpha_1 - \cos \beta)^2} - \coth \alpha_1 - \frac{(7 - 8\nu)}{4(1 - \nu)} \frac{\cos \beta}{\sinh \alpha_1} + 2e^{-\alpha_1} \cos \beta \sum_{n=2}^{\infty} R_n \cos n\beta \right\} \quad (11)$$

where  $\alpha_1$  is the value of  $\alpha$  corresponding to the boundary of the tunnel:

$$\begin{aligned} \frac{Z}{R} &= \cosh \alpha_1 \\ A &= Z \tanh \alpha_1 = R \sinh \alpha_1 \\ R_n &= N_n - ne^{-n\alpha_1} \\ N_n &= \frac{ne^{-n\alpha_1}(\sinh n\alpha_1 \cosh n\alpha_1 - n \sinh \alpha_1 \cosh \alpha_1)}{\sinh^2 n\alpha_1 - n^2 \sinh^2 \alpha_1} \end{aligned} \quad (12)$$

Poulos and Davis [35] gave in figures and tables values of  $\sigma_\beta$  for  $1 < (Z/R) < 4$ .

For Cases II and III above, the solution for  $\sigma_\beta$  is obtained by adding in the solution given by Eq. (11) a further expression [33]. From the above analysis, the importance of taking into account the influence of the ground surface on the stress distribution on tunnel boundary is obvious. A further important notice is the influence of Poisson ratio on the expressions of stresses. From Eq. (10), it is clear that this influence is important when  $Z < 25R$ .

Malvern [36] in a rigorous analysis concluded that when the resultant force on each boundary is zero, then the stress distribution is independent of the elastic constants. Otherwise the stress distribution will depend on  $\nu$ . When the boundary conditions include displacement constraints, then the stress distribution will, also, depend on  $\nu$ . These conclusions are important to be known for the stress analysis of tunnels by using computational methods like finite elements or boundary elements. Frocht [14] examined the influence of stresses from  $\nu$  by extensive photoelastic experimental study. In this short presentation of the elastic solutions for the shallow circular tunnel, we restricted to the stress field influence. For an extensive presentation of elastic solutions for the important problem of surface settlements induced by tunneling, which is of great practical significance, see [37–39]. Another important topic in the case of shallow tunnels is the problem of stress field due to seismic loading. Recently, Pelli and Sofianos [40] published a paper addressing the very important topic of the stress field around shallow tunnels under seismic loading of SW waves.

### 3. Analytical methods: plastic problems

#### 3.1 The case of deep tunnels

We now turn our interest in plastic problems. This is a case of great theoretical and practical significance, as every underground excavation must be analysed against failure. For a detailed coverage, see, almost, every standard textbook of soil and rock mechanics. For a more detailed coverage, see [41–44]. Here, we examine

special topics related to tunnel engineering. In order to analyse the competence of an excavation in soil or rock, we must extend our analysis beyond elasticity to the plastic behaviour of the surrounding medium. To this end, we have to introduce to our analysis a failure criterion. The most common criterion is the Mohr-Coulomb [11]. Extensive research resulted in a remarkable advancement in understanding the material behaviour and the implementation of advanced models in computational methods. A milestone was the introduction of critical state theory in the 1960s [45]. Further research in rock mechanics resulted in the introduction of the Hoek-Brown criterion, which is widely used in rock engineering practice [13]. Yu [46], in an extensive review, presented a wide range of failure criteria applicable on, almost, every kind of materials under every possible type of loading. In this review Yu discussed as well the soil and rock materials.

Let us examine the problem of practical significance of the formation of plastic zone around a deep tunnel of circular section. Closed-form solutions can be found for every criterion of soil and rock literature in the case of isotropic or hydrostatic stress field ( $k = 1$ ), because of the axisymmetric type of this case. Recently, Vrakas and Anagnostou presented an extension of the small strain analyzes to obtain finite strain solutions [47]. For the general case of stress fields ( $k \neq 1$ ), closed-form solutions have been obtained so far for the Tresca criterion [48] and for the Mohr-Coulomb criterion [49]. Here, the elliptic paraboloid criterion developed by Theocaris [50, 51] is introduced to solve the problem of plastic zone around a circular deep tunnel in rock. First, the mathematical expression of the criterion is given, and a comparison of it with Griffith and Hoek-Brown criteria is presented.

This criterion is expressed through the three principal stresses. It is an energy criterion having as parameter the absolute value of the ratio  $R$  of uniaxial compressive strength over the uniaxial tensile strength. The elliptic paraboloid criterion is developed for applications in applied mechanics in general, so the tensile stresses are positive, and the compressive stresses are negative. Then:

$$(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 + 2(R - 1)(\sigma_1 + \sigma_2 + \sigma_3) \sigma_t = 2 R \sigma_t^2 \quad (13)$$

$$\text{where } R = \frac{|\sigma_c|}{\sigma_t}.$$

Eq. (13) is the addition of two components, which express two parts of the total elastic energy. Indeed, the first part is, up to a constant multiplicative factor, the distortion energy expressed through the deviatoric stresses. This part represents the energy of the elastic change of shape [52] given by Eq. (14):

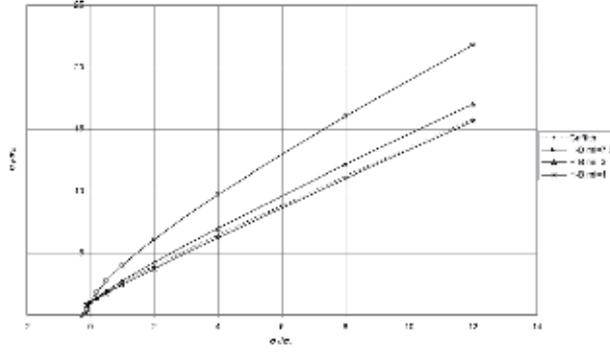
$$(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 \quad (14)$$

The second part depends on the hydrostatic or spherical part of the stresses and represents the energy of elastic change in volume. In this way, the elliptic paraboloid criterion combines both the change of shape and volume. The latter is important for soil and rock materials as their strength depends on the confining pressure.

For the isotropic case of field stress, conditions of axisymmetry are valid, i.e.  $\sigma_1 = \sigma_2$ . Then from Eq. (13), we obtain:

$$(\sigma_3 - \sigma_1)^2 + \frac{R - 1}{R} (2\sigma_1 + \sigma_3) \sigma_c - \frac{\sigma_c^2}{R} = 0 \quad (15)$$

In the case of axisymmetric conditions, the criterion becomes paraboloid of revolution. In **Figure 11**, a comparison of Griffith criterion with Hoek-Brown criterion being shown. It is obvious that the deviation between their representation



**Figure 11.** Comparison of Griffith criterion with Hoek-Brown criterion for different  $m$  values [52].

for  $m$  value being equal to 7.88, close to the restriction of Griffith's corresponding value being 8 [52].

Now, we can proceed to a comparison of elliptic paraboloid criterion with Griffith. We assume an  $R$  value of 8 (**Figure 12**).

From the above figures, we may conclude that Griffith and elliptic paraboloid criteria agree, although their assumptions are completely different. Griffith assumed that failure initiates from pre-existed cracks because of the stress concentration at the tips of the cracks. This assumption is fundamental in fracture mechanics. On the other hand, the assumption of elliptic paraboloid criterion is an extension of von Mises criterion. Both criteria are identical in the case of equal strengths under compression and tension ( $R = 1$ ). This assumption is close to the experimental results for metals. Theocaris applied this criterion in igneous and metamorphic rocks [45].

We, now, come to examine the problem of plastic zone formation around a deep tunnel of circular section under isotropic stress field ( $k = 1$ ) [53] (**Figure 13**).

To solve the problem, we introduce the equation of equilibrium in polar coordinates (Eq. (16)) and the flow rule (Eq. (17)). Then, by introducing the expression of elliptic paraboloid criterion in Eq. (17), we obtain Eq. (18) [53]:

$$\frac{d\sigma_r}{dr} + \frac{\sigma_r - \sigma_\theta}{r} = 0 \quad (16)$$

$$\frac{\partial f}{\partial \sigma_2} = 0 \quad (17)$$

$$\frac{\partial \left[ (\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 + 2 \cdot (R_b - 1) \cdot (\sigma_1 + \sigma_2 + \sigma_3) \cdot \sigma_{tb} - 2 \cdot R_b \cdot \sigma_{tb}^2 \right]}{\partial \sigma_2} = 0 \quad (18)$$

After complicated algebraic manipulations, we obtain the condition for plastic zone formation around the tunnel (Eq. (19)) [53]:

$$p_o < \frac{(R_i - 1) \cdot \sigma_{ti}}{2} - p_i - \sqrt{-(R_i - 1) \cdot \sigma_{ti} \cdot p_i + \frac{1}{3} \cdot R_i \cdot \sigma_{ti}^2 + \frac{(R_i - 1)^2 \cdot \sigma_{ti}^2}{3}} \quad (19)$$

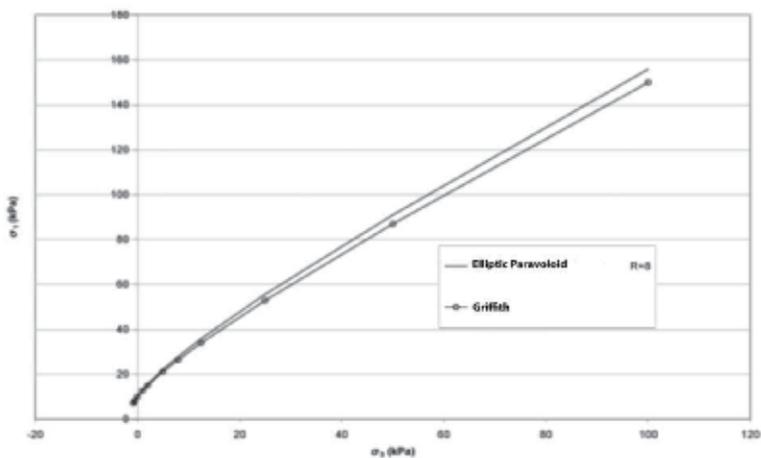
Finally, the radius of the plastic zone  $r_c$  is obtained (Eq. (20)) [53]:

$$r_e = r_i \cdot e^{\frac{L-K}{2(R_b-1)\sigma_{tb}} - \frac{1}{2} \ln \frac{K}{L}} \quad (20)$$

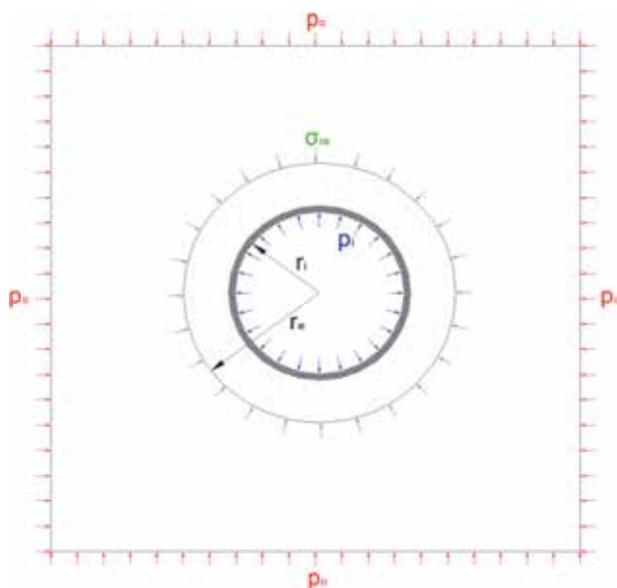
L and K above are constants given by Eq. (21), (22):

$$K = -(R_b - 1) \cdot \sigma_{tb} - \sqrt{\frac{4}{3} \cdot R_b \cdot \sigma_{tb}^2 - 4 \cdot (R_b - 1) \cdot \sigma_{tb} \cdot \sigma_{re} + \frac{4 \cdot (R_b - 1)^2 \cdot \sigma_{tb}^2}{3}} \quad (21)$$

$$L = -(R_b - 1) \cdot \sigma_{tb} - \sqrt{\frac{4}{3} \cdot R_b \cdot \sigma_{tb}^2 - 4 \cdot (R_b - 1) \cdot \sigma_{tb} \cdot p_i + \frac{4 \cdot (R_b - 1)^2 \cdot \sigma_{tb}^2}{3}} \quad (22)$$



**Figure 12.** Comparison of elliptic paraboloid and Griffith criteria ( $R = 8$ ) [52].



**Figure 13.** The geometry of the problem. The plastic zone around the circular tunnel is shown [53].

Hoek and Brown [9] obtained the radius of the plastic zone for Hoek-Brown criterion as follows [13]:

$$r_e = r_i e^{\left\{ N - \frac{2}{m_r \sigma_c} (m_r \sigma_c p_0 + s_r \sigma_c^2)^{\frac{1}{2}} \right\}} \quad (23)$$

$$N = \frac{2}{m_r \sigma_c} (m_r \sigma_c p_0 + s_r \sigma_c^2 - m_r \sigma_c^2 M) \quad (24)$$

$$M = \frac{1}{2} \left( \left( \frac{m}{4} \right)^2 + m \frac{p_0}{\sigma_c} + s \right)^{\frac{1}{2}} - \frac{m}{8} \quad (25)$$

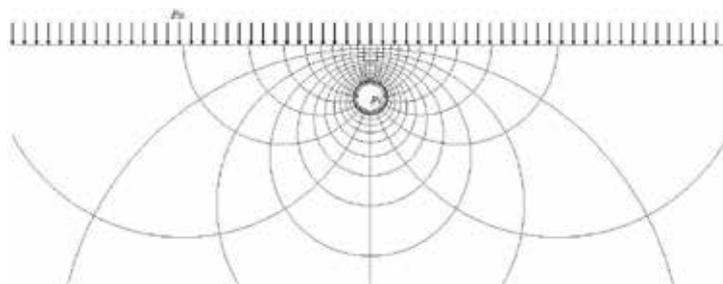
In **Table 3** a comparison of the above criteria is shown. We can conclude that, except for low values of in situ stresses, both criteria are close in their predictions of radial stresses at the elastic–plastic interface. On the contrary, their predictions regarding the extent of the plastic zone differ with Hoek-Brown criterion being somehow more conservative.

### 3.2 The shallow tunnel case

The shallow tunnel case is complicated because of the influence of the proximity of ground surface on the stress field and the influence of gravity. As we notice for the case of deep tunnels, there are closed-form solutions for the plastic zone formation around circular tunnels for isotropic stress field and for the general case of Tresca [48] and Mohr-Coulomb [49] criteria. For the case of shallow tunnels, there were no closed-form solutions until 2009 when a solution was published for the Mohr-Coulomb solution under the assumption of isotropic stress field ( $k = 1$ ) and no gravitational stresses, based on bipolar coordinates (**Figure 14**).

Elliptic paraboloid			Hoek-Brown	
$P_0$ (MPa)	$\sigma_{re}$ (MPa)	$r_e$ (m)	$\sigma_{re}$ (MPa)	$r_e$ (m)
4	2.10	10.25	1.96	13.45
3	1.35	8.29	1.31	11.22
2	0.64	6.55	0.71	9.07
1.5	0.31	5.76	0.44	7.99
1	0.01	5.02	0.20	6.84

**Table 3.** Comparison of elliptic paraboloid and Hoek-Brown criteria for  $\sigma_c = 1$  MPa,  $\sigma_t = 0.125$  MPa,  $R = 8.00$ ,  $m = 8$  and  $s = 1$  [53].



**Figure 14.** Bipolar coordinates used for the solution of shallow tunnel problem [54].

In the following the solution published in [54] and further applied [55] is presented omitting the detailed mathematical analysis:

$$P_{cr} = \frac{2\kappa^2}{2(d_i^2 - r_i^2 \cos^2 \beta) + \kappa^2(\lambda - 1)} \left[ P_0 \left( \frac{d_i^2 - r_i^2 \cos^2 \beta}{\kappa^2} \right) - \frac{Y}{2} \right] \quad (26)$$

$$= \frac{2\kappa^2}{2(\kappa^2 + r_i^2 \sin^2 \beta) + \kappa^2(\lambda - 1)} \left[ P_0 \left( \frac{\kappa^2 + r_i^2 \sin^2 \beta}{\kappa^2} \right) - \frac{Y}{2} \right]$$

where  $r_i$  is the tunnel radius and  $d_i (= \kappa \coth \alpha_i)$  is the depth of the tunnel axis from the surface. Considering the conditions of the problem, following [54] the final form of the equation giving the shape of plastic zone is given below:

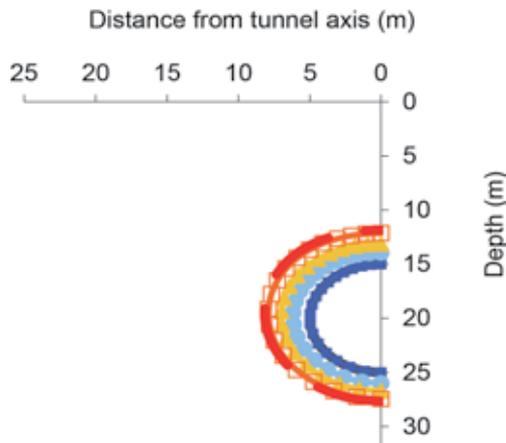
$$\left( \frac{r_c d_i - r_i \cos \beta}{r_i d_c - r_c \cos \beta} \right)^{1-\lambda} = \frac{[2M_0 + \kappa^2(\lambda - 1)][Y + P_i(\lambda - 1)]}{2M_0[Y + P_0(\lambda - 1)]} \quad (27)$$

where  $M_0 = \kappa^2 + r_c^2 \sin^2 \beta$ .

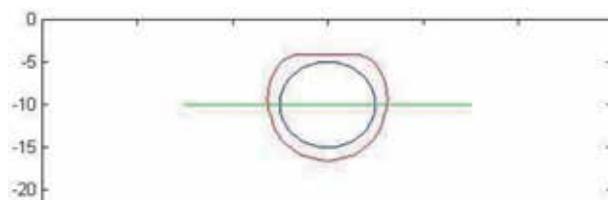
In **Figure 15**, a parametric study of the plastic zone shape is shown [54].

Eq. (27) can be solved, also, by using MATLAB [56]. In **Figure 16**, an example is shown.

The analytical solution of this problem apart from its theoretical interest could be used in conjunction with computational analysis in practical problems [57]. In [55] a very important case is presented where the task was for a new tunnel to



**Figure 15.** Plastic zone formation for different  $P_i/P_0$  values for  $P_0 = 500$  kPa,  $c = 100$  kPa,  $\varphi = 25^\circ$ , tunnel's depth = 20 m and  $r = 5$  m [54].



**Figure 16.** Plastic zone around shallow tunnel with MATLAB. Example SMG1 [54]:  $D = 10$  m,  $r = 5$  m,  $\gamma = 25$  kN/m<sup>3</sup>,  $P_0 = 250$  kPa,  $P_i = 50$  kPa,  $c = 60$  kPa,  $\varphi = 25^\circ$ .

underpass the monumental Chandpole Gate in Jaipur, India. From the closed-form solution, the critical internal pressure was calculated to start with a further computational search for the optimum EPBM pressure in order to minimize the surface settlements induced by the excavation. For a more deep analysis of this problem, it seems that we have to proceed using semi-analytical methods. For the current status of this research, see [58, 59]. Important developments have been presented by Schofield [60] and Mair [61] in their Rankine Lectures of 1980 and 2008, respectively.

#### 4. Applications of computational methods in tunnel engineering

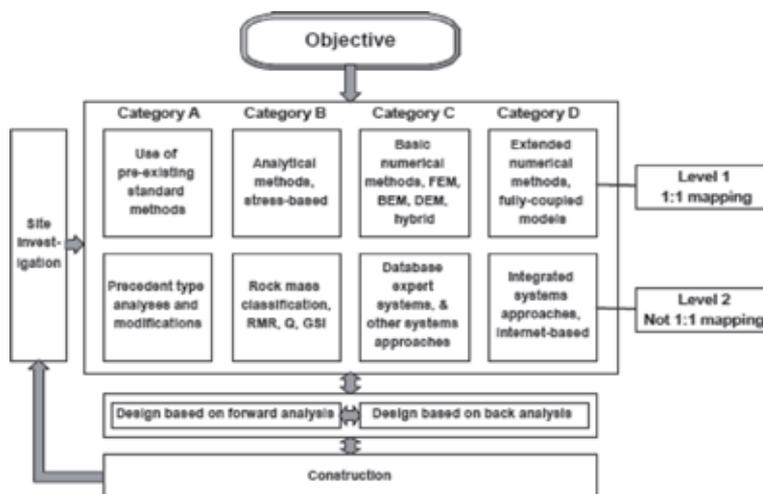
In the last section of the chapter, a short, not complete, survey of computational methods in tunnel engineering is presented. It is a rather chronological account of methods and tools based on author's personal experience.

To start with, in **Figure 17** a very useful and clear classification of methods of analysis is presented [62–64]. For an extensive review of numerical analysis, see Potts [65].

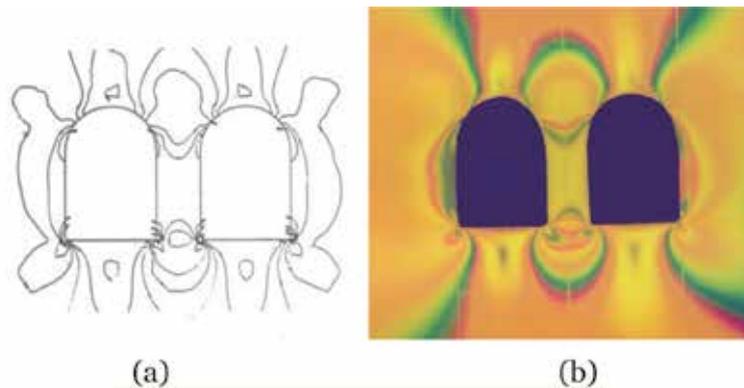
##### 4.1 Level 1: basic numerical methods: 1:1 mapping

The available numerical methods (FEM, BEM, DEM) belong to category “C” making “one-to-one mapping”. The meaning of this term is that they make a direct modeling of geometry and physical mechanisms [63]. In the 1970s finite element method (FEM) codes, based on differential equation formulation, were developed running in mainframe computers. Towards the end of the 1970s, the boundary element method (BEM) was developed, based on integral equation formulation. Bray and his co-workers introduced a 2D indirect formulation of BEM, and they developed a code included in [13].

Based on the example presented in Hoek and Brown ([13], p. 499), a twin cavern problem was analysed using the indirect BEM formulation [66]. In order to check the numerical analysis, a photoelastic model was, also, analysed. The BEM code was modified, to plot isochromes and isoclinics [14] obtained from



**Figure 17.** The four basic methods, in two levels, comprising eight different approaches to rock mechanic modeling [62–64].

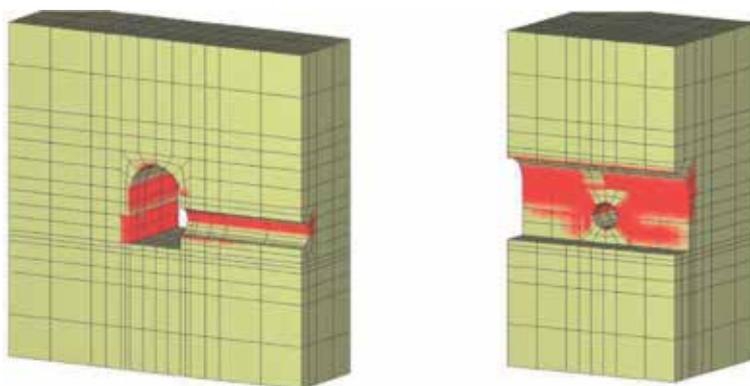


**Figure 18.** Plotting of isochromes of the twin cavern case. (a) Curves obtained from the numerical analysis and (b) curves obtained from the experiment [66].

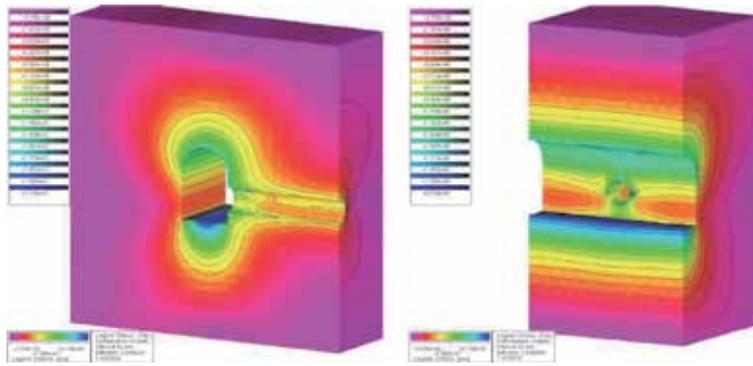
photoelasticity, for comparison. In **Figure 18**, plotting of isochromes and those obtained from the experiment is shown. The agreement between the two is remarkable [66].

In a later stage (2002), with the advances in information systems and methods, the numerical methods to solve engineering problems advanced exponentially. A 3D FEM code was developed, and an extensive study was undertaken for the modeling of rock mass and underground excavations [52]. In that code several failure criteria were implemented. It may be the only 3D FEM code running the elliptic paraboloid criterion described in Section 3.1. In order to benefit from the 3D capabilities of the code, a more complicated problem was analysed. The cavern section included in [13] was modified to include, also, a tunnel of circular section cutting the cavern at a right angle (**Figures 19** and **20**). The rock mass is assumed to be anisotropic with two families of joints [51]. In formulating the problem, the following assumptions have been made: modulus of elasticity  $E = 7$  GPa,  $\nu = 0.25$ ,  $\gamma = 25$  kN/m<sup>3</sup>, tunnel's depth  $Z = 400$  m, vertical stress at infinity  $p = 10$  MPa and in situ stress ratio  $k = 0.8$ . The model had 1441 nodes with 4323 degrees of freedom. About 236 isoparametric hexahedral elements with 20 nodes were used.

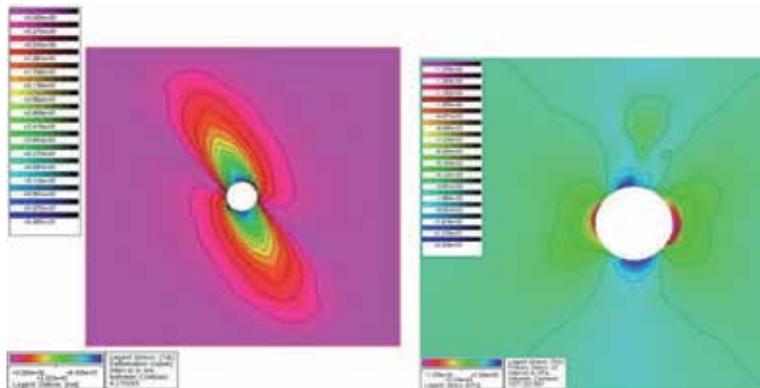
In the following figures, some characteristic results are presented. In **Figure 19**, the formation of plastic zones around the tunnels in the conjunction area is presented. Because of the anisotropy of the problem, the plastic zones are not symmetric. In **Figure 20**, the convergences of the excavation boundaries are shown.



**Figure 19.** Plastic zone formation around the excavations according to elliptic paraboloid failure criterion [52].



**Figure 20.**  
Convergences at the excavation boundaries [52].



**Figure 21.**  
Displacement contours ( $k = 0$ ,  $\beta = 30^\circ$ ,  $r = 0.05$ ,  $s = 0.013$ ) and  $\sigma_3$  stress contours ( $k = 0$ ,  $\beta = 30^\circ$ ,  $r = 0.60$ ,  $s = 0.91$ ) [52].

Finally, in **Figure 21**, the displacement and stress contours are shown. The  $r$  and  $s$  coefficients are defined as  $r = E'/E$  and  $s = G'/G$ . All excavations are assumed to be unlined.

#### 4.2 Level 2: system approaches: non-1:1 mapping

In the last paragraph of this chapter, the application of artificial intelligence (AI) methods and tools in tunnel engineering is shortly discussed. This category of methods belongs to Level 2 methods achieving a non-1:1 mapping (**Figure 17**). In this category of methods, the rock or soil mass is mapped indirectly by a network of nodes [67]. In the 1990s some applications of AI in geotechnical engineering were published [68–74]. From 2001 a great number of application of AI methods in geotechnical engineering have been published. There are two categories of approaches: the supervised learning and the unsupervised learning. Backpropagation, which was the first method applied in geotechnical engineering, belongs to the first category. Interconnected nodes corresponding to parameters involved in the problem represent the physical problem. The output of the training process is taken as a known target [67]. On the contrary, in unsupervised learning methods, the system has to extract knowledge from the data resulting in the underlying interconnections of the parameters of the problem [75]. Currently, a great number of publications are available using more advanced and sophisticated AI

methods. A critical factor is the quality of data and the engineering judgment of the users. There are cases where the overtraining of the system resulted in a “blind” learning of the data guiding to wrong conclusions. The ability to “include creative ability, perception and judgment” [67] is, still, not achieved.

Here, an attempt to train a backpropagation network system using surface settlements induced by underground excavations is presented [76]. The total amount of the available data is 90 records, coming from different types of ground profiles and referring to tunnels constructed with different methods of excavation [77]. The provided information includes tunnel size and depth ( $Z_o$ ), maximum settlement ( $w$ ), settlement trough width ( $i$ ), volume ( $V_s$ ), ground description, geological properties and method of working. An indicative part of the available data is given in **Table 4**.

The range of values of the data, the type of geological profiles and the excavated methods are given in **Table 5** [77].

In training the ANN, two approaches followed. In the first approach, all the available data were included. The tunnel depth and diameter have been used as input data and the surface settlement as output. Qualitative “data” as the geological conditions and the excavation method were not initially taken into consideration as parts of the training process. The training was not successful because we mixed the nonhomogeneous data. Therefore we proceeded to the second approach using all available data and information. To this end, we assigned a number to distinct different geological conditions and excavation method. In this training four inputs were used and the training was successful. In **Figure 22**, the relative importance of the parameters involved to the problem is shown. From this figure, we may conclude that the geometry of the tunnel, i.e. radius and depth, is the main factor with geological profile and excavation method having an important contribution too [76].

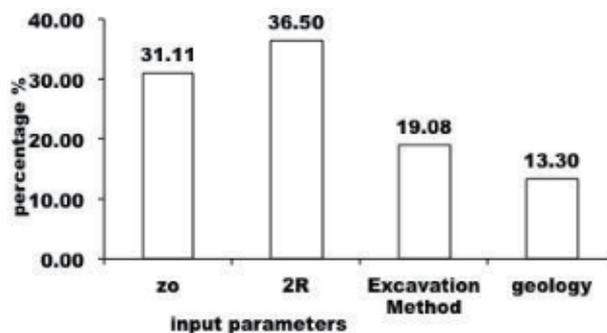
Tunnel	Tunnel data		Maximum recorded settlement $w$ (mm)	Ground geotechnical properties		Volumes $V_s$ (%)	Settlement trough width $i$ (m)	Tunneling method and soil conditions
	Depth $Z_o$ (m)	2R (m)		$c_u$ (kN/m <sup>2</sup> )	$\gamma z_o / c_u$			
London transport fleet line green park	29.30	4.15	6.17	270	2.1	1.40	12.6	Shield construction Stiff, fissured overconsolidated London clay
NWA sewerage scheme, Hebburn	7.50	2.01	7.86	75	2.0	2.42	3.9	Shield construction Laminated clay overlain by stony clay
NWA sewerage scheme, Willington Quay Syphon, Contract 32	13.37	4.25	81.50	33	9.4	13.10	9.1	Shield construction Silty

Case history data on some tunnels and tunneling conditions [76, 77].

**Table 4.**  
 Indicative examples of case history data [76, 77].

	zo (m)	2R (m)	w (mm)
Min	3.40	1.26	1.50
Max	37.49	30.50	280.00
Geology	Homogeneous formations		
	Heterogeneous formations		
Excavation method	Shield construction		
	Hand excavation		
	Slurry shield		
	Shieldless excavation		
	Full face tunneling machine		
	Hydro-shield		
	Full face blasting		

**Table 5.** Range of values of the available data and the categories of geological profiles and excavation methods [76, 77].



**Figure 22.** Relative importance of the parameters contributing to induced settlements due to tunneling [76].

Further tunneling applications using AI tools can be found in the literature. In Zaré and Lavasan [78] an objective system approach is adopted to quantify the interaction of parameters involved in the problem of tunnel face stability. The method is based on a backpropagation ANN approach. A different approach of objective system approach, which adopted the unsupervised type of learning based on self-organizing maps, is, also, published with applications in rock engineering problems [75, 79–85].

## 5. Conclusions

In this chapter, a rather subjective view of tunnel engineering is presented. Nevertheless, the progress made in analytical and computational methods was followed with an effort to be well documented. Starting from a well-known problem of the elastic stress field around a circular deep tunnel, we investigated the influence of tunnel’s shape in stress and displacement fields around a tunnel of the horseshoe section, as well as the extent of the influence zone for several stress fields. To this end Muskhelishvili’s complex variable formulations of stress functions were

used. The next problem was the case of the shallow circular tunnel for which established elastic solutions are presented. Proceeding to the more difficult problem of the plastic analysis, we examined the case of deep tunnels, and then we presented a closed-form solution of the plastic zone formation for the shallow circular tunnel. This is a topic still needing further investigation because of its mathematical difficulty. Computational methods were the last part of the chapter. A classification of methods was presented followed by a problem of deep tunnels analyzed using 3D finite element analysis. The increasing exploitation of artificial intelligence tools in analyzing geotechnical problems was the last topic. The presentation was based on the tunneling-induced surface settlements as a paradigm. The cited references listed below are a basic and indicative selection of literature for further reading.

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# Impact of Tunnels and Underground Spaces on the Seismic Response of Overlying Structures

*Prodromos Psarropoulos*

## Abstract

Depending on the circumstances, the design and construction of tunnels and underground spaces may be very challenging. In the case of an underground project located at a relatively shallow depth in an urban area, the design and construction will probably be more demanding since there is a potential interaction between the underground project and the overlying pre-existing structure(s) that are founded at the ground surface, such as buildings, bridges, etc. This interaction is generally related to the (usually differential) settlements at the ground surface due to the excavation and the consequent distress of the overlying structures. Nevertheless, in areas that are characterized by seismicity, this interaction may be more complicated, since, apart from the aforementioned static interaction, various phenomena of soil dynamics and dynamic interaction may take place, dominating thus the seismic excitation, response, and distress of the overlying structure(s). The current chapter deals with this interesting topic of geotechnical earthquake engineering. After a literature review, some indicative numerical analyses have been performed in order to determine the impact of the main parameters involved. Although the problem is generally complex and multi-parametrical, the numerical results are indicative of the dynamic interaction between the underground project, the ground, and the overlying structure(s).

**Keywords:** tunnel, underground space, seismic response, dynamic interaction

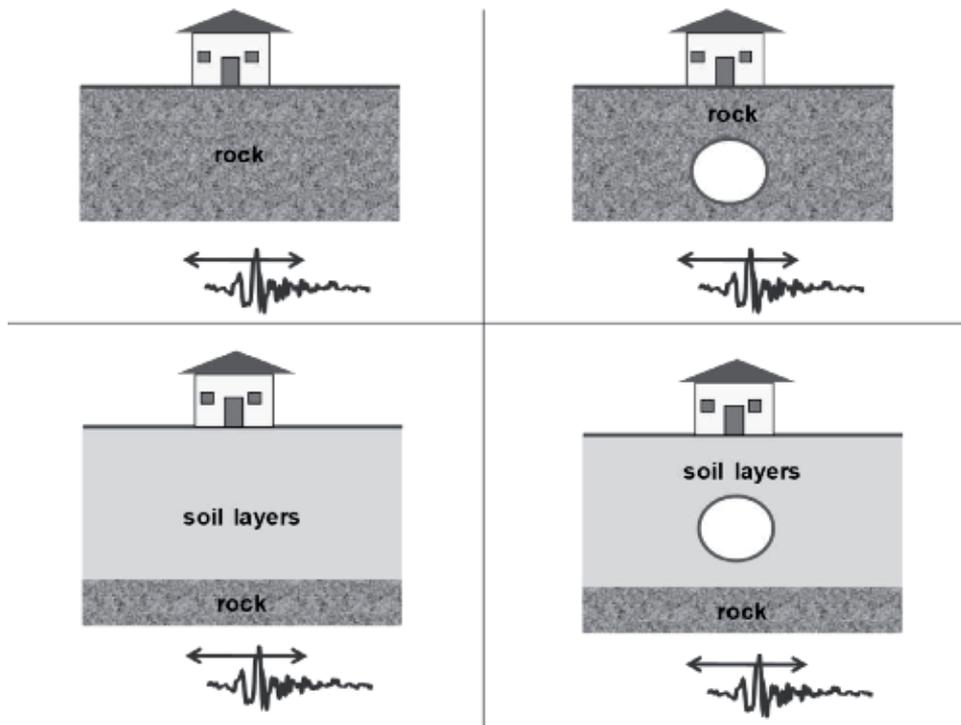
## 1. Introduction

Undoubtedly, during the last decades, a significant progress has been made in the design and construction of underground projects worldwide. Apart from the construction of underground spaces for various purposes (e.g., environmental, military, etc.), the most important underground projects are long tunnels which usually comprise important elements of highways and railways. In urban areas and especially in big cities, the increase in population and the need for fast transportation means have led to the development of metropolitan railways (i.e., subways), and therefore, there has been a large increase in the number and size of underground structures (i.e., metro stations and tunnels). Depending on the local site conditions, in an urban area, one of the main issues during the construction of a tunnel at a relatively shallow depth is the potential (static) interaction between the tunnel and a pre-existing overlying structure at the ground surface, such as a building or a bridge. It is evident that the development of (usually differential) settlements at the ground surface will probably distress any pre-existing structure.

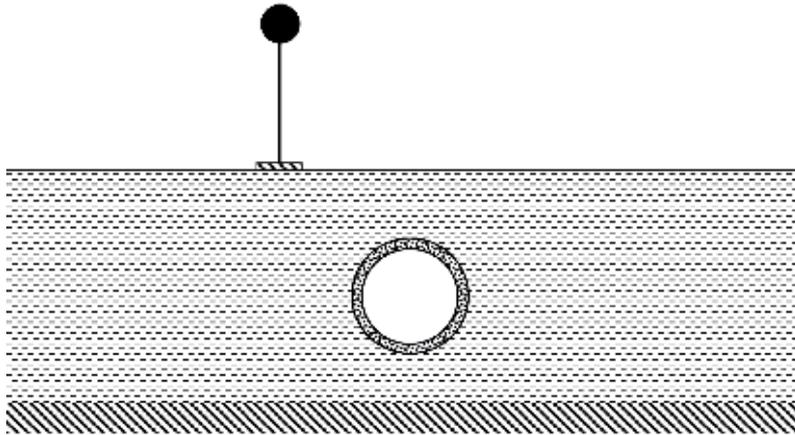
Nevertheless, in areas that are characterized by seismicity, the construction of a tunnel under a pre-existing structure may have an impact, not only on the seismic excitation of the structure but on its seismic (i.e., dynamic) response as well. In geotechnical (earthquake) engineering, the term “local site conditions” is usually used to describe the prevailing topographical and geotechnical conditions. In the case of the existence of an underground project at a relatively shallow depth, the local site conditions include the underground project as well. As shown in **Figure 1**, in the case of a structure founded at the ground surface and subjected to a seismic excitation, there exist four general cases of local site conditions: (a) a structure founded on rock without a tunnel, (b) a structure founded on rock with a tunnel underneath, (c) a structure founded on soil layers without a tunnel, and (d) a structure founded on soil layers with a tunnel underneath.

Additionally, it is generally acknowledged that underground structures suffer less from earthquakes than buildings on the ground surface. Nevertheless, earthquakes in Kobe (1995), Chi-Chi (1999), and Duzce (1999) (see [1–5]) have caused extensive failures in tunnels (and buried pipelines), reviving the interest in the associated analysis and design methods.

The current chapter attempts to shed some light on the seismic (i.e., dynamic) behavior of underground projects and mainly on their impact on the overlying structures. **Figure 2** shows a sketch of the problem under consideration. An underground project (i.e., a circular tunnel) is constructed within a soil layer. A structure is founded on the surface of the soil layer, at a relatively short distance from the underground project. For the sake of simplicity, the structure is considered to be a single-degree-of-freedom system (i.e., a concentrated mass on a single beam). The soil layer and the two structures are subjected to seismic loading, i.e., an acceleration time history applied at the base of the soil layer. Therefore, the dynamic



**Figure 1.** Sketch showing the four potential cases of “local site conditions” of a structure founded at the ground surface and subjected to a seismic excitation.



**Figure 2.**

*Sketch of the problem under consideration: a simple structure founded on a soil layer is overlying a tunnel. The soil layer and the two structures (i.e., tunnel and simple structure) are subjected to an acceleration time history being applied at the base of the soil layer.*

interaction between the underground project, the soil, and the overlying structure is investigated, while emphasis is given to the impact of the underground project on the dynamic response of the overlying structure.

The next sections are involved with various topics of the problem under examination. More specifically, a literature review on the impact of underground structures on the characteristics of the seismic motion at the ground surface is described in Section 2. The aim of the literature review is to identify the most important parameters of the problem. This section also includes some indicative numerical results related to the distribution of accelerations on the ground surface in the presence of an underground circular lined tunnel. Finally, Section 3 is devoted to the dynamic soil-structure interaction phenomena and the seismic response of structures overlying circular tunnels. Although the literature on this issue is relatively limited, various parameters are considered, and useful conclusions are drawn. In this section the numerical simulations also include the overlying structures. The results are indicative of the complexity of the dynamic interaction between the tunnel, the ground, and the structure.

## **2. Impact on the seismic motion at the ground surface**

One of the first studies on the impact of underground projects on the seismic motion was the study of Lee and Trifunac [6]. In their work, Lee and Trifunac analyzed the scattering and refraction of SH shear waves due to a circular tunnel in a homogeneous elastic half-space using an analytical solution.

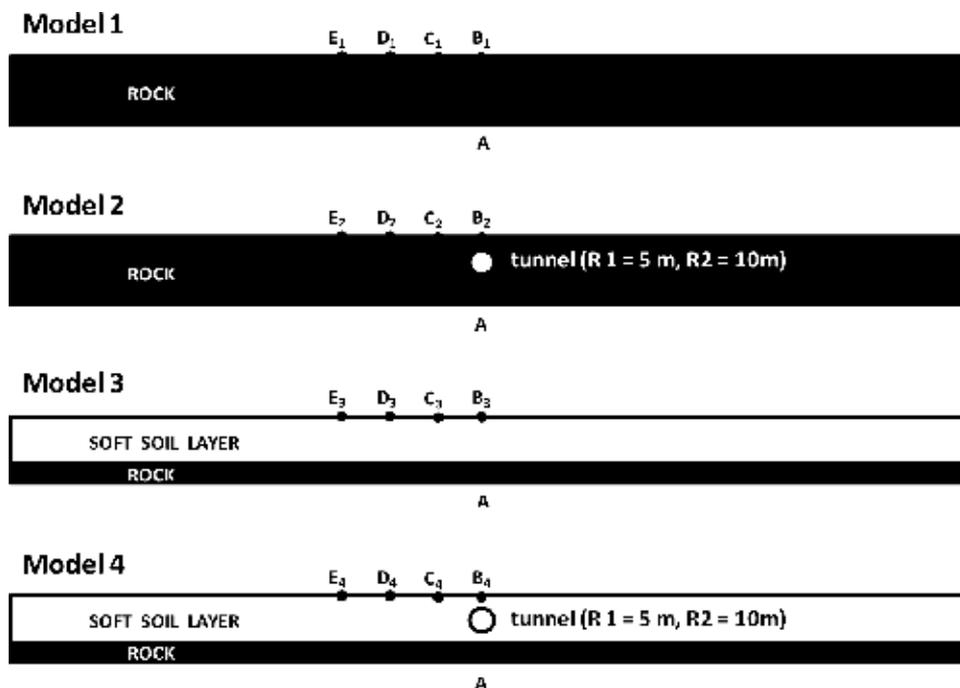
The main parameters that determine the response at the ground surface are (a) the angle of incidence and the frequency content of the seismic waves, (b) the distance from the vertical axis of the tunnel, and (c) the tunnel depth.

During the last four decades, various researchers have studied similar problems. More specifically, many researchers have examined analytically the impact of a circular underground structure on the surface ground motion, while few researchers have examined the problem numerically using mainly the finite element method. In addition, some researchers have verified their analytical results with numerical simulations or vice versa. Finally, very few attempts have been made in order to simulate experimentally the seismic response of underground structures. For more details the reader may refer to [7–20].

The evaluation of the aforementioned publications shows that the impact of underground projects on the seismic motion at the ground surface consists of the following:

- The horizontal seismic motion may be increased or decreased along the ground surface (compared with the corresponding seismic motion observed without the underground project).
- A “shadow zone” is created over the underground project. This phenomenon consists of a reduction of the seismic motion right above the tunnel and an increase at the two corners of the shadow zone.
- The seismic response at the ground surface is further complicated by the appearance of a parasitic vertical component of seismic motion, which may be substantial, especially at the two corners of the shadow zone.
- There is a specific range of frequencies (or periods) in which the seismic motion is increased. That range depends on the characteristics of the underground project and the eigenfrequencies (or eigen-periods) of the ground. Ground response for excitations with wavelengths larger than the tunnel diameter is not affected by the presence of the tunnel.

Figure 3 shows the four numerical models that have initially been examined in order to indicatively demonstrate the potential impact of a tunnel on the seismic motion at the ground surface. Model 1 is actually a rigid rock, while Model 2 is a rigid rock with a lined tunnel. On the other hand, Model 3 consists of a soil layer



**Figure 3.** The four examined numerical models showing the points of interest at the base and at the ground surface. Model 1 is a rigid rock, Model 2 is a rigid rock including a tunnel, and Model 3 is a soft soil layer on rock, while Model 4 is the same model with a tunnel. Point A is located at the base, while the points  $B_i$ ,  $C_i$ ,  $D_i$ , and  $E_i$  (with  $i = 1$  to 4) are at the ground surface.

with a height,  $H$ , of 25 m, overlying a rigid rock (i.e., one-dimensional model). The soil layer is characterized by a shear-wave velocity,  $V_s$ , equal to 200 m/s. Model 4 is the same model including also the aforementioned lined tunnel. Two cases of tunnel radius,  $R$ , have been examined:  $R1 = 5$  m and  $R2 = 10$  m.

All numerical analyses have been performed with PLAXIS2D which is a commercial finite element program capable to perform dynamic ground response analyses in the time domain. Special transmitting boundaries have been applied at the two vertical boundaries of both models in order to avoid unrealistic trapping of the seismic waves.

The four models have been horizontally excited by three acceleration time histories. As shown in **Figure 4**, the first is a sinusoidal motion with frequency  $f_o = 2$  Hz, the second is a simple Ricker wavelet of central frequency  $f_o = 2$  Hz (characterized by a wide range of frequencies up to  $3f_o = 6$  Hz), while the third is a real accelerogram that has been recorded during the 1990 Upland earthquake, in California, with a peak ground acceleration (PGA) of the order of 0.15 g. All excitations have intentionally been scaled to low values of peak acceleration (i.e., 0.01 g) in order to keep the behavior of the geomaterials in the elastic range.

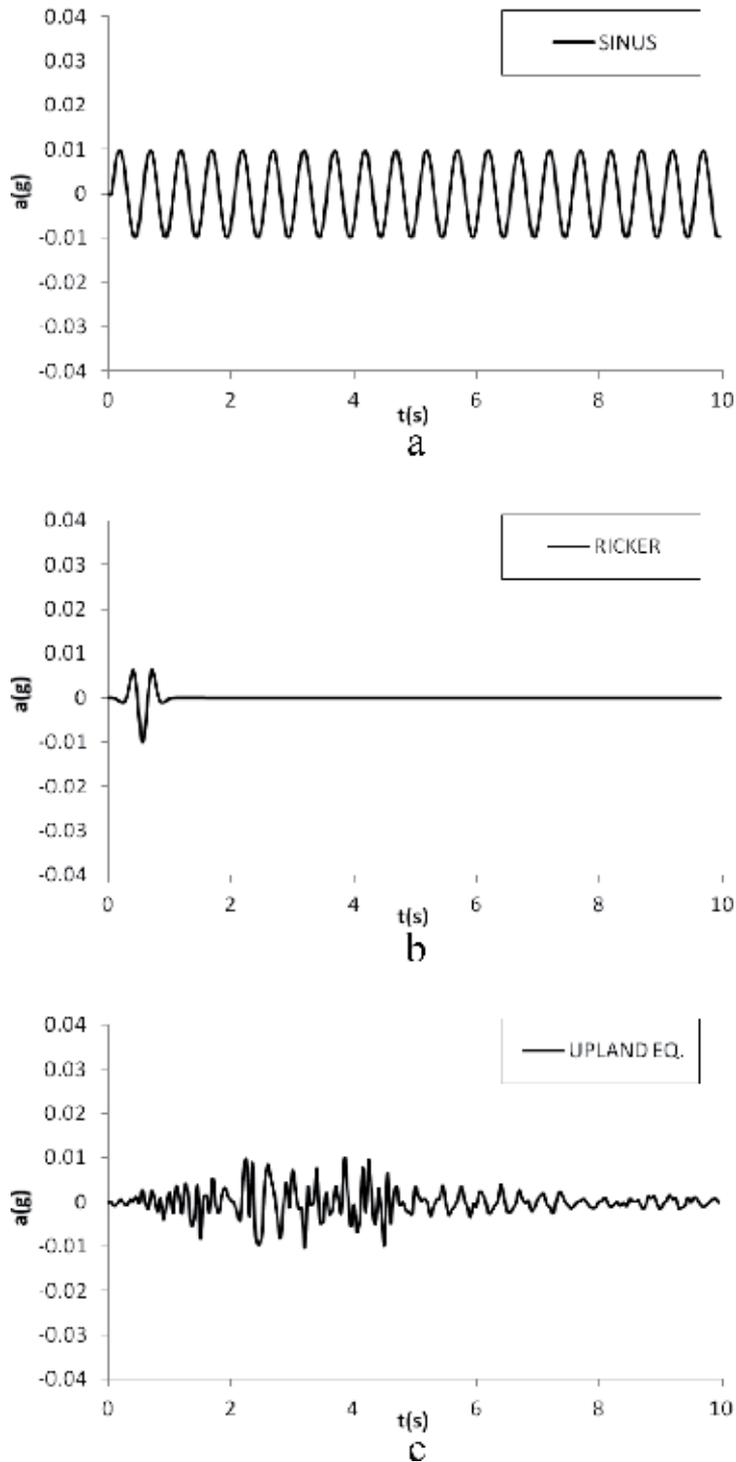
According to the wave propagation theory, the first eigenfrequency of Model 3 (i.e., a single soil layer) is equal to  $f_1 = V_s / (4H) = 2$  Hz, while its maximum theoretical response at resonance is  $2 / (\pi \xi_s)$ , where  $\xi_s$  is the material damping. Therefore, the sinusoidal motion with frequency  $f_o = 2$  Hz has been used in order to verify the numerical simulations with the corresponding analytical solutions. If we suppose that the material damping of soil,  $\xi_s$ , is 5%, the amplification factor,  $AF$ , is 12.7.

**Figures 5–11** show some indicative numerical results.

More specifically, **Figure 5** shows the dynamic response of Model 3 in the case of sinusoidal excitation. As it was expected, resonance phenomena are evident at the ground surface. As aforementioned, the peak ground base acceleration is only 0.01 g, while the peak ground surface acceleration has been amplified almost 12 times. The discrepancy between the  $AF$  from the analytical solution and the corresponding  $AF$  from the numerical modeling is attributed mainly to some deficiencies of the numerical modeling, such as the Rayleigh-type material damping, the size of the finite elements, and/or the rather medium accuracy of the vertical transmitting boundaries.

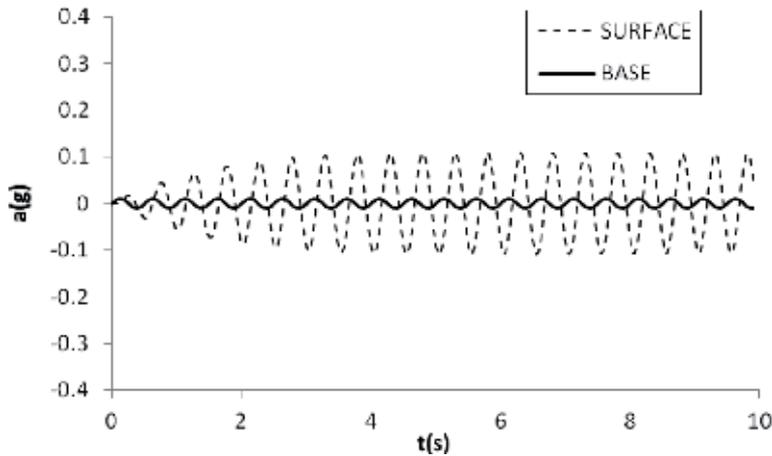
**Figure 6** shows the dynamic response of Model 3 in the case of Ricker excitation and in the case of the record from Upland earthquake. In the case of Ricker excitation, the peak ground surface acceleration is almost 0.02 g, while the duration of the ground motion has been substantially increased from 1 second to almost 5 s. In the case of the recorded acceleration from the Upland earthquake, the ground acceleration has been amplified almost 2.5 times since the peak ground surface acceleration is almost 0.025 g.

As it was expected, in the case of Model 1, Model 2, and Model 3, there are no differences of the dynamic response at the ground surface. Model 1 and Model 2 are rigid, while Model 3 is characterized by one-dimensional conditions. **Figure 8** and **Figure 9** show the calculated time histories of horizontal acceleration at various locations at the ground surface in the case of Model 4 with the small tunnel ( $R1 = 5$  m) and in the case of Model 4 with the big tunnel ( $R2 = 10$  m), respectively. It is evident that the existence of the small tunnel has actually no impact on the variation of the response at the ground surface, a phenomenon that can be attributed to the fact that the size of the tunnel is relatively small compared to the examined wavelengths. On the contrary, in the case of the big tunnel, there is an impact, although it is rather minor. More specifically, the acceleration levels are lower right above the tunnel (i.e., a “shadow zone” has been created), while few meters away (at point C4) the acceleration is locally increased. Similar are the results shown in **Figure 10** where the seismic responses of the points at the ground surface are being compared in the case of Upland excitation.

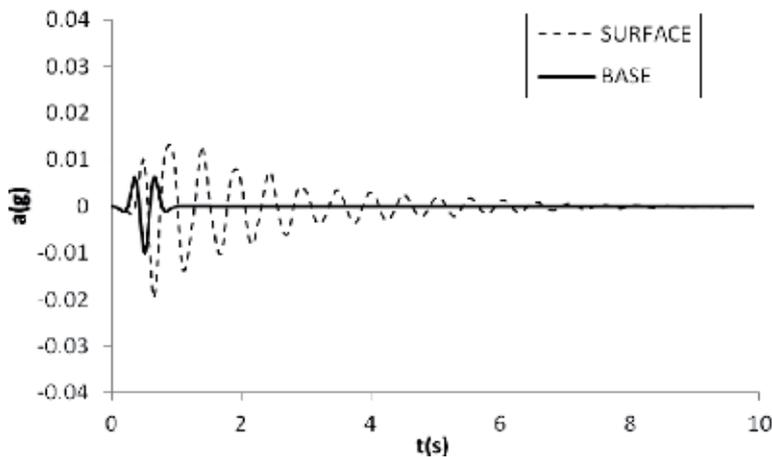


**Figure 4.** The three acceleration time histories that have been used as seismic excitations (all scaled to 0.01 g): (a) a sinusoidal excitation, (b) a Ricker pulse, and (c) the record from the Upland earthquake.

As aforementioned, another phenomenon that may take place due to the presence of a tunnel is the development of parasitic vertical accelerations. Since the seismic excitation in all numerical analyses was only horizontal (i.e., S waves), no



**Figure 5.**  
*The dynamic response of Model 3 subjected to the sinusoidal excitation.*

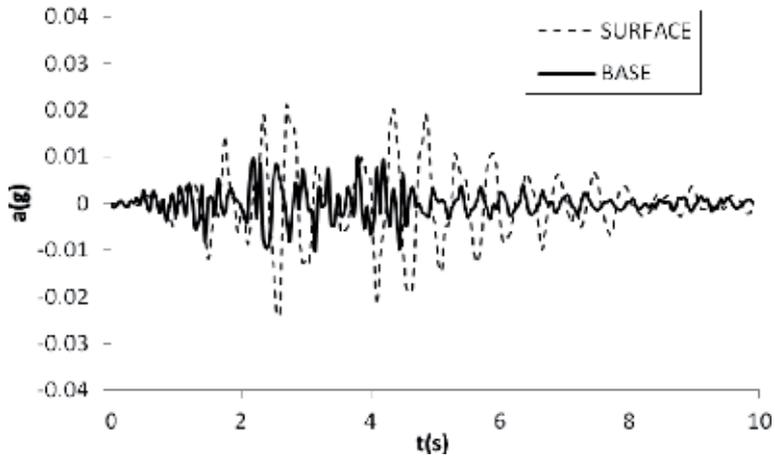


**Figure 6.**  
*The dynamic response of Model 3 subjected to the Ricker excitation.*

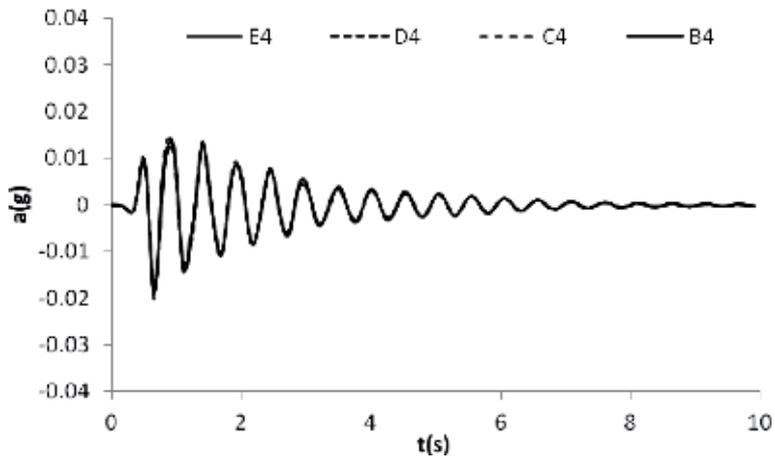
vertical ground motion is expected. This is reasonable for Model 1 and Model 2 that are rigid and for Model 3 that is one-dimensional. Nevertheless, the points at the surface of Model 4 exhibit this vertical parasitic acceleration. **Figure 11** shows the vertical acceleration at various locations along the ground surface in the case of the big tunnel and the Upland excitation. It is noted that the maximum vertical acceleration is observed at point C4 where its value is almost 30% of the corresponding peak ground surface acceleration (i.e., 0.03 g) and in parallel comparable to the peak ground base acceleration (i.e., 0.01 g).

Judging from the numerical results of this rather limited parametric study, it becomes evident that the existence of the tunnel alters the acceleration pattern along the ground surface. The “shadow zone” right above the tunnel and the vertical parasitic seismic motion are rather obvious.

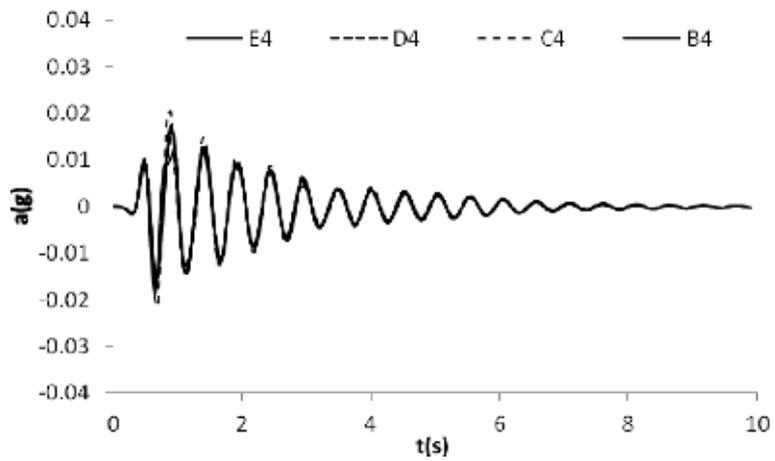
In any case, it has to be emphasized that the interaction between the soil, the structure, and the tunnel is a problem with several parameters, and therefore, in any other case the patterns of horizontal and vertical acceleration at the ground surface may be completely different.



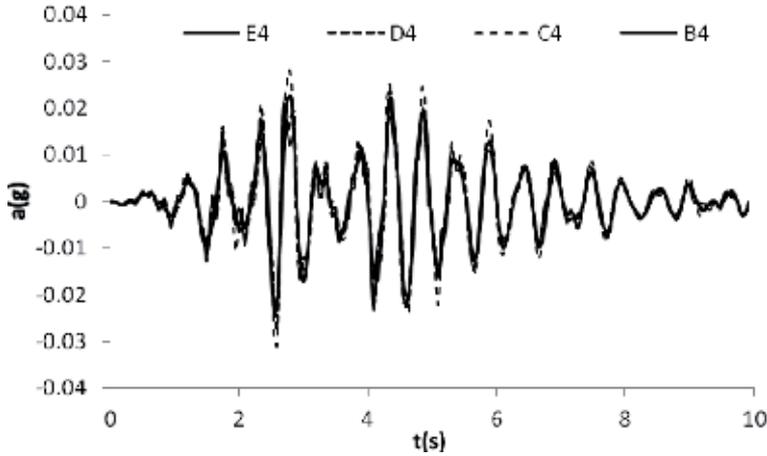
**Figure 7.**  
*The dynamic response of Model 3 subjected to the Upland excitation.*



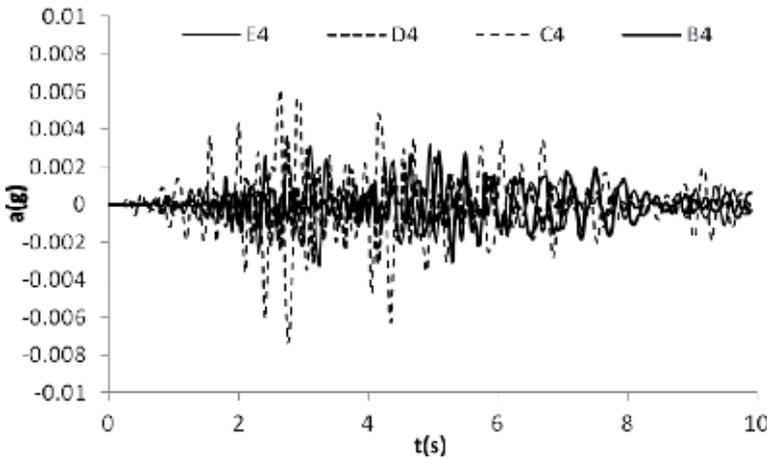
**Figure 8.**  
*The dynamic response of Model 4 with the small tunnel ( $R_1 = 5\text{ m}$ ) subjected to the Ricker excitation.*



**Figure 9.**  
*The dynamic response of Model 4 with the big tunnel ( $R_2 = 10\text{ m}$ ) subjected to the Ricker excitation.*



**Figure 10.**  
 The dynamic response of Model 4 with the big tunnel ( $R_2 = 10\text{ m}$ ) subjected to the Upland excitation.



**Figure 11.**  
 The vertical dynamic response of Model 4 with the big tunnel ( $R_2 = 10\text{ m}$ ) subjected to the Upland excitation.

### 3. Dynamic interaction between tunnel, ground, and structure

The seismic response of any structure founded at the ground surface is an issue that depends on various factors, such as the mechanical and geometrical properties of the structure and the characteristics of the seismic excitation.

When the structure is a single-degree-of-freedom (SDOF) structure with a fixed base, then it is characterized by its eigen-period  $T_o$ , which is given by the following simple expression:

$$T_o = 2\pi\sqrt{\frac{M}{K}} \quad (1)$$

where  $M$  is the concentrated mass of the structure and  $K$  is its stiffness.

Therefore, if the fundamental period of the seismic excitation is close to  $T_o$ , resonance phenomena are expected, and therefore, the dynamic distress of the structure may have its maximum value.

Nevertheless, according to [21, 22], the potential existence of soft soil layers under the structure will lead to the following phenomena:

a. Soil amplification (or de-amplification in very few cases) will take place, which will certainly lead to an alteration of the seismic excitation of the structure.

b. Dynamic soil-structure interaction, which actually consists of the following phenomena:

- The soil compliance will reduce the stiffness of the structure, a fact that will certainly lead to an increase of the eigen-period of the structure.
- The overall damping of the system will be increased since the existence of soil layers will introduce other means of energy dissipation apart from the material damping of the structure, such as the material damping of the soil and the radiation damping.

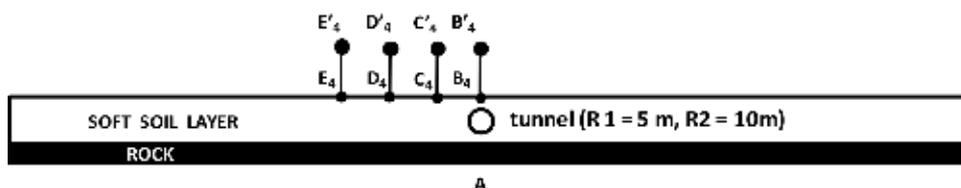
Although the increase of the damping is always beneficial, the reduction of the stiffness (and the subsequent increase of the eigen-period) may be either beneficial or detrimental for the distress of the structure, depending on the circumstances.

In this section all the previous numerical models have been modified in order to include four identical simple structures (4) at the ground surface. As shown in **Figure 12**, the four structures are above the tunnel, while the distance between them is the same (15 m). All of them are single-degree-of-freedom (SDOF) structures, and they are characterized by (a) a material damping of 5% and (b) an eigen-period  $T_o = 0.5$  s or eigenfrequency  $f_o = 2$  Hz (identical to the first eigenfrequency of the soil layer). Note that in Model 3 and Model 4, the actual eigenfrequency of the structures is smaller due to the soil compliance.

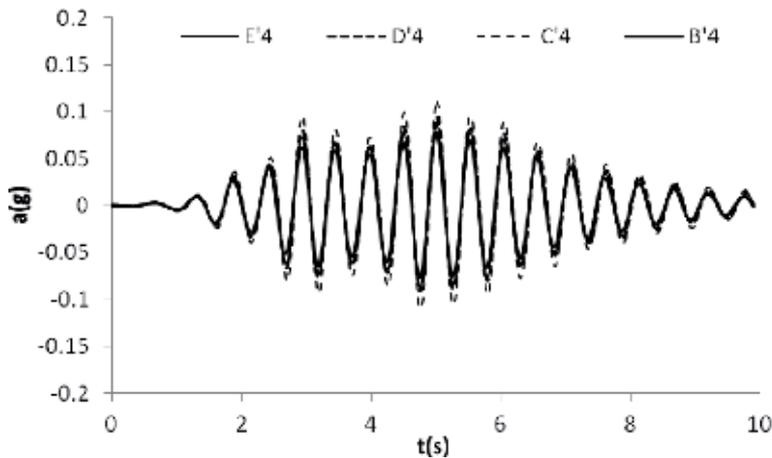
The following figures show some indicative numerical results. More specifically, **Figure 13** shows the horizontal acceleration time histories that have been developed on the top of the structures in the case of Model 4 with the big tunnel subjected to the Upland excitation. It is evident that the acceleration levels are relatively high (of the order of 0.1 g). This fact is attributed to the resonance phenomena between the soil and the structures (since they have comparable eigenfrequencies). The initial peak ground base acceleration (of 0.01 g) has been amplified up to 0.03 g (i.e., almost three times) at the ground surface, while the peak ground surface acceleration has been amplified again, reaching a value of the order of 0.1 g.

In parallel, minor differences exist between the structural responses of the four structures. As it was expected, the minimum response is observed in the case of the structure located at point B, while the maximum response is on the structure located at point C.

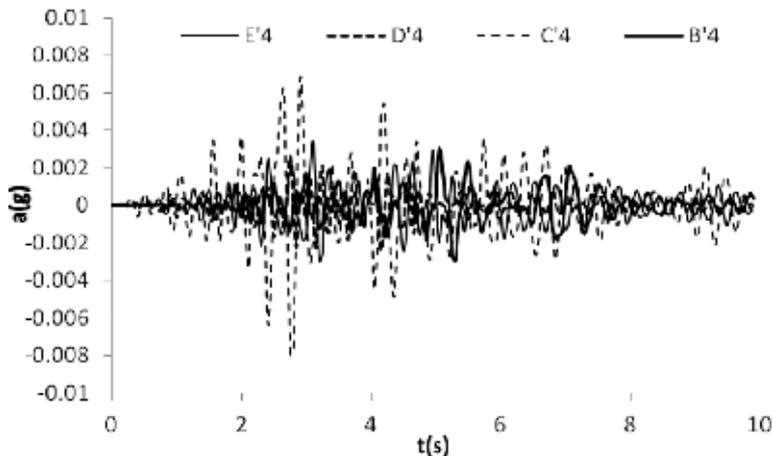
**Figure 14** shows the corresponding (parasitic) vertical accelerations that have been developed on the top of the structures. The maximum response is also observed in the case of the structure located at point C. Note that these accelerations are comparable to the acceleration levels at the ground surface (see **Figure 11**).



**Figure 12.** The modified Model 4 including four (4) equally spaced single-degree-of-freedom structures at the ground surface.



**Figure 13.**  
*The horizontal accelerations developed on the top of the four structures in the case of the modified Model 4 with the big tunnel subjected to the Upland excitation.*



**Figure 14.**  
*The vertical accelerations developed on the top of the four structures in the case of the modified Model 4 with the big tunnel subjected to the Upland excitation.*

This phenomenon was actually expected since the single-degree-of-freedom structures have no vertical response. Note that in a more realistic case with multi-degree-of-freedom systems, the vertical component would have been amplified.

#### 4. Conclusions

In urban areas and especially in big cities, the increase in population and the need for fast transportation means will lead to the development of metropolitan railways (i.e., subways), and therefore, there will be a large increase in the number and size of underground structures (i.e., metro stations and tunnels).

In areas that are characterized by moderate or high seismicity, it is evident that the construction of an underground project (e.g., tunnel or underground space) under a pre-existing structure may alter more the seismic excitation of the structure, modify the soil-structure interaction pattern, and consequently have an impact on the structural response and distress.

The numerical results that have been presented in the previous sections have shown that the existence of a tunnel may alter the pattern of horizontal acceleration at the ground surface in the time domain (and in the frequency domain). This fact means that the construction of a tunnel under a pre-existing structure will complicate more the aforementioned dynamic soil-structure interaction phenomena.

Finally, it has to be emphasized that the anticipated vertical parasitic acceleration may have an impact on the structural response and distress of structures with many degrees of freedom, especially when the acceleration levels of the seismic excitation are high and a nonlinear behavior of the structure is expected.

Based on all aforementioned, when a new underground structure is constructed in urban areas, a special study should be performed in order to assess quantitatively the impact of the underground structure on the seismic response and distress of any pre-existing overlying structure.

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# Designing a Tunnel

*Spiros Massinas*

## Abstract

Designing a tunnel is always a challenge. For shallow tunnels under cities due to the presence of buildings, bridges, important avenues, antiquities, etc. at the surface and other infrastructures in the vicinity of underground tunnels, parameters like vibrations and ground settlements must be tightly controlled. Urban tunnels are often made in soils with very low values of overburden. Risks of collapse and large deformations at the surface are high; thus negative impact on old buildings are likely to occur if appropriate measures are not taken in advance, when designing and constructing the tunnel. For deep tunnels with high overburden and low rock mass properties, squeezing conditions and excessive loads around the excavation can jeopardize the stability of the tunnel, leading to extensive collapse. The aim of the chapter is to give details on advance computational modelling and analytical methodologies, which can be used in order to design shallow and deep tunnels and to present real case studies from around the world, from very shallow tunnels in India with only 4.5 m overburden to a deep tunnel in Venezuela with extreme squeezing conditions under 1300 m overburden.

**Keywords:** tunnel, shallow tunnelling, deep tunnels, surface settlements, tunnel squeezing, analytic solution, numerical analysis, lining stress controllers, sliding joints, monument underpassing, monitoring, plasticity, TBM, earth pressure balance shield, high overburden, high deformations, NATM, conventional tunnelling

## 1. Introduction

The aim of the current chapter is to give details and guidance in designing underground tunnels, to be constructed with tunnel boring machines (TBM).

In the sequence of paragraphs to follow, the reader will get a grasp of tunnelling principles and theory related to mountainous and urban tunnelling—deep and shallow tunnels will be explained (paragraph 2).

In paragraph 3, details for designing mechanized constructed (TBM) Metro tunnels in urban environment will be given, and real cases from India will be presented. Special case for monument underpass with earth pressure balance machine (EPBM) under extreme low overburden will also be discussed, and the real case study from Chandpole Gate in Jaipur will be presented.

Mountainous and deep tunnels design will be presented in paragraph 4. Squeezing and non-squeezing conditions will be explained, and methods of designing will be given, along with examples from around the world.

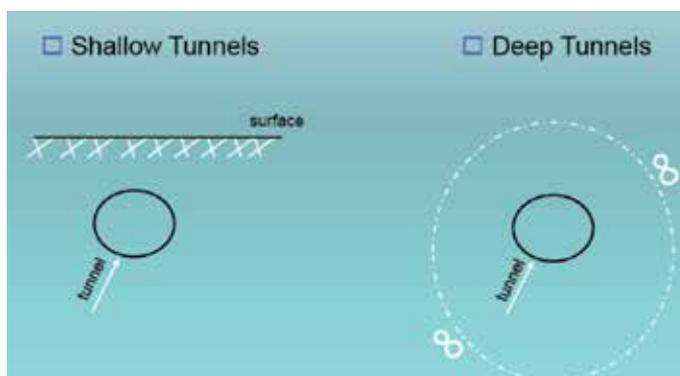
The final paragraph will summarize the primary conclusions from the presented design methodologies. It is noted that all the examples presented herein are real cases of tunnels already constructed, with the personal involvement of the author in their designs elaboration.

## 2. Tunnelling principles

Tunnelling is divided in two general categories: the deep tunnel case and the shallow tunnel case (**Figure 1**). The most common example of a shallow tunnel is the Metro Lines in a big city. For example, underground tunnels realize the connection between the underground stations. Since the stations are constructed, as underground structures, to serve the surface mass transit system, the depth of the train platform from the surface is limited to meters; in a normal typical station, the platform depth can vary from 15 to 25 m, while for other cases like flagship stations connecting different Metro Lines, platforms can be in different levels, and consequently their depth can reach and even exceed 40 m. Therefore, the tunnels connecting the stations can also vary in depth, and the typical overburden height (distance between the tunnel crown and the surface) can be 10–20 m, while for deeper sections can reach or even exceed 35 m. Of course along a Metro Line, there are always unique cases where the tunnel depth can be very limited, and thus the overburden height can be even less than a tunnel diameter (e.g., 5 m); for such a special case, the real case study from Chandpole Gate in Jaipur will be presented in the following paragraphs. Therefore, from a mathematical perspective, in shallow tunnelling, two boundaries are introduced, the tunnel geometry (circular or not) and the surface (**Figure 1**).

On the contrary, for a deep tunnel case, the problem can be described only by one boundary, the tunnel geometry (**Figure 1**). Furthermore, the influence of the variation of the in situ stress with the depth is more intense and critical in a shallow tunnel rather than in a deeper tunnel case. In the latter, the in situ stress difference between tunnel crown and invert is insignificant compared to the absolute value of the in situ stress at this depth. To understand the main difference between shallow and deep tunnel, a characteristic example of a motorway with multiple tunnels crossing a mountainous terrain can be used. In such a case where the tunnel pierces a mountain, the overburden height can be decades of meters (120, 150 m, etc.) or even hundreds of meters and even can exceed 1000 m; a real case study for a very deep tunnel in Venezuela will be presented later on.

The first to differentiate the shallow from the deep tunnel, covering, also, the intermediate zone between deep and shallow tunnels, was Bray [2]. In order to do so, he introduced the dimensionless ratio of the depth (from tunnel center— $d_i$ ) to the radius ( $r_i$ ) of a tunnel. For a ratio ( $d_i/r_i$ ) equal or greater to 25—e.g., a tunnel with radius 5 and 120 m overburden height—the deep tunnel case is described, while the shallow tunnel is defined by a ratio ( $d_i/r_i$ ) smaller or equal to 7—e.g., a



**Figure 1.** Shallow and deep tunnels [1].

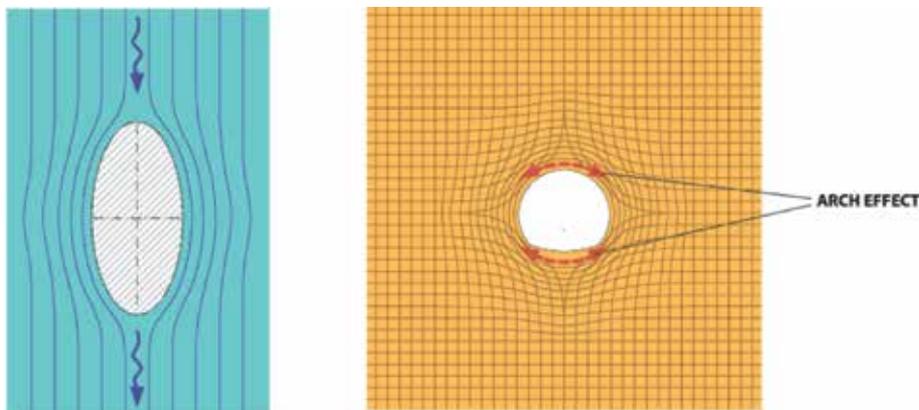
tunnel with radius 5 and 25 m overburden. For the intermediate values of the ratio, thus between 7 and 25, a transition zone is defined. The stress distribution around an underground cavity either shallow or deep is the key factor to design a tunnel.

In a similar fashion to the lines of flow in the current of a river, which are deviated by the pier of a bridge and increase in speed as they run around it, the flow lines of the stress field in a rock mass are deviated by the opening of a cavity (tunnel) and are channelled around it to create a zone of increased stress around the walls of the excavation. The channelling of the flow of stresses around the cavity introduces the arch effect (**Figure 2**).

Arch effect can occur, depending on the size of the stresses (overburden height) and the geomechanical properties of the rock mass (strength and deformation properties), (a) close to the profile of the tunnel, (b) far from the profile of the tunnel, and (c) not at all.

Case (a) occurs when the rock mass around the tunnel withstands the deviated stress flow, responding elastically in terms of strength and deformation. In case (b), due to the low properties of the rock mass, the ground around the excavation is not able to withstand the deviated stress flow and thus responds nonelastically, plasticizing and deforming in proportion to the volume of ground involved in the plasticization phenomenon. The latter, that often causes an increase in the volume of the ground affected, propagates radially and deviates the channelling of the stresses outwards into the rock mass until the triaxial stress state is compatible with the strength properties of the rock mass. In this situation, the arch effect is formed far from the line of the excavation and the ground around the tunnel which has been plasticized (plastic zone), contributing to the final tunnel stability with its own residual strength giving rise to deformations, which is often sufficient to compromise the safety of the excavation. With proper support measures, the ground can be “helped”; the plasticization phenomena can be limited, and thus the formation of the arch effect by natural means can be produced, and the tunnel stability can be ensured. In the third case (c), the ground around the cavity is completely unable to withstand the deviated stress flow and responds in the failure range producing the collapse of the tunnel. In such case the arch effect cannot be formed naturally, and thus pre-support measures must be used before the excavation, in order artificially to initiate the arch effect.

The reaction is the deformation response of the medium (ground) to the action of excavation (tunnelling). It is always generated ahead of the excavation face within the area that is disturbed, following the generation of greater stress in the medium around the cavity. The deformation response always depends on the

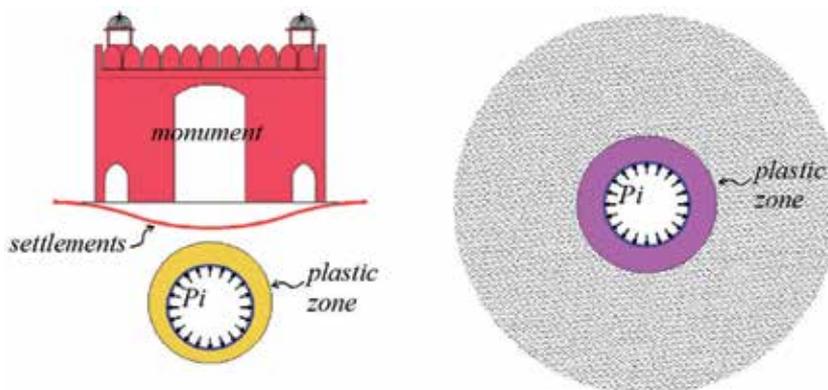


**Figure 2.** Flow lines in the current of a river around a pier (left) and stress field flow lines around a tunnel [1].

medium's properties and its stress state and is affected by the tunnel's face advance. As the face advances, the tunnel passes from a triaxial to a plane stress state. In case that the progressive decrease in the confinement pressure at the face ( $\sigma_3 = 0$ ) produces stress in the elastic range ahead of the face, then the excavation face remains stable with limited and absolutely negligible deformation. In this case the channelling of stresses around the cavity (arch effect) is produced by natural means close to the profile of the excavation, and no artificial support is required to secure the tunnel stability. If, on the other hand, the progressive decrease in the stress state at the face ( $\sigma_3 = 0$ ) produces stress in the elastoplastic range ahead of the face, then elastic-plastic deformation of the face will give rise to a condition of short-term stability. This means that in the absence of any intervention, plasticization is triggered, which, by propagating radially and longitudinally from the walls of the excavation, produces a shift of the "arch effect" away from the tunnel further into the rock mass. This shift from the theoretical profile of the tunnel can only be controlled by intervention to stabilize the ground.

Therefore, in respect to the in situ ground properties, the in situ stress field, and the applied support measures/pressure, the stress redistribution remains within the elastic domain, or a phenomenon of plasticization is triggered which may result to the formation of a plastic domain around the tunnel. Extent/width and shape of the plastic zone around the cavity are the main parameters for calculating/evaluating the stability conditions of an underground excavation. The impact of this mechanism is different in a shallow and a deep tunnel.

In a shallow tunnel, the overburden height is limited. Therefore, any underground deformation that may result from the soil plasticization will affect the development of the surface settlements. In case that the extent of the plastic zone gives rise to excessive surface settlements (**Figure 3**), damages on the surface structures can also be significant or even severe. For a shallow tunnel design, the key parameter is to minimize the magnitude of the developed surface settlements, and thus the redistribution of the stresses around the tunnel needs to be controlled accordingly by designing and applying proper support measures and consequently proper support pressure ( $P_i$ ). However, for the deep tunnel case the redistribution of the stresses around the excavation can be the main challenge. The increased overburden height combined with low ground mass properties can give rise to excessive loads exerted around the excavation and thus can lead to tunnel collapse, if non-proper design of the support measures is elaborated. In such difficult cases, safe tunnel advance can only be achieved with controlled plasticization of the



**Figure 3.** Impact of plastic zone formation around a shallow (left) and a deep (right) tunnel.

groundmass; intervening with application of proper yielding support measures if the groundmass is deformed plastically under controllable manner, and thus the exerted loads can significantly be reduced.

Three main different methods can be applied to solve tunnelling problems:

- a. Analytic
- b. Computational
- c. Combination of the above

Analytic methods are related with the use of closed form solutions, while the finite element method (FEM) and the finite difference method (FDM) describe the computational procedures. Finally, combination of closed form solutions with FEM or FDM modelling can be more beneficial than the other two methods along; closed form solution can give the opportunity to the designer for quick and accurate calculations and thus to be properly “guided” in the elaboration of the final FEM/FDM modelling. Indeed, empirical methods (e.g., Protodiakonov, Terzaghi, etc.) are also used in some cases for solving tunnelling problems.

### **3. TBM shallow tunnelling in urban environment**

The principle objective when designing a shallow tunnel in an urban environment is to minimize the induced surface settlements. Therefore, the face stability along with the tunnel’s induced displacements are the key factors to control the extent of the plastic zone formation and consequently to secure the surface structures from undesirable settlements. In order to design the tunnel, the method of construction to be adopted for the work’s execution needs to be defined. In the current paragraph, the principles for designing a Metro tunnel constructed with tunnel boring shield machine (earth pressure balance (EPB)-TBM) will be discussed, and the main design phases will be explained in detail.

After the alignment is fixed, the geological and geotechnical conditions along the alignment of a Metro project are always the main input required in order to further design the tunnels. Knowledge of the regional geology of the area along with geotechnical investigation campaign is always mandatory in order to determine the ground properties (**Figure 4**).

Boreholes with sampling, executed typically every 50–150 m, aim to investigate the ground conditions at least one to two diameters below the tunnel invert.

Mainly the complexity of the geological model of the area is a key factor to determine the required number of the boreholes along the alignment and if additional investigations such as geophysical surveys will be needed. For example, for the phase 3 of Delhi Metro, the general geology of the area (**Figure 4**) reveals the existence of polycyclic sequence of brown silty clay with kankar, fine- to medium-grained micaceous sand, c-silty/clay, and s-sandy facies. Those thick layers of alluvium deposits mainly are known as Delhi Silt with almost uniform properties along the alignment. Indeed, the presence of quartzite rock is also visible in certain areas of Delhi. Although the exposures are very less in number, structurally it occurs in the form of anticline and syncline with axial trend as N-S and SE-NW; their appearance within the tunnel excavation face (mainly consisted of soft soil—Delhi silt) can be detrimental for the tunnel construction with TBM. Therefore, geotechnical campaign with dense boreholes were executed (varying from 50 to 70 m distances) with the aim to investigate

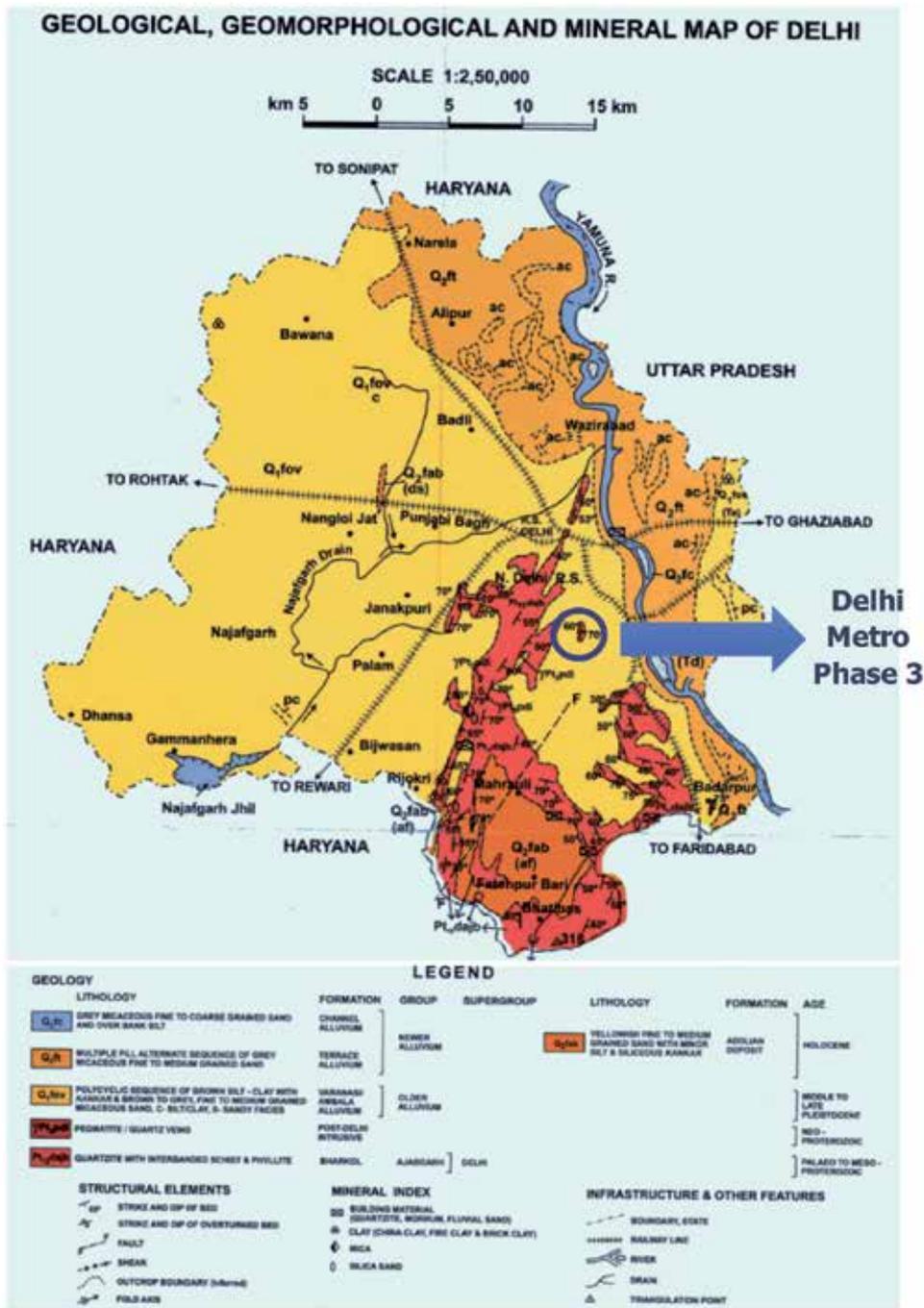
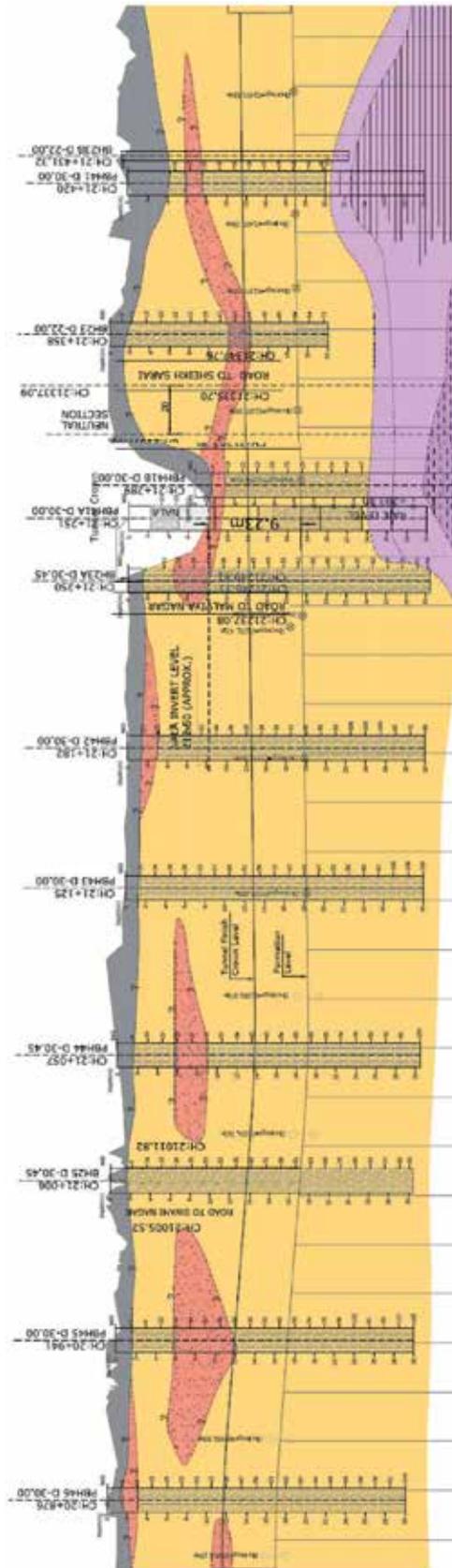


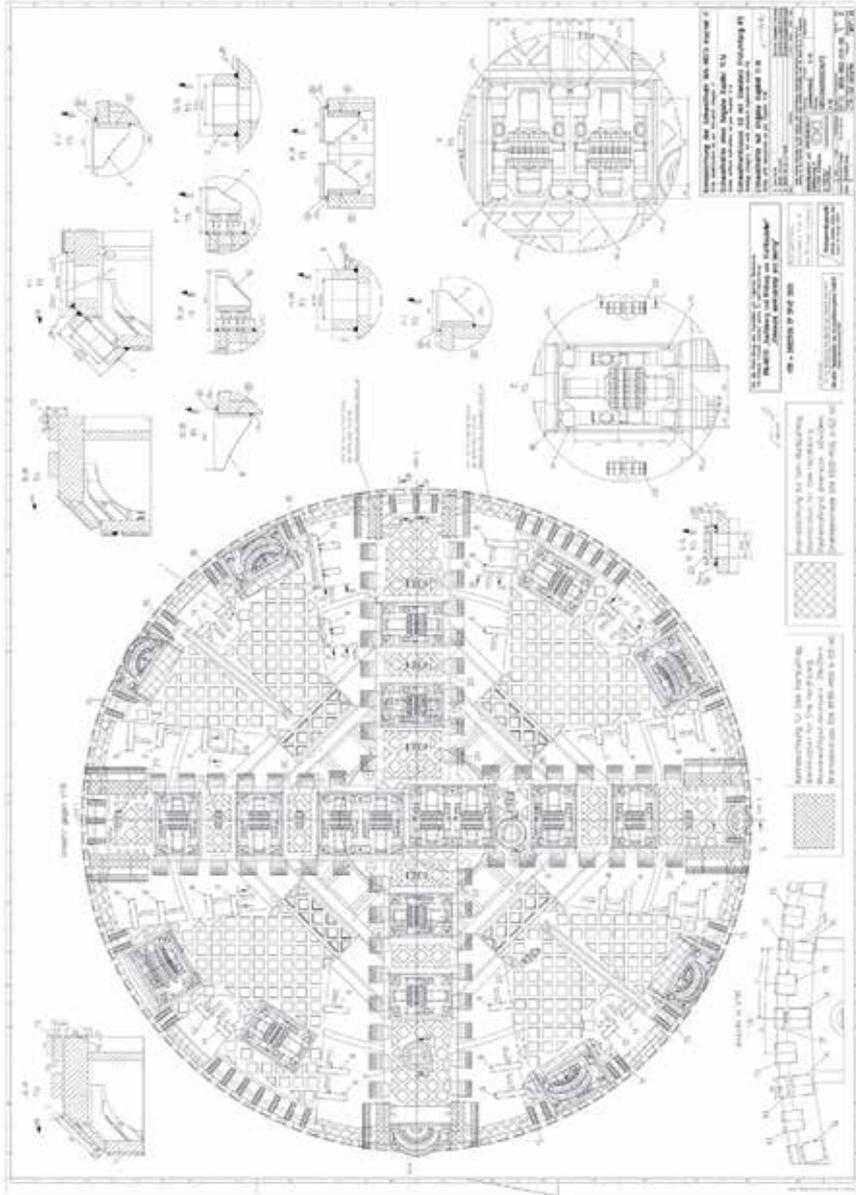
Figure 4. Geological map of Delhi (Source: Geological Survey of India) (Delhi Metro Phase 3 area marked with blue circle).

the actual depth of the bedrock (Figure 5), to quantify the risk from having mixed face conditions (soft soil with hard rock), and finally to determine the arrangement of the EPB-TBM cutterhead (to be designed for mixed face conditions) (Figure 6).

For another case, where the stratigraphy is uniform along the entire alignment, the boreholes can be in greater distance. Let us investigate another example, again from India but from another area, the state of Rajasthan and the Jaipur Metro Phase 1B.



**Figure 5.** Geological longitudinal section of shallow tunnel in Delhi Metro Phase 3 (CC23), with boreholes [3]—Delhi silt marked with yellow color and quartzite rock with magenta.



**Figure 6.** EPB-TBM cutterhead designed by Herrenknecht for mixed face conditions. Cutter discs are visible (Delhi Metro Phase 3—CC23). Source: FEMC-Pratibha JV.

Quaternary sediment occupies the major northeastern part of Jaipur, but hills and ridges are present around the city (**Figure 7**). Granite gneisses occupy the southern part of the district. Other important lithological units exposed in Jaipur are limestone, sandstone, etc. Soil alluvium is the main formation along the alignment of the project, in the form of sand to silty sand with gravels. The uniformity of the geological conditions dictated the determination of a geotechnical campaign with boreholes in greater distances varying from 100 to 150 m (**Figure 8**).

On the contrary to the Delhi Metro, the EPB-TBM for Jaipur is designed with an open spoke-type cutterhead with opening ratio of 60%, suitable for the sandy formations (**Figure 9**). The increased opening ratio of the cutterhead is more suitable in applying uniformly the earth pressure on the excavation face but is essential to maintain always the pressure in every stroke of the machine as it moves forward, in order to avoid face instabilities and excessive surface settlements.

For both cases, Delhi and Jaipur Metro, the water table is revealed below the tunnel invert.

### 3.1 Geotechnical assessment and interpretation

Following the geotechnical campaign and the laboratory tests on the samples, which are taken from the executed boreholes, the geotechnical model will be determined along with the characteristic geotechnical parameters of the soil formations. The modulus of elasticity ( $E$ ), the effective cohesion ( $c'$ ), effective friction angle ( $\phi'$  or  $\phi$ ), and the unit weight ( $\gamma$ ) of the soil formations are the main geomechanical properties required for designing the tunnel excavation. While the effective cohesion and friction angle are critical parameters that will govern the extent of the plastic zone shape, the modulus of elasticity controls the magnitude of the surface settlements, and the tunnel depth is the geometrical parameter that will affect the shape of the surface settlement trough.

When tunnelling with EPB shields, the surface settlement development is related with the machine operation in conjunction with the soil properties. The so-called volume loss, resulted from the face extrusion (face loss), the steering gap closure (radial loss on the shield), and the annular void between the segmental lining and the soil, are basic key parameters that will affect the magnitude of the soil's deformations. It is obvious that the soil properties are the governing factors for determining the operation parameters of the TBM, in order to keep the volume loss within acceptable limits, to control the plastic zone formation, and thus to minimize the surface settlements.

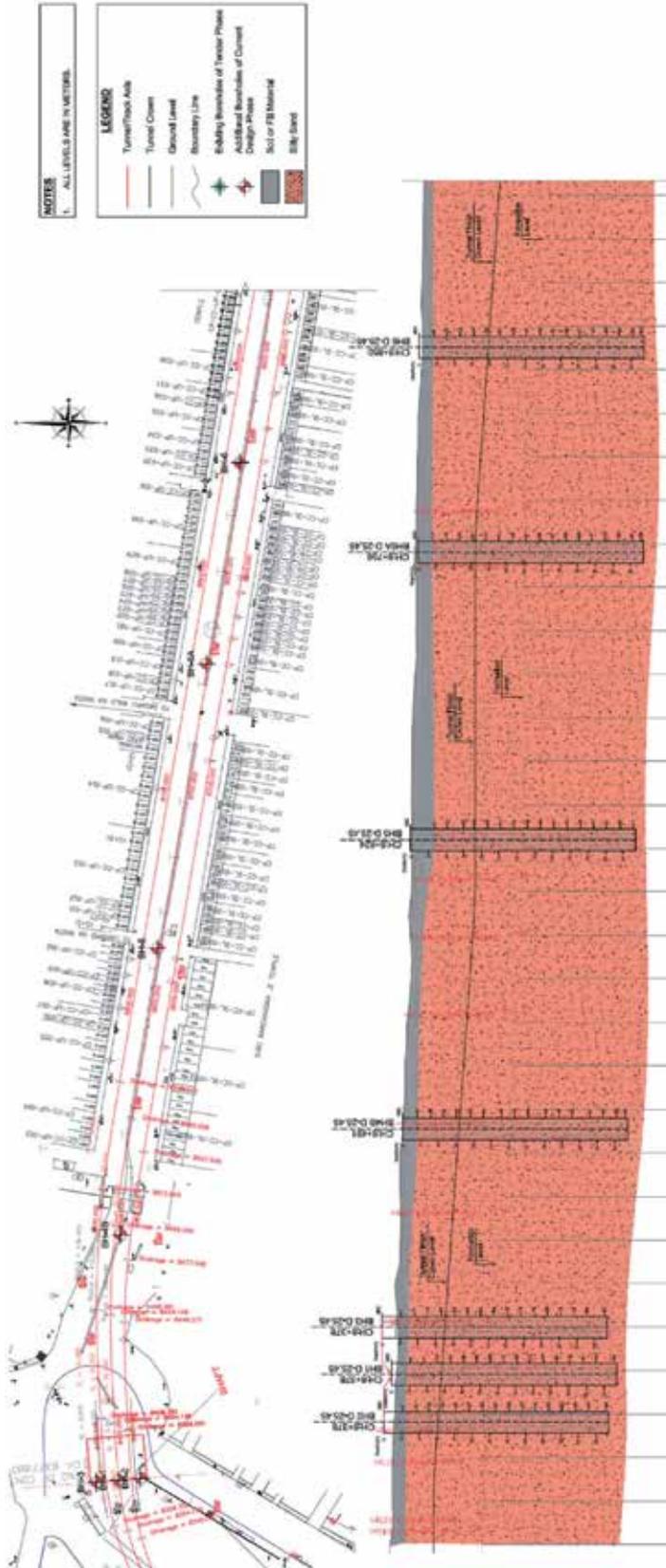
Since the tunnel excavation with EPB shields is a short-term procedure, the properties of the soil should be addressed in a careful manner in order to describe the actual geotechnical conditions. Continuous support on the tunnel face is applied by using the freshly excavated soil, which completely fills up the work chamber (muck). The supporting pressure is achieved through control of the incoming and outgoing materials in the chamber, i.e., through regulation of the screw conveyor rotation speed and of the excavation advance rate.

Underestimation of the mechanical characteristics of the soil medium and especially underestimation of the modulus of elasticity  $E$  can lead to the calculation of unreasonable and extremely high values of support pressure, with detrimental effect on the operation of the machine, as demonstrated below:

- a. The muck exhibits a shear resistance that, for a given internal friction angle, increases with the support pressure. Since muck with shear resistance does not behave like a fluid, the stress field in the work chamber, and thus the distribution of the support pressure along the tunnel face, is not under control (**Figure 10a**).

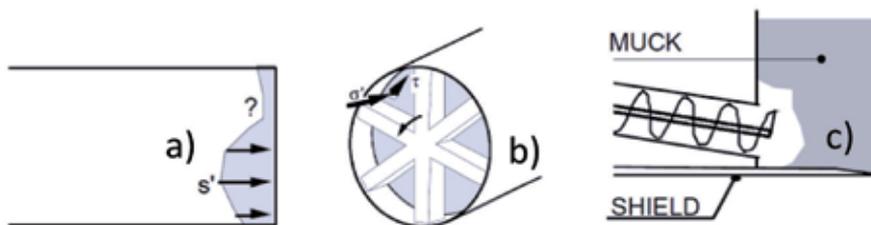


**Figure 7.** Geological map of greater area of Jaipur—Rajasthan (Source—Geological Survey of India) (Jaipur Metro Phase 1B area marked with magenta circle).



**Figure 8.** Geological longitudinal section of shallow tunnel in Jaipur Metro Phase 4B, with boreholes [4]—Silty sand along the entire alignment marked with orange color.





**Figure 10.**  
 Typical problems caused by high support pressure in EPB shields.

This is a nonoptimal situation from a stability point of view. The problem of nonuniform distribution of support pressure becomes even worse in the case of a mixed tunnel face due to the widely differing stiffnesses of the rock and soil layers.

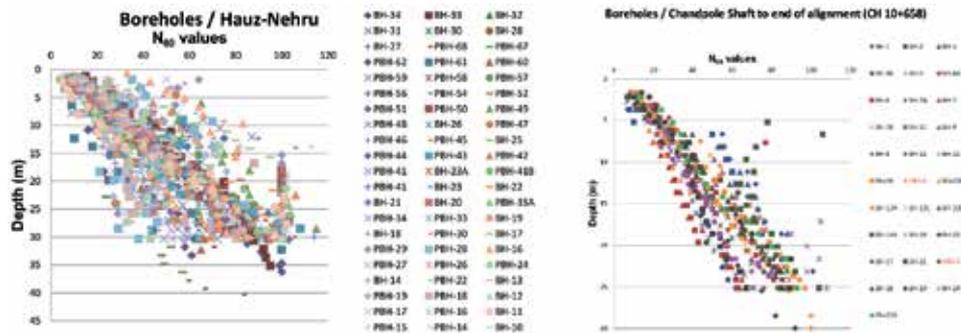
- b. For a full work chamber, the cutting wheel not only separates the ground from the tunnel face but also shears the compacted muck during the rotation. The larger the support pressures in the work chamber, the larger the shear resistance that must be overcome (**Figure 10b**). Thus, larger support pressures result in greater torque and in excessive cutterwear.
- c. For high support pressures and low clay percentages, arching of the muck occurs at the entrance to the screw conveyor and inhibits further discharge. In case that the excavation continues with impeded discharge, the muck becomes further compacted. It will be obvious that a silty, sandy soil is especially susceptible to arching and that the cutting wheel could then be brought to a complete halt due to the very high necessary torque (**Figure 10c**).

Conservatism in determining the geotechnical properties of the soil not only can result in overdesigning the permanent segmental lining of the tunnel but also can lead to unreasonably high support pressures with immediate detrimental effect on the operation of the machine. Furthermore, for very shallow tunnel cases, the blowout is also a case that needs to be carefully studied before determining the final EPB support pressure.

For soft soil formations, modulus of elasticity can be evaluated on the basis of the SPT results and the respective uncorrected  $N_{60}$  value, as per CIRIA report 143. Based on this report, the consistency of soils is determined by SPT N values as illustrated in **Table 1**.

Cohesionless soils		Cohesive soils	
SPT N (uncorrected value)	Classification	SPT N (uncorrected value)	Classification
0–4	Very loose	0–4	Very soft
4–10	Loose	4–8	Soft
10–30	Medium	8–15	Firm
30–50	Dense	15–30	Stiff
>50	Very dense	30–60	Very stiff
>60	Hard	>60	Hard

**Table 1.**  
 Consistency of soil formation based on SPT N values as per CIRIA, R143.



**Figure 11.** Characteristic plots showing the variation on SPT  $N_{60}$  value with depth along the tunnel alignment. Delhi Metro (CC23), left plot; Jaipur Metro Phase 1B, right plot.

In order to establish the representative SPT  $N_{60}$  values along the tunnel, the respective values should be plotted versus depth for the entire alignment considering the results from all the boreholes (**Figure 11**).

Further plots can also be prepared for different stretches, e.g., considering the boreholes results of the same stretch (between two stations). With this approach the entire tunnel stretch can be divided in sub-areas, and different fit lines can be determined as linear equations linking the representative  $N$  value with depth (e.g.,  $N = 2.8z + 5$ , whereas  $z =$  depth below ground surface). Therefore, by considering the CIRIA report, the modulus of elasticity can be calculated versus the SPT  $N_{60}$  values. As an example, for silts, sandy silts, and silty sands, the equation  $E = 0.7\text{--}1.0 N_{60}$  can determine the modulus of elasticity. For the derivation of the cohesion and friction angle, consolidated undrained triaxial strength tests (CU) when possible and direct shear strength tests can be used.

As already mentioned, underestimation of the soil properties when calculating the EPB pressure can lead to excessive problems during construction of the tunnel. Therefore, it is essential to study in detail the soil behavior not only through the results of the laboratory tests but also by investigating physical exposed open cuts in order to understand the “stand-up time” of the formations. “Stand-up time” is the short-term stability of the soil without the application of support measures and mainly reveals the existence of increased shear strength on the formation which sometimes is difficult to be determined by laboratory tests.

As a conclusive remark, it is worth mentioning that when designing the excavation of a tunnel with shield machine, it is proper to use the upper values of the soil parameters, while when designing the permanent lining, lower values can be used with caution to avoid unnecessary overdesign.

### 3.2 EPB-TBM principles

The tunnel boring machines that provide immediate peripheral and frontal support simultaneously belong to the closed-face group. They excavate and support both the tunnel walls and the face at the same time. Except for mechanical support machines, they all have the, so-called, cutterhead chamber at the front, separated by the remaining part of the machine by a bulkhead, where a confinement pressure is maintained in order to actively support the excavation and/or balance the hydrostatic pressure of the groundwater. The TBM is moving forward through hydraulic cylinders that are pushing the already erected segmental lining (**Figure 12**). Respective video for the operation of an EPB can be found in the following link from Herrenknecht: <https://www.herrenknecht.com/en/products/productdetail/epb-shield/>.



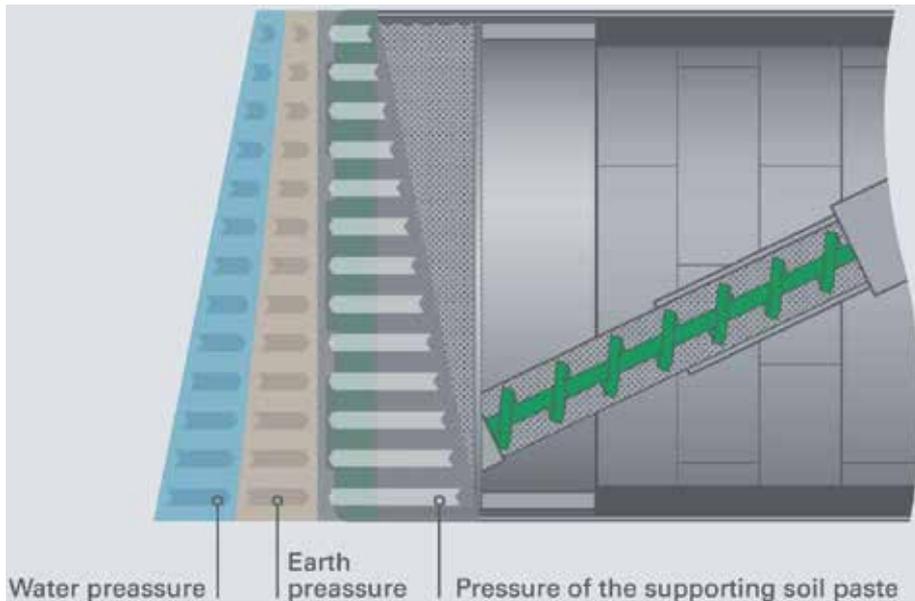
**Figure 12.**

*Typical view of earth pressure balance (EPB) shield (Source: Herrenknecht).*

With earth pressure balance shields, the cohesive soil loosened by the cutting wheel serves to support the tunnel face, unlike other shields which are dependent on a secondary support medium (e.g., slurry shields). The area of the shield in which the cutting wheel rotates is known as the excavation chamber and is separated from the section of the shield under atmospheric pressure by the pressure bulkhead. The soil is loosened by the cutters on the cutting wheel, falls through the openings of the cutting wheel into the excavation chamber, and mixes with the plastic soil already there. Uncontrolled penetration of the soil from the tunnel face into the excavation chamber is prevented because the force of the thrust cylinders is transmitted from the pressure bulkhead onto the soil. A state of equilibrium is reached when the soil in the excavation chamber cannot be compacted any further by the native earth and water pressure. The excavated material is removed from the excavation chamber by an auger conveyor (screw of Archimedes). The amount of material removed is controlled by the speed of the auger and the cross section of the opening of the upper auger conveyor driver. The auger conveyor conveys the excavated material to the first of a series of conveyor belts. The excavated material is conveyed on these belts to the so-called reversible conveyor from which the transportation gantries in the backup areas are loaded when the conveyor belt is put into reverse. The tunnels are normally lined with steel- or fiber-reinforced lining segments, which are positioned under atmospheric pressure conditions by means of erectors in the area of the shield behind the pressure bulkhead and then temporarily bolted in place. Mortar is continuously forced into the remaining gap between the segments' outer side and the rock through injection openings in the tailskin or openings directly in the segments.

The principle of EPB-TBM operation is that pressurizing the spoil held in the cutterhead chamber to balance the earth pressure exerted holds up the excavation. If necessary, the bulkhead spoil can be made more plastic by injecting additives from the openings in the cutterhead chamber, the pressure bulkhead, and the muck-extraction screw conveyor. By reducing friction, the additives reduce the torque required to churn the spoil, thus liberating more torque to work on the face. They also help maintain a constant confinement pressure at the face. The hydrostatic pressure is withstanding by forming a plug of confined earth in the chamber and screw conveyor; the pressure gradient between the face and the spoil discharge point is balanced by pressure losses in the extraction and pressure relief device (**Figure 13**).

Face support is uniform. It is obtained by means of the excavated spoil and additives. Injecting products through the shield can enhance additional peripheral support. For manual work to proceed in the cutterhead chamber, it may be necessary to create a sealing cake at the face through controlled substitution (without loss of confinement pressure) of the spoil in the chamber with bentonite slurry. The architecture of this type of TBM allows for rapid changeover from closed mode to open mode operation and vice versa. The tunnel lining is erected inside the TBM tail skin, with a tail skin seal, ensuring there are no leaks. Back grout is injected behind the lining as the TBM advances.



**Figure 13.**  
*Earth pressure balance principle (Source: Herrenknecht).*

Following the EPB operation principles, it is obvious that the tunnelling-induced soil movements can occur at the longitudinal and radial direction. The face extrusion can give rise to longitudinal displacements, while the gap around the shield and the tail of the machine will introduce radial convergence. The volume of soil that intrudes into the tunnel owing to the pressure release at the excavated face will be excavated eventually. This movement of soil is defined as the volume of material that has been excavated in excess of the theoretical design volume of excavation and is called “ground loss” or “volume loss.” Therefore the volume loss for a tunnel excavated with TBM occurs in three stages:

- Face loss (longitudinal ground movement into the tunnel face)
- Shield loss (radial ground movement into the gap created by the TBM overcut)
- Tail loss (due to the gap closure at the tail).

Since the above ground losses are related with the cavity displacements, consequently this is the mechanism that will give rise to the development of the surface settlements. Designing a TBM tunnel means control of the volume loss, and this can be achieved with:

- Proper calculation and application of face support
- The earth pressure support that is applied at the tunnel face is also transferred to the shield gap (thus around the shield), through the cutterhead openings, and therefore reduces the shield’s radial displacements
- Injecting products (e.g., bentonite) through the shield openings can also enhance additional peripheral support
- Tail skin grouting to seal the gap between erected segments and the tunnel’s excavated profile will minimize the so-called development of secondary settlements

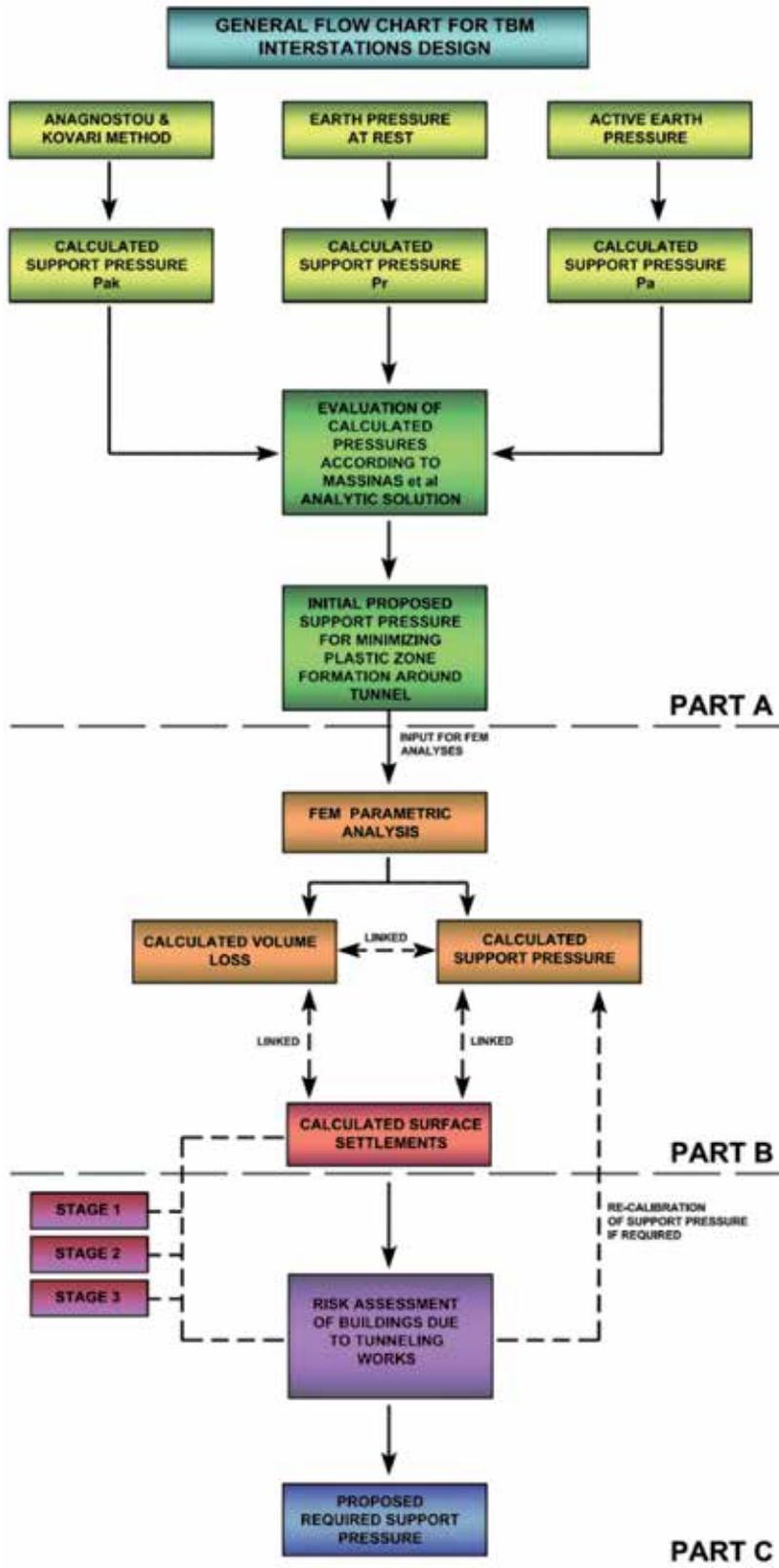


Figure 14. Overview of a TBM tunnel interstation design.

### 3.3 TBM tunnel interstation design (TID)

Following the finalization of the ground design parameters, documented in the Geotechnical Interpretation Report (GIR), the design for the excavation and support of the tunnel with the TBM will follow and is called TBM tunnel interstation design. The scope of the TBM TID is the complete tunnel boring design in order to ensure, the safety of the tunnel structure itself, the surface and subsurface structures adjacent to the project and, finally, the confinement of the ground deformations within permissible limits. A TBM TID (**Figure 14**) is divided in the following parts:

- Calculation of the required TBM support pressure. The required support pressure per ring, to be applied by the EPB-TBM on the excavation face and consequently around the shield gap, which is produced due to the overcut.
- Calculation of the ground surface settlements contours within the influence zone of the tunnelling works.
- Risk assessment of the existing structures due to tunnelling works, through different staged analysis (stage 1–3).
- Determination of protection measures for existing structures, if required according to the results from the staged analysis.

According to the analysis methodology that is related with the control of the induced surface settlements, the following basic phases are considered in a TBM TID:

- i. *Evaluation phase*: At the current stage, the geotechnical-geomechanical design parameters of the different formations need to be examined and evaluated, which are the input data for describing the initial conditions of the problem. All the above are always given in the Geotechnical Interpretation Report.
- ii. *Diagnostic phase*: The tunnel face and cavity behavior is examined along the tunnel alignment for different overburden heights, since the face and cavity stability affects the development of the ground displacements and hence the induced surface settlements. Therefore in a TID, three different categories need to be considered in order to describe the excavation face reaction:
  - Category 1: “Stable,” elastic behavior
  - Category 2: “Stable for limited time,” elastic-plastic behavior with limited plastic zone width
  - Category 3: “Unstable,” elastic-plastic behavior with extended plastic zone width

Two categories to describe the deformation phenomena:

- Elastic
- Elastoplastic

And finally two categories to describe the cavity behavior:

- Stable
- Unstable

- iii. *Therapy phase*: According to the results derived from the previous phase (II), the required parameters (e.g., support pressure, TBM operation parameters) are determined in order to secure the face and cavity stability and thus to minimize the induced surface settlements.

After studying the tunnel alignment, the designer will define the critical sections of the project, which is a combination of ground properties, water table height, overburden height, axial distance between the two tunnel bores (in cases of twin bore tunnels), locations of the existing buildings, and important structures. The outcome of the study will determine the tunnel sections of the TID. In detail the characteristic tunnel sections that need to be used in a TID will include mainly the following cases:

- Section with the highest overburden
- Section with the shallowest overburden
- Section with the highest (expected) groundwater table
- Section with the lowest (expected) groundwater table
- Section with the highest loading acting on the ground surface
- Section with possible non-horizontal ground surface
- Section in which existing buildings or important structures exist
- Section in which adjacent present or future tunnel exists
- Section that includes the existence of possible future works (expect of tunnels)
- Sections with any changing geometry or geology

During the tunnel excavation, the development of the surface settlements will be recorded through an established geomechanical monitoring program, in order to check the predictions of the TID and to calibrate if required the operation parameters of the EPB machines (e.g., support pressure).

### *3.3.1 Part A: face pressure calculation*

The tunnel face and cavity reaction is always related with the following main key parameters:

- Ground mass geotechnical properties and water table level. “Output” of the GIR
- The overburden thickness. Determined based on the tunnel’s alignment
- Tunnel radius. Defined by the project’s requirements
- Tunnel construction methodology

Section	Overburden (m)	Ground condition	Water table	Existing structures above tunnel
F1	25	Delhi silt	Nil	≤5-storey
F2	22	Delhi silt	Nil	≤5-storey
F3	19	Delhi silt	Nil	≤5-storey
F4	17	Delhi silt	Nil	≤5-storey
F5	15	Delhi silt	Nil	≤5-storey
F6	13	Delhi silt	Nil	≤5-storey
F7	12	Delhi silt	Nil	≤5-storey
F8	11	Delhi silt	Nil	≤5-storey
U1	4.5	Delhi silt	Nil	Existing Metro Line-2 with tunnel invert 4.5 m above tunnel crown of CC23
U2	3.8	Delhi silt	At surface	Existing bridge and “Nala” (water canal) with bridge’s foundation level 3.8 m above tunnel crown

**Table 2.**  
Critical sections along CC23 of Delhi Metro Phase 3.

Section	Overburden (m)	Ground condition	Water table	Existing structures above tunnel
J1	7	Silty sand	Nil	Roadway
J2	10	Silty sand	Nil	Roadway
J3	12	Silty sand	Nil	Roadway
CH	4.5	Silty sand	Nil	Chandpole Gate with foundation level 4.5 m above tunnel crown

**Table 3.**  
Critical sections along Jaipur Metro Phase 1b.

For the determination of the required support pressure, different calculation sections are required to be defined as explained in the previous paragraph. To present the design methodology in more detail, the Delhi Metro Phase 3 and Jaipur Metro Phase 1b will be used as case studies hereafter.

Considering the geological longitudinal section of the contract CC23 of Delhi Metro Phase 3, snapshot of it presented in **Figure 5**, the critical sections defined along the entire alignment are given in **Table 2**.

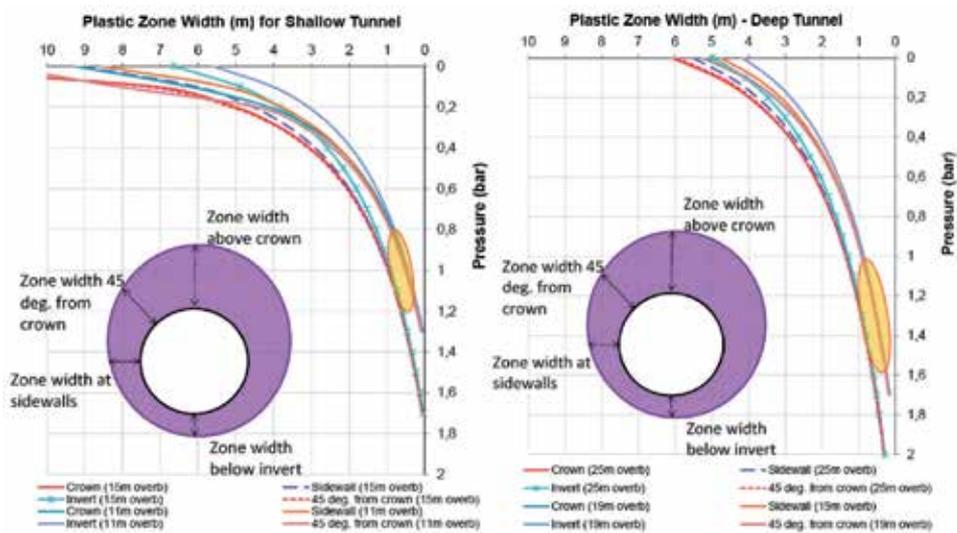
Considering now the geological longitudinal section of the Jaipur Metro Phase 1b, snapshot of it presented in **Figure 8**, the critical sections defined along the entire alignment are given in **Table 3**.

For all sections of **Tables 2** and **3**, where the tunnel alignment is below roadway and buildings with less or equal to 5 storeys, a uniform surface load of 50 kPa is considered. This approach is common when designing shallow tunnels in urban environment, since a minimum surface load is always required in order to consider any unforeseen load case or any low height (<5 storeys) future structure. Sections U1, U2, and CH are the most critical cases, since the existing structures are in very close proximity with the tunnel crown; thus special considerations will be presented hereafter.

The geomechanical parameters of the soils for the abovementioned Metro projects (Delhi and Jaipur) are also given in **Table 4**.

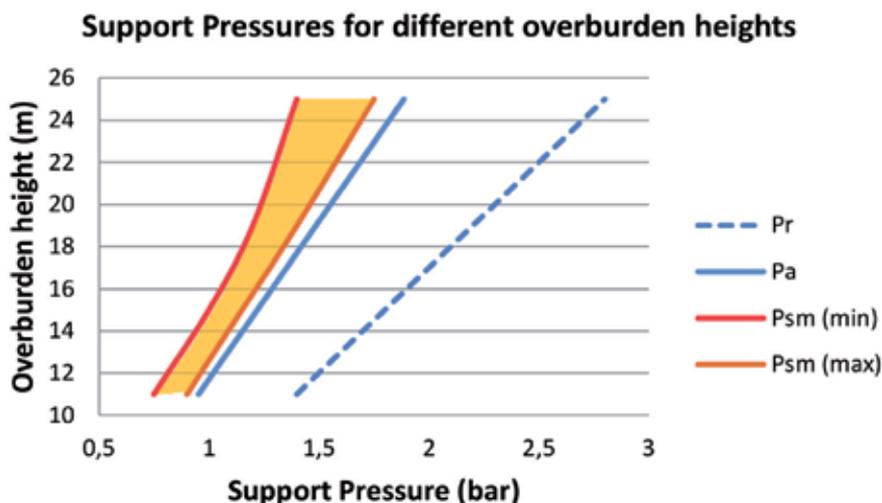
Design parameter	CC23 Delhi Metro Phase 3		Jaipur Metro Phase 1b	
Characteristic $N_{60}$ value	$2.8z + 5$ , $z =$ depth below ground surface (m)		$3.2z + 5$ , $z =$ depth below ground surface (m)	
Modulus of elasticity, $E$ [MPa]	$1.0 \times N_{60}$		$1.0 \times N_{60}$	
Effective friction angle, $\varphi'$ [°]	Depth 0–15 m	30	Depth 0–10 m	27
	Depth 15–25 m	32	Depth >10 m	30
	Depth 25–>30 m	34		
Effective cohesion, $c'$ [kPa]	Depth 0–7.5 m	0	Depth 0–>25 m	5
	Depth 7.5–15 m	10		
	Depth 15–25 m	20		
	Depth 25–>30 m	30		
Unit weight, $\gamma$ [kN/m <sup>3</sup> ]	20		19	
Coefficient of earth pressure at rest, $K_0$	0.61		0.57	
Poisson's ration, $\nu$	0.3		0.4	

**Table 4.**  
 Geotechnical design parameters.



**Figure 15.**  
 CC23 Delhi Metro. Calculation of plastic zone width according to Massinas and Sakellariou solution [6] for different tunnel depths (as per sections of Table 2). Proposed range of support pressure also shaded with orange color.

According to the international literature and case studies from various projects around the world, the applied support pressure for the tunnel construction with TBM shields is considered according to the earth pressure at rest and active earth pressure or according to Anagnostou and Kovari [5]. Recently, the Massinas and Sakellariou [6] analytic solution is also used to determine the required support pressure for EPB



**Figure 16.**

CC23 Delhi Metro. Calculated EPB support pressure for different overburden heights as per the critical sections F1–F8 of **Table 2**. Pressures  $P_r$  and  $P_a$  are calculated based on earth pressure at rest and active earth pressure, respectively.  $P_{sm} (min)$  and  $P_{sm} (max)$  give the range of calculated support pressure based on Massinas and Sakellariou closed form solution [6].

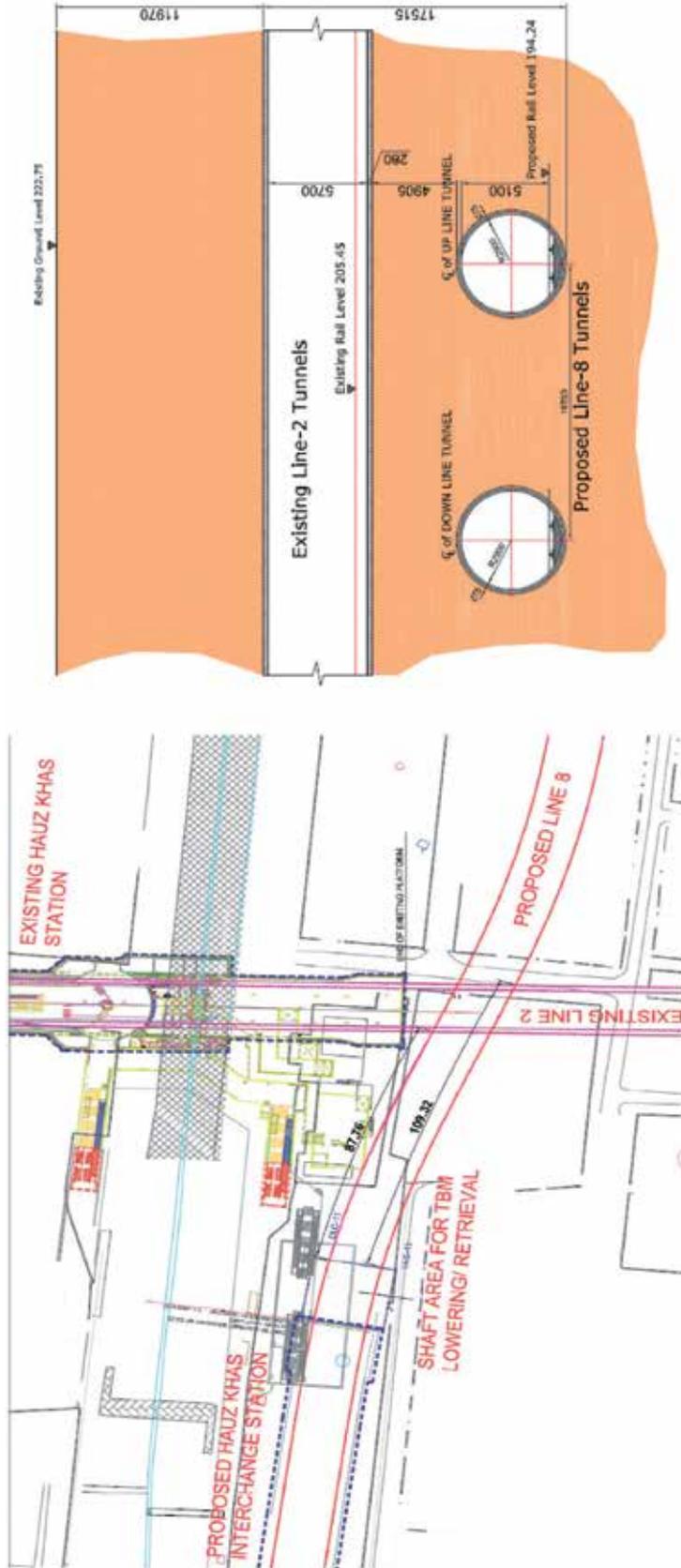
shields [7, 8]. Considering the geotechnical parameters of the soil, the Massinas and Sakellariou solution [6] is used to calculate the min and max value of the support pressure (**Figure 15**). The proposed range of pressure, given by Massinas and Sakellariou, coincides with minimum plastic zone width (less than 0.5–1 m). Since the aim is to minimize the induced surface settlements, controlling the plastic zone formation to minimum width can determine the required value of support pressure.

Considering also earth pressure at rest ( $K_0$ ) and active ( $K_a$ ), the EPB support pressure is calculated for all the different sections F1–F8 of **Table 2**, for CC23 Delhi Metro. All the calculation results are plotted in the diagram of **Figure 16**.

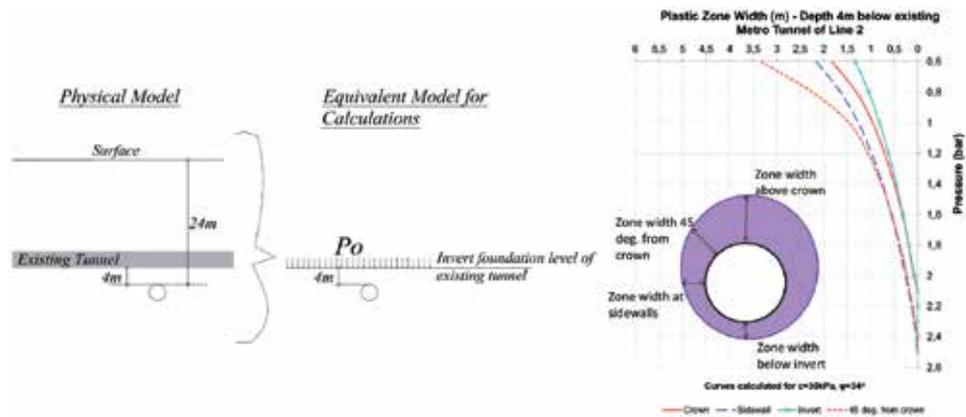
It is obvious that the calculation of  $P_r$  by considering the earth pressure at rest leads to non-pragmatic values, not feasible for the tunnel construction. On the other hand, the active earth pressure seems to give more realistic results since  $P_a$  is almost coincide with the upper values of the pressures range, as calculated according to Massinas and Sakellariou solution. Most realistic results can be derived by Massinas and Sakellariou analytic solution; since the soil-tunnel-surface interaction is considered and the plastic zone width is derived, therefore the proposed support pressure to be applied by the EPB is concluded, and the input for part B is defined. More details for the application of Massinas and Sakellariou method can be found in Refs. [6–8].

For the special case U1, the bored tunnels of CC23 (Line-8) underpass the existing tunnels of Line-2 as shown in **Figure 17**. The objective of securing the safe operation of the existing Metro Line-2 is related with the reduction of the soil deformations and thus minimizing the displacement of the existing segmental lining. In order to examine the soil-structure interaction by calculating the plastic zone formation around the tunnel and calculating the EPB support pressure ( $P_{sm}$ ) range, again the method of Massinas and Sakellariou needs to be applied, with the assumptions that follow.

The presence of the existing Line-2 is taken into consideration by assuming as an upper boundary of the semi-infinite space its invert foundation level; thus a total overburden of 4 m (conservatively instead of 4.5 m) is considered, to examine the interaction between the new and the existing tunnel (**Figure 18**—left); the total earth pressure (due to gravity) at the real depth of the tunnel is taken into account, by applying a uniform load ( $P_0$ ) at the upper boundary of the half-space.



**Figure 17.** Key plan and typical cross section of Line-8 bored tunnels under existing Metro tunnels of Line-2 [6, 7].



**Figure 18.** CC23 Delhi Metro. Physical and analytical model for calculating the required support pressure of Line-8 TBM under the existing Metro tunnels of Line-2 (left). Analytic calculation of plastic zone width (right) [6, 7].

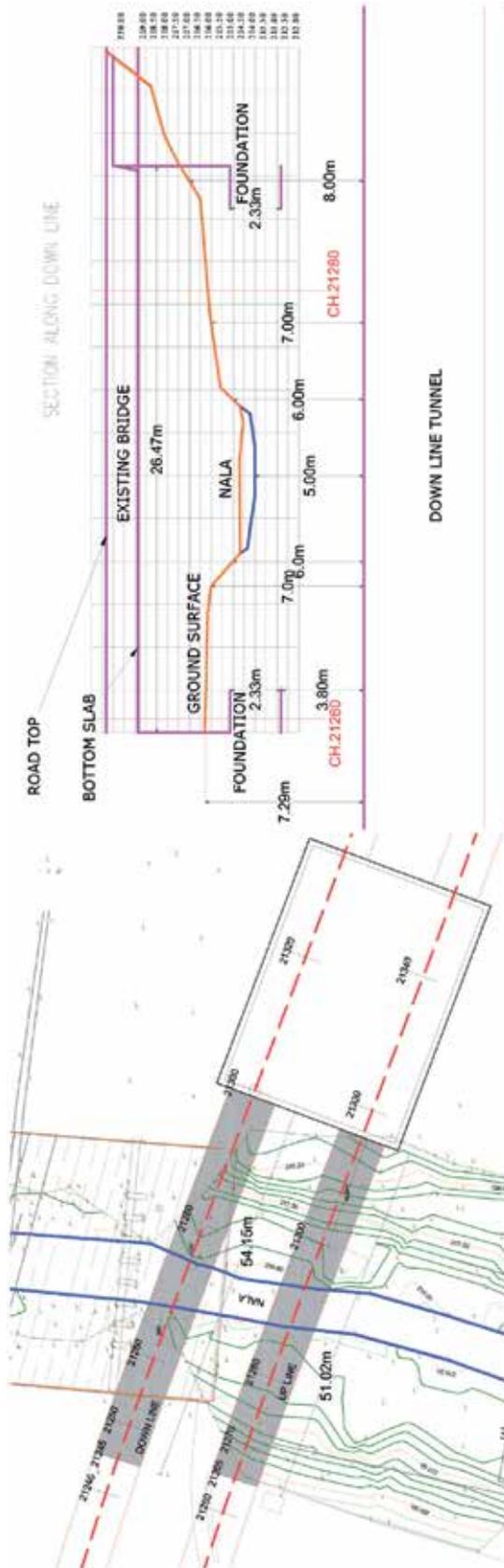
By using the analytic solution formula [6], the calculation of the plastic zone shape around the tunnel is performed for different values of the support pressure (**Figure 18**—right). Minimum plastic zone of less than 0.5 m is derived for support pressure range of 1.8–2.2 bar, while for 2.4 bar, the soil around the cavity remains within the elastic domain. Therefore by considering a support pressure within the range of 1.8–2.2 bar, the stress redistribution remains within the elastic domain, and thus minimum displacements are expected to be developed. The above-derived conclusion is the initial observation for the tunnel interactions and is further analyzed (part B of TID) in full detail through 3D elastic-plastic FDM multistaged simulations with powerful software Fast Lagrangian Analysis of Continua in 3 Dimensions (FLAC3D) [9].

For the second special case U2 of CC23 Line-8, the TBM working on the up line underpass the existing Nala, while for the construction of the down line, the TBM will underpass both the abutments of the existing bridge as well as the Nala. Therefore, the critical area is approximately 50 m, starting from the neutral zone (cut and cover structure). Typical plan view of the horizontal alignment and the longitudinal sections of the down line tunnel is given in **Figure 19**.

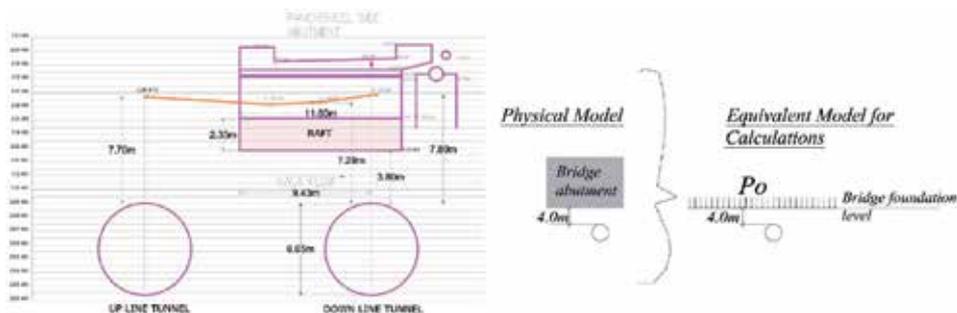
Special survey on the bridge’s foundation (trial pit) revealed pad foundation with thickness of approximately 2.33 m. The bridge is new, and the span is approx. 26 m with general dimensions in plan 35 × 14 m (length × width). The superstructure is made of prestressed reinforced concrete box and lay on the abutments through bearings, as presented in **Figure 20**.

As in the previous case U1, also for this critical section U2, the presence of the existing bridge is considered by assuming as an upper boundary of the semi-infinite space its foundation level; thus a total overburden of 4 m is considered, to examine the interaction between the bridge and the tunnel. Furthermore, the total earth pressure (due to gravity) at the real depth of the tunnel is also taken into account, and a total uniform load ( $P_o$ ) at the upper boundary of the half-space is applied (**Figure 20**—right). Considering the longitudinal section of the tunnel, different overburden heights are considered for the analytic calculations in order to determine the required support pressure. **Table 5** summarizes the calculated support pressure as per TID—part A for different overburden heights along the bridge and Nala area.

For the calculation of  $P_{sm}$  pressures, the same principle, as in previous case U1, is used. Thus, the required pressure is calculated in order to keep the plastic zone width around the tunnel below 0.5 m. As per part B of TID, further analysis of the



**Figure 19.** Horizontal alignment (left) of up and down lines in Nala (water canal) area. Longitudinal section (right) of down line shows tunnel under the bridge abutments and the Nala [40].



**Figure 20.** CC23 Delhi Metro Line-8. Cross section of Line-8 tunnels under existing bridge (left). Physical and analytical model for calculating the required support pressure of Line-8 TBM under the existing bridge (right) [10].

Pressure	11 m overburden (bar)	8 m overburden (bar)	Bridge location (bar)	Nala deepest part (bar)
$P_r$ (at rest)	1.7	1.3 bar	2.3 bar	1.0 bar
$P_a$ (active)	0.9	0.7	1.3	0.5
$P_{em}$ (Massinas and Sakellariou)	1.0–1.2	0.8–1.0	1.5–1.7	0.6–0.8
Adopted pressure for part C of TID	1.0	1.0	1.5	1.0

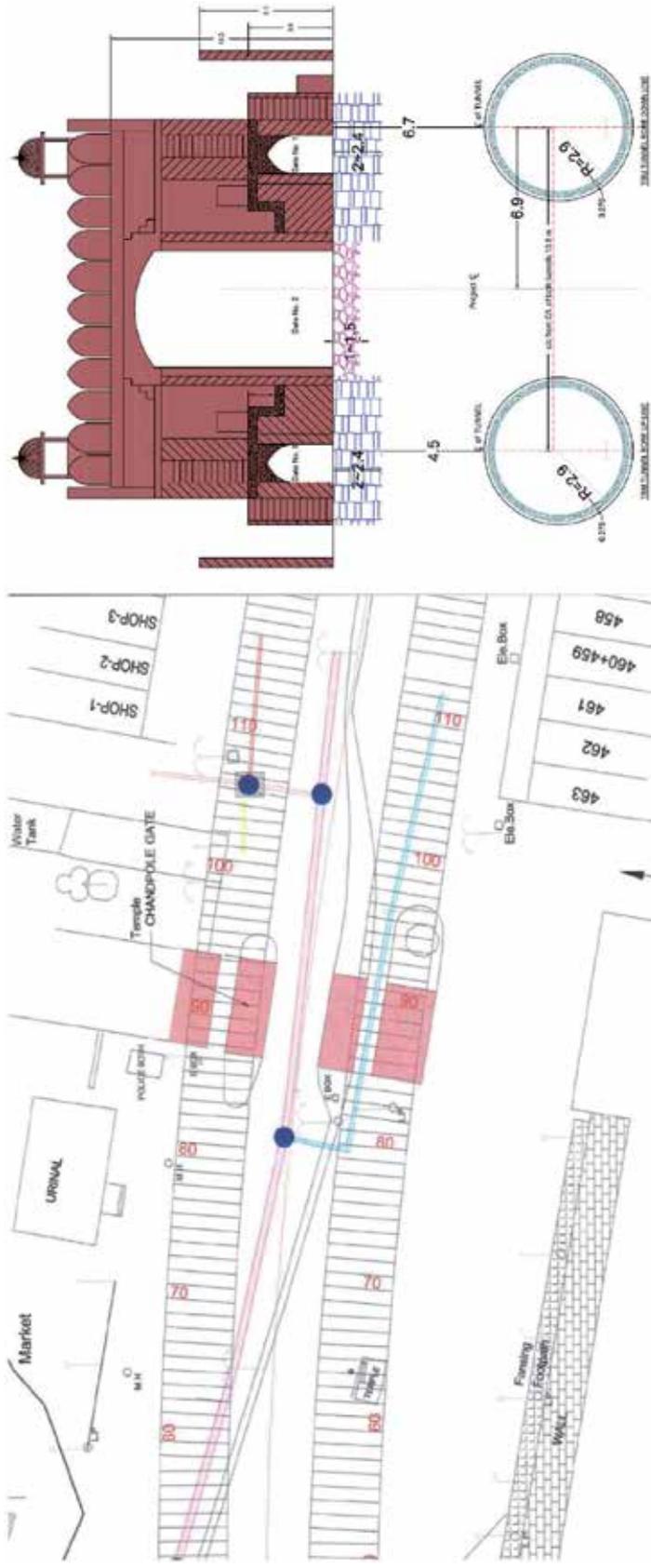
**Table 5.** Support pressure along tunnel alignment, for different overburden heights.

settlement development is required to be elaborated through 3D calculations with FLAC3D [9], considering the different support pressures given in **Table 5**.

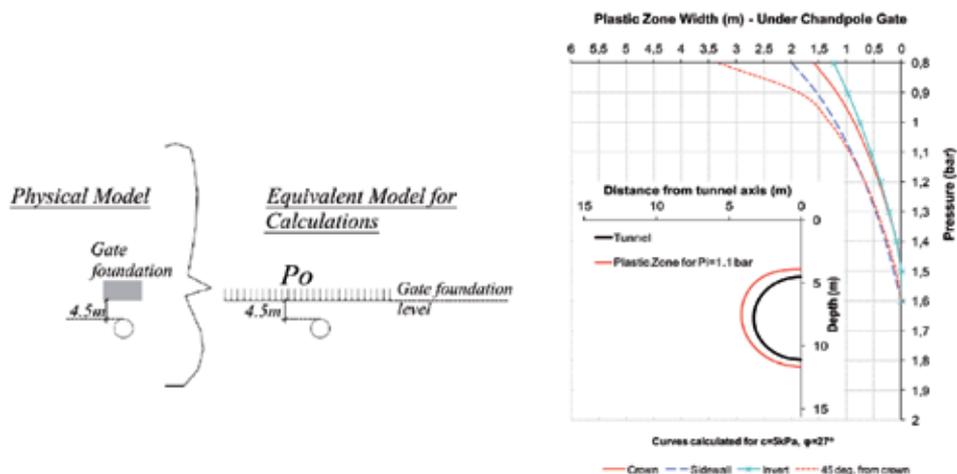
For the case of Jaipur Metro Phase 1b, the critical section is more complex than in previous examples from Delhi Metro. The importance of the monument of Chandpole Gate and the extremely shallow depth of the tunnels (**Figure 21**) required further study to determine the most appropriate method for underpassing with the TBM.

For the determination of the structure’s foundation system, dimensions and condition, special survey was carried out, consisting of nine trial pits that were executed in certain locations near the Gate. According to the findings from the trial pits on both sides of the structure, the Gate rests on stone masonry foundation with depth of approx. 2.0–2.4 m, whereas below the main arch, a boulder layer was found with depth ~1.5 m near the Gate’s walls and ~1.0 m below the mid part. Water pipes and other utilities were found below the two passageways and the main arch. A typical section of the Gate with its foundations and the underpassing TBM tunnels is given in **Figure 21** (right), indicating the soil cover of approx. 4.5 m between the bottom of the Gate’s foundation and the tunnels’ crown.

Examination and co-evaluation of the geotechnical investigation and Gate foundation survey results was jointly performed by the designer and the contractor, aiming to decide on the necessity of soil strengthening through grouting below and around the Gate’s foundation. The grain size distribution of the in situ soil practically excluded the successful and efficient application of low-pressure cement grouting, even with use of microfine cement. Other grouting methods that would be more appropriate for this soil type, e.g., jet grouting and compensation grouting, were excluded, as the risk of soil disturbance, instability, temporary liquefaction, and settlement below the Gate foundation was considered too high for



**Figure 21.** Jaipur Metro Phase 1b. Horizontal alignment and typical cross section of TBM tunnels under Chandpole Gate [8].



**Figure 22.** Jaipur Metro Phase 1b. Physical and analytical model for TBM support pressure calculation under Chandpole Gate (left). Analytic calculation of plastic zone width (right) [8].

Section	Overburden (m)	Ground condition	Adopted support pressure (bar)	Settlement analysis method
F1	25	Delhi silt	1.5	3D FDM with FLAC3D
F2	22	Delhi silt	1.4	3D FDM with FLAC3D
F3	19	Delhi silt	1.2	3D FDM with FLAC3D
F4	17	Delhi silt	1.1	3D FDM with FLAC3D
F5	15	Delhi silt	1.0	3D FDM with FLAC3D
F6	13	Delhi silt	0.8	3D FDM with FLAC3D
F7	12	Delhi silt	0.8	3D FDM with FLAC3D
F8	11	Delhi silt	0.8	3D FDM with FLAC3D
U1	24/4.5	Delhi silt	1.8	3D FDM with FLAC3D
U2	11	Delhi silt	1.0	3D FDM with FLAC3D
	8	Delhi silt	1.0	3D FDM with FLAC3D
	Bridge	Delhi silt	1.5	3D FDM with FLAC3D
	Nala	Delhi silt	1.0	3D FDM with FLAC3D

**Table 6.** Critical sections along CC23 of Delhi Metro Phase 3 with proposed support pressure for settlement analysis.

the significance and vulnerability of the structure. Additionally, the loading of the structure itself and the long period of its application was naturally assumed to have made the soil below the Gate compact. This assumption could be partly verified by the soil inspection in the trial pits. In the light of the above considerations, the designer and the contractor decided to underpass the structure without any pre-treatment of the soil, relying on the appropriate TBM operation, mainly in terms of applied face pressure and annular gap grouting application.

Following the same procedure as for the Delhi Metro cases, the presence of the existing Gate was considered by assuming its foundation level as an upper boundary of the semi-infinite space. Thus a total overburden of 4.5 m has been considered to examine the interaction between the Gate and the tunnel, as presented in **Figure 22**. Furthermore, the total earth pressure (due to gravity) at the real depth

Section	Overburden (m)	Ground condition	Adopted support pressure (bar)	Settlement analysis method
J1	7	Silty sand	1.1	3D FDM with FLAC3D
J2	10	Silty sand	1.4	3D FDM with FLAC3D
J3	12	Silty sand	1.5	3D FDM with FLAC3D
CH	Chandpole Gate/4.5	Silty sand	1.5	3D FDM with FLAC3D

**Table 7.**  
 Critical sections along Jaipur Metro Phase 1b with proposed support pressure for settlement analysis.

of the tunnel is also considered, and a total uniform load ( $P_o$ ) at the upper boundary of the half-space is applied. The comparison between the physical model of the problem and its equivalent model for the analytical calculations is illustrated in **Figure 22** along with the results derived by the application of Massinas and Sakellariou analytic solution.

A minimum plastic zone width ( $<0.5$  m) is calculated for a mean support pressure within the range of 1.4–1.5 bar, while for 1.6 bar, the soil around the excavation remains within the elastic domain. The value of 1.5 bar was selected for the execution of 3D numerical analyses for settlement prediction.

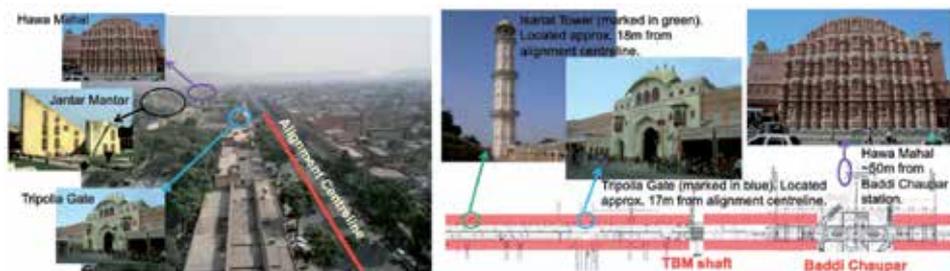
To summarize the results from part A of TID, **Tables 6** and **7** give the final proposed TBM support pressures used for the settlement analysis in part B of TID.

For all the above cases as summarized in **Tables 6** and **7**, the adopted support pressure for the settlements analysis is based on the calculation results as per Massinas and Sakellariou solution [6]. Comparing the two tables, it is obvious that for the case of Jaipur Metro Phase 1b, higher pressures are considered, which resulted from the requirement to keep the plastic zone width around the tunnel less than 0.5 m since the entire tunnel alignment is very shallow (7–12 m) compared to CC23 Delhi Metro.

Furthermore, even along the Jaipur Metro Phase 1b, the tunnel alignment is below the busy roadway; in close vicinity with the tunnel’s sides, the very old buildings of the “Pink City” and other important monuments exist (**Figure 23**), and thus the settlement needs to be kept to absolute minimum values to avoid any damages.

### 3.3.2 Part B: surface settlements

Following the derivation of the support pressures in part A, settlement analysis is required in part B of TID in order to numerically calibrate the range of the derived support pressures. The outcome from this step of the design will finalize the set of values to be used during construction that will result to minimum volume loss and consequently will reduce the magnitude of the surface settlements. Since the

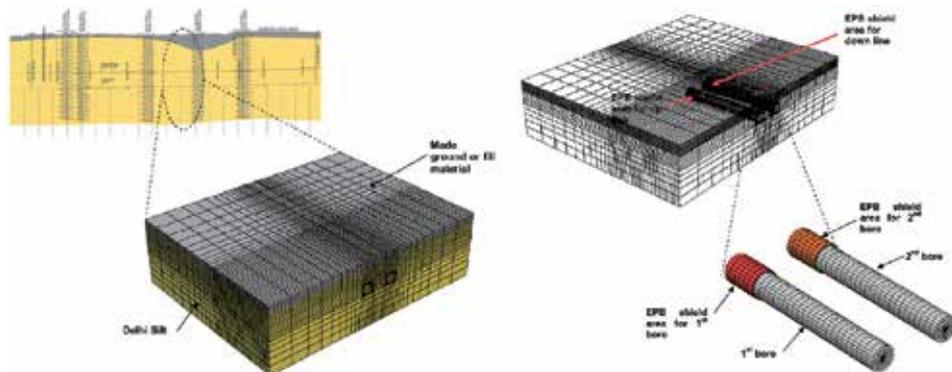


**Figure 23.**  
 Jaipur Metro Phase 1b. Important monuments of the “Pink City” along the tunnel alignment [12].

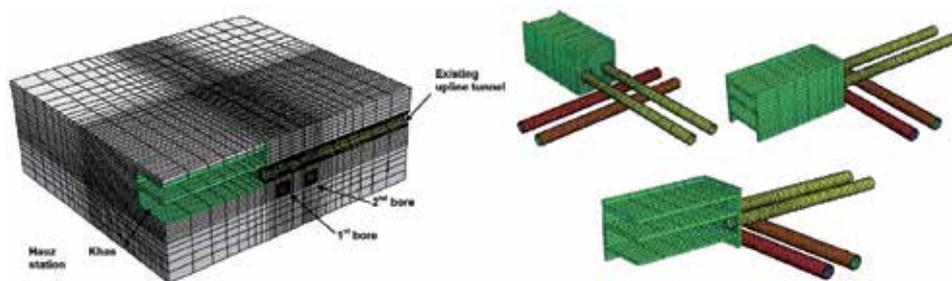
“geometric constraints” of the problem is bounded with the construction sequence of the tunnel (excavation-segment installation-grouting) that takes place near the face, the most effective type of analysis in order to investigate the face behavior and the cavity convergence around the shield is to perform 3D calculations.

Assuming stress-induced failure mode of ground mass, the computational three-dimensional analysis is performed with a continuum model approach, using the finite difference software FLAC3D [9]. In order to ensure the accuracy of the results and to avoid any “boundary effects,” certain parametric 3D analyses are performed (benchmark tests) with different 3D meshes in dimension and density. In all vertical boundaries, the horizontal movements normal to the boundary were restricted. On the bottom boundary, soil movements were restricted in all directions. In order to avoid any effects on the surface settlements shape and magnitude, due to boundary positions, the proper grid dimensions were implemented in the models. Thus, for the vertical boundaries on each side of the tunnels, a distance of 10 x diameters is selected. For the extension of the grid below the tunnel spring line, a distance of 2 x diameters is considered. The distance between the vertical boundary normal to the tunnel direction in front of tunnel’s face and the final excavation face is selected equal to 10 x diameters [13]. Typical views of the models are presented in **Figure 24**.

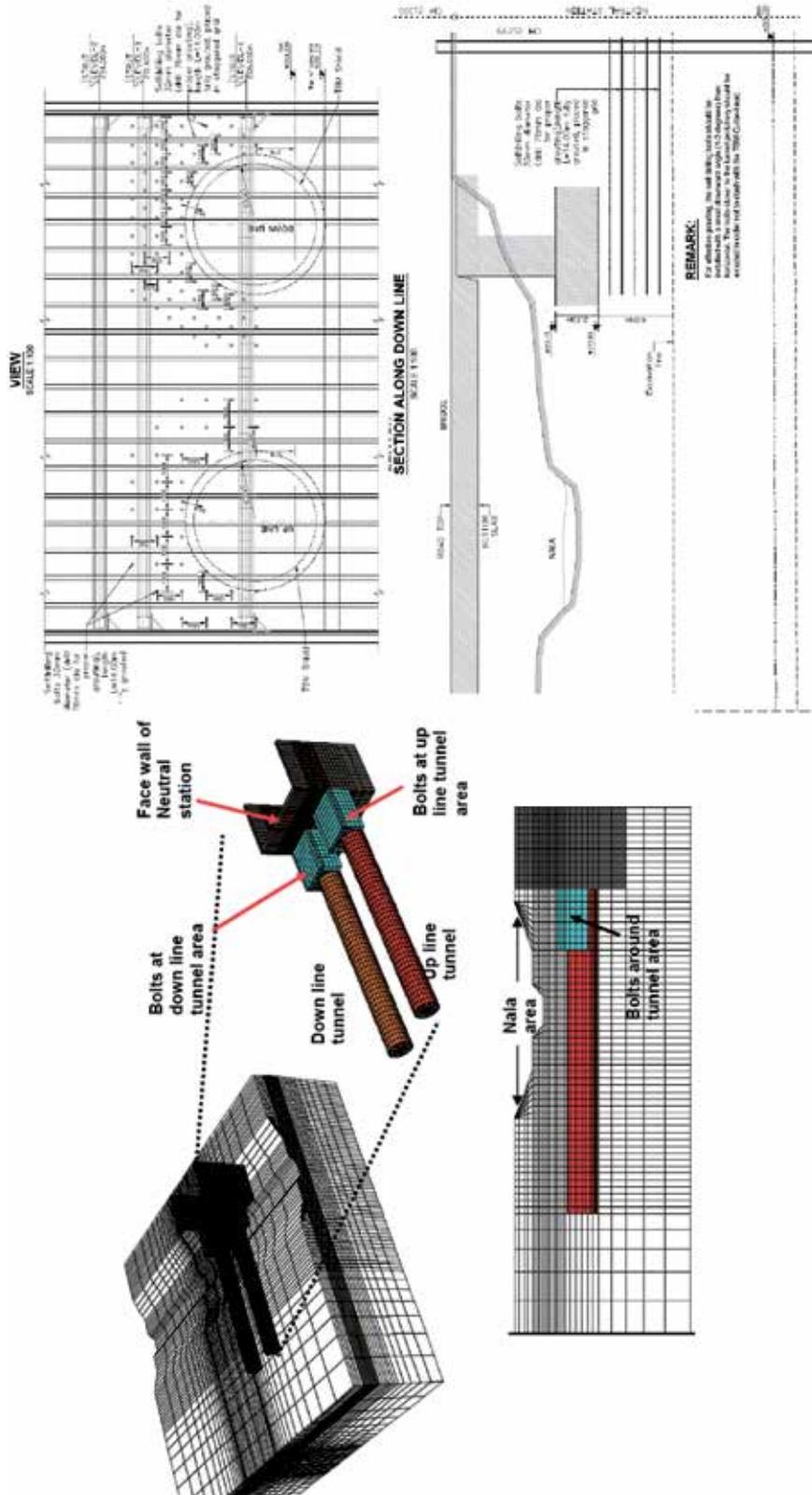
For the special cases U1, U2, and CH, different models were constructed following the same boundary conditions and dimensions as presented before. For the underpass of Yellow Line (U1), both the existing Line-2 and the Hauz Khas station (in close vicinity with the tunnels) were simulated (**Figure 25**).



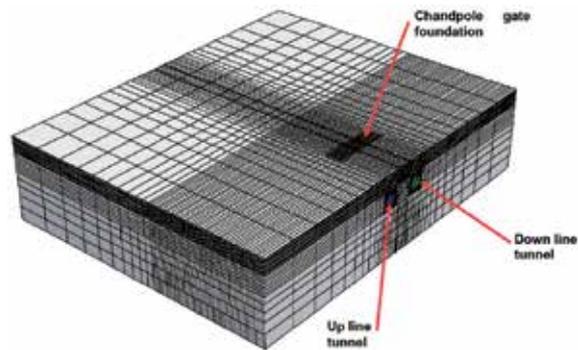
**Figure 24.** Typical FDM models used for settlement analysis. CC23 Delhi Metro on the left and Jaipur Metro Phase 1b on the right [11, 12].



**Figure 25.** Section U1: underpass of Line-2. Typical FDM model used for the settlement analysis. CC23 Delhi Metro [14].



**Figure 26.** Section U2: underpass of bridge and Nala. Typical FDM model used for the settlement analysis (left). Additional support measures for soil strengthening under the bridge's abutment (right). CC23 Delhi Metro [10].

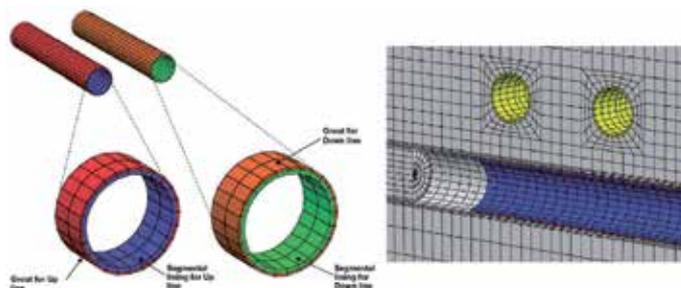


**Figure 27.** Section CH: underpass of Chandpole Gate. Typical FDM model used for the settlement analysis. Jaipur Metro Phase 1b [15].

For the settlement analysis at bridge and Nala area (U2), the 3D model considered the cut and cover structure of the neutral station the bridge's foundation. The anaglyph of the Nala with the physical slopes is also detailed simulated (**Figure 26**). Due to the fact that the tunnel excavation below the Nala area and the bridge would start from neutral station, additional support measures are required for strengthening the soil (in front of the launching area) during the TBMs entrance in the ground, since at this stage there will not be adequate support pressure. Therefore, fully grouted self-drilling bolts 14 m in length 32 mm in diameter are foreseen to be installed perpendicular to the face wall of the neutral station, around the tunnel excavation area (**Figure 26**). The respective support measures have been also simulated in the respective 3D analysis as illustrated in **Figure 26**.

The last simulation (CH) is related with the settlement analysis of Chandpole Gate during the TBM tunnel boring below the monument. Detailed 3D model prepared simulating the Gate's foundations and the sequential construction of the tunnels with the two TBM shields (**Figure 27**).

In all the 3D analyses, the exact geometry of the segments and the annular gap of the tail shield grouting were simulated as presented in **Figure 28**. The excavation simulation is sequential, and in each step certain actions are considered. The advance of the TBM can be simulated either by considering the total length of the shield (e.g., 9 m in length) as one excavation step or slower advance of the shield equal to one segment can also be considered in the simulation. In the first case, the total load cases can be reduced, and thus the calculation time can be significantly



**Figure 28.** Typical 3D view (left) of segmental lining and grout for backfilling the annular gap simulation. Typical 3D view (right) of segmental lining (blue) and grout (red) applied on soil (white) [11, 12].

minimized. Both methods of simulating the TBM advancement will give reliable results. It is preferable in cases of critical underpasses (e.g., U1, U2, and CH) for the excavation step to follow each segment length. Therefore, in such cases the analysis can commence with an initial excavation step equal to the shield's length, and as the TBM simulation reaches the important structure, the excavation step is to be reduced and should follow the segment length.

In each excavation step simulation, the EPB mean support pressure (as calculated in part A of TID) is applied at the tunnel face. Around the shield the same pressure with the face can be applied either with a triangular distribution reaching zero value on the tail of the shield or uniformly up to the half-length of the shield. Both methods are feasible, and the designer can decide based on the abilities of the software that he uses. In the previous step (behind the excavation), green grout (a modulus of elasticity equal to 1 GPa or less can be used) to simulate the backfill on the annular gap is applied along with the segmental lining. Two steps behind, the grout mature is simulated by applying its final properties (modulus of elasticity equal to 10 GPa). There are also other methods to simulate the TBM excavation, for example, interface elements can be used in order to simulate the behavior of the shield gap or the annular gap around the segments. Those methods need advance modelling and increased computational time, which are more appropriate for academic research and are not common practice in the design industry, since there are many unknown parameters that need to be defined in order to assign the correct properties to the interface elements.

In order for the calculated surface settlements to reach equilibrium behind the shield, a minimum excavation simulation of total tunnel length equal to 5 x diameters is mandatory. For special cases additional excavation length may be required to be simulated. In **Table 8** the total simulated length of tunnel excavation is given for all the performed 3D analyses along with geometrical required information.

In all the 3D analyses performed (**Table 8**), the plastic zone width was calculated below 0.5–1.0 m (**Figure 29**). It is obvious that this value was expected as it was initially derived based on Massinas and Sakellariou solution [6].

As it is evident from the diagram in **Figure 30**, the simulated support pressure ( $P_{fdm}$ , black dotted line) in the FDM 3D analysis is within the lower range of the Massinas and Sakellariou support pressure. The higher the overburden, the higher the pressure required to support the tunnel excavation, following second-order polynomial curve (e.g., the equation of trendline is  $y = 8.89x^2 - 0.4x + 5.67$ ). On the contrary, the calculation of the support pressure, based on the earth pressure at rest, follows, as expected, linear line that results to unrealistic values, the application of them can jeopardize the effective operation of the TBM.

Section	Nos of octahedral elements	Tunnel axial distance (m)	Tunnel excavation radius (mm)	Tunnel internal radius (mm)	Segment ring length (mm)	Ring thickness (mm)	Total excavation length simulated (m)
F1–F8	65,000	15.5	3320	2900	1500	280	45
J1–J3	70,000	16.0	3320	2900	1200	280	48
U1	150,000	15.5	3320	2900	1500	280	108
U2	120,000	15.5	3320	2900	1500	280	88.5
CH	70,000	14.5	3320	2900	1200	280	48

**Table 8.**  
3D models simulation properties.

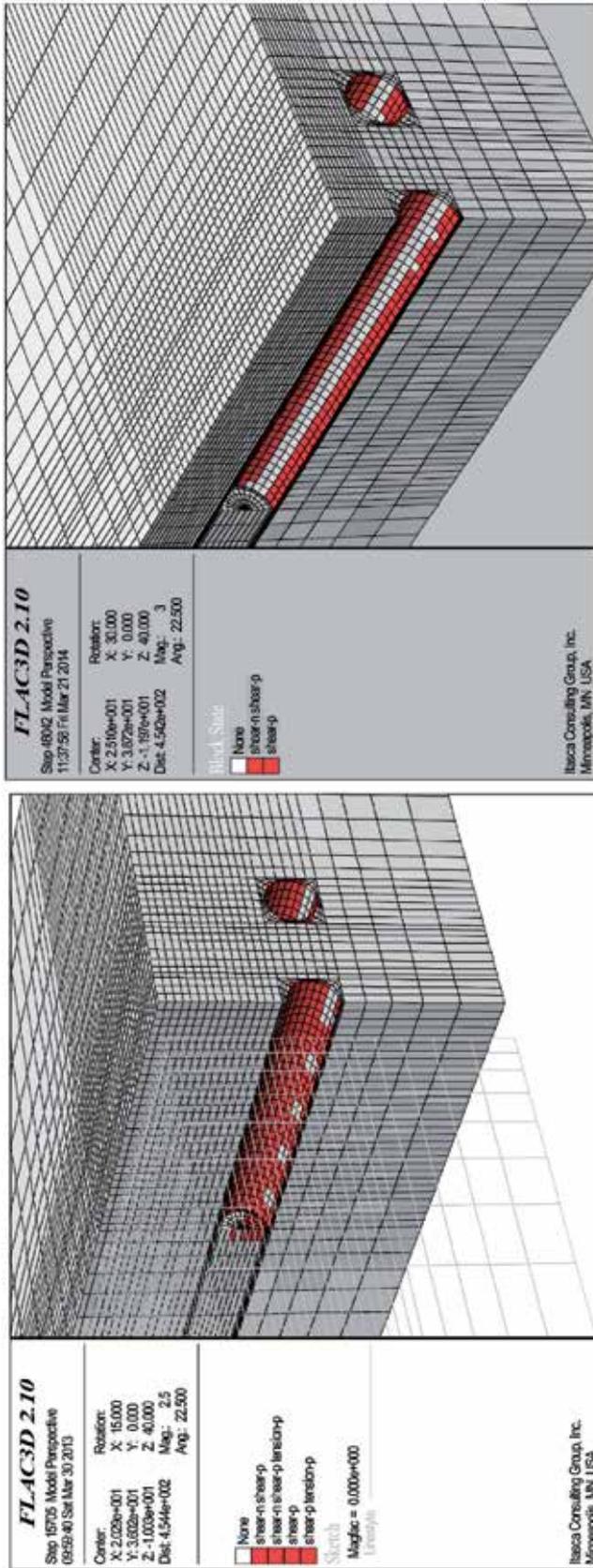
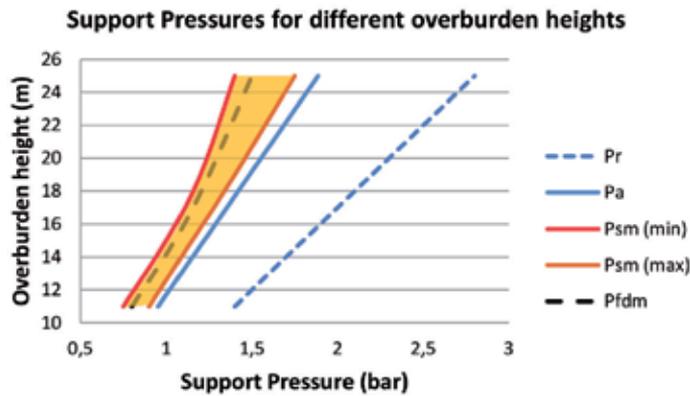


Figure 29. Calculated plastic zone width (in red) as per TID of CC23 Delhi Metro (left) and as per TID of Jaipur Metro Phase 1b (right) [11, 12].



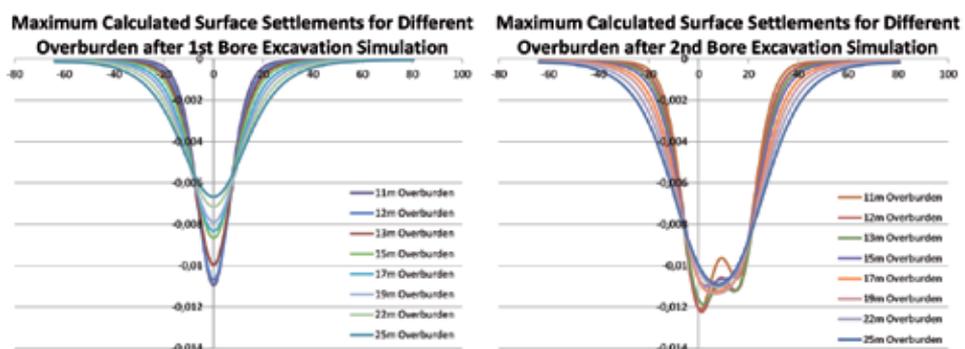
**Figure 30.** CC23 Delhi Metro. Calculated EPB support pressure for different overburden height as per critical sections F1–F8 of **Table 2** and including the results from the 3D FDM analysis ( $P_{fdm}$ ).

Therefore, it is of greatest importance to avoid unnecessary conservatism when calculating the required TBM support pressure. The knowledge of the geotechnical conditions of the groundmass and the groundwater through proper established geotechnical campaign, in conjunction with the knowledge of the TBM operation principles, are the key factors for determining the required support pressure.

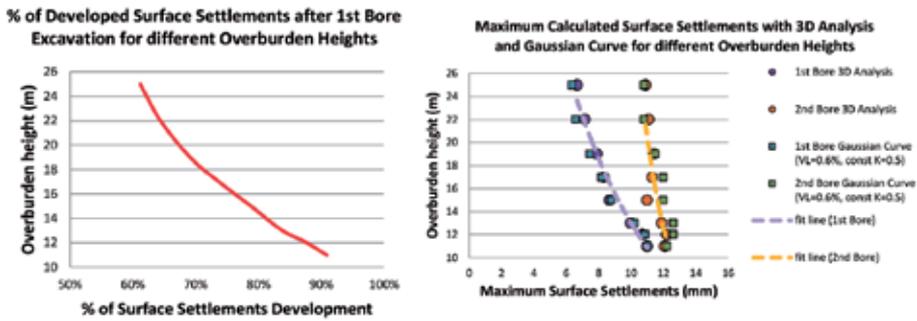
Following the establishment of the required support pressure in order to restrict the plastic zone formation width, the next step as per the TID is to evaluate the calculated induced surface settlements, in respect with the vulnerability of the existing surface and subsurface structures.

Following as example the CC23 Delhi Metro, the adopted support pressures in the 3D analyses resulted in minimum plastic zone widths in the order of 0.5–1.0 m and maximum surface settlements that are not exceeding 12 mm. After the total excavation of the two bores, the maximum calculated figures (independent of the overburden height) are bounded between 11 and 12 mm, as clearly illustrated in **Figure 31**.

It is evident that the range of maximum calculated surface settlements (7–11 mm) after the first bore excavation simulation is wider than the range calculated after both bore excavations. The overburden height is the key factor for this, and it is evident from the analyses results that the lower the overburden height, the greater the % of the developed surface settlements during the excavation of the first bore. As presented in the diagram of **Figure 32**, almost 90% of the total developed settlements are calculated during the first bore excavation for the shallowest case



**Figure 31.** CC23 Delhi Metro. Maximum calculated surface settlements after first (left) and second (right) bore excavation simulations for sections F1–F8 of **Table 2** (horizontal axis in m and vertical axis in mm).



**Figure 32.** CC23 Delhi Metro. Percentage of maximum developed surface settlements after first (left) bore excavation simulation for sections F1–F8 of **Table 2**. Diagrammatic comparison of maximum calculated surface settlements after the first and second bore excavations (right).

(11 m overburden), while for the deepest part of the alignment (25 m overburden), 60% of the total settlements are calculated (**Figure 32**). Of course the two bores’ axial distance is another parameter which affects the surface settlements development range, but for shallow tunnelling in Metro projects, optimum value for the dimensionless ratio of the tunnels’ axial distance to the diameter of the tunnel varies between 2.2 and 2.5. Even greater values than 2.5 can be applied, but it is economically preferable to keep the above range for the following reasons:

- Reduces the tunnel influence zone width, thus the surface monitoring is applied to less area.
- In case that the alignment does not permit reduction on the bores’ axial distance before entering the station, results in wider station island platform.
- In case that the tunnel alignment permits the axial distance reduction before entering the station, additional complication is added in TBM alignment survey and navigation.

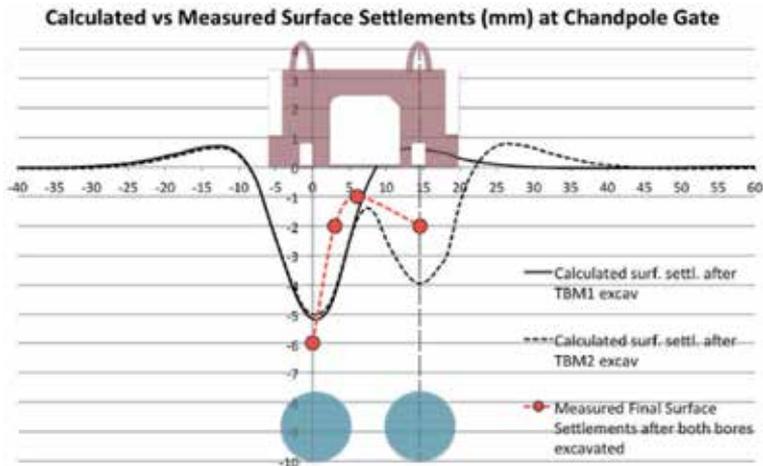
As it is evident from **Figure 32** (right), the projected maximum calculated surface settlements from first bore excavation, in respect to the different overburden heights, follow an exponential fit curve bounded by extreme values of 7 and 11 mm. With the proposed and simulated range of support pressure (0.8–1.5 bar), the total maximum calculated settlements after the excavation of the second bore follows a much steeper exponential fit curve bounded between 11 and 12 mm.

Further analysis of the curves of **Figure 32** (right), using the Gaussian formulae

$$S = S_{max} \exp\left(\frac{-y^2}{2i^2}\right) = \frac{V_L}{i\sqrt{2\pi}} \exp\left(\frac{-y^2}{2i^2}\right) \quad (1)$$

to calculate the surface settlements through the volume loss and the constant K, give results that are in good agreement with the 3D FDM, considering volume loss per bore equal to 0.6% for all the different overburden heights. Thus, proper calculation of the support pressure range controls the surface settlements and can lead to the same volume loss across the alignment independent of the overburden height.

The analysis of section CH (Chandpole Gate underpass) is another example to discuss the surface settlement development. Following the simulation of the support pressure as derived by Massinas and Sakellariou solution [6], the maximum 3D FDM calculated surface settlements are given in **Figure 33**.



**Figure 33.**

*Jaipur Metro Phase 1b. 3D FDM calculated surface settlements after the first and second bore excavation simulations versus actual final monitored surface settlements [8].*

After the first bore excavation simulation, the maximum surface settlement is calculated as 5 mm, while the second bore excavation only increases the surface settlement to 4 mm above the second bore without giving rise to the total maximum settlements. As already discussed in **Figure 32** (left), the shallow depth can give rise to the absolute maximum value of the surface settlements after the first bore excavation. The section CH, which is a case with extreme low overburden height, shows that even 100% of the final value of the surface settlements after the first bore excavation, while the second tunnel construction can extend only the shape of the settlement trough without increasing the absolute maximum value. This is also visible from the actual monitored results; thus after the first bore construction (**Figure 34**), the maximum measured settlements at the Gate area are 6 mm, while after the second TBM underpass (**Figure 33**), only extent of the trough above its axis is measured, with absolute maximum value of the settlements equal to 2 mm.

Indeed, the application of the design support pressure (**Figure 35**—left) during the construction is the key factor along with the adequate tail shield grouting, in order to control the soil's settlements; the secret is to keep the chamber pressure constant in each stroke of the TBM, thus in each excavation step as the shield is moving forward (**Figure 35**—right).

To understand how the earth pressure is properly applied in the excavation chamber, the respective plots during the calibration of the TBM 1 are given in **Figure 36**. As it can be seen from the left diagram of **Figure 36**, during the calibration stage, the pressure drops at the beginning of the excavation for the construction of ring 78. This means that the chamber pressure during the last stroke of the previous ring 77 is not kept constant and as per the design requirements.

Therefore, at the beginning of ring 78 excavation, the support pressure inside the chamber is zero, and only after the excavation advanced 250 mm the pressure starts to rise. After the calibration stage, the support pressure inside the excavation chamber (**Figure 35**, right; and **Figure 36**, right) is kept constant along the entire excavation length. Along the calibration area, the maximum measured surface settlements were in the order of 10–14 mm. During the excavation simulation of the first tunnel, the maximum calculated deflection of the Gate's foundation (above the tunnel) is 1/1300, while the maximum differential settlement between the two foundations of the Gate is calculated 5 mm with an angular distortion of 1/1200.

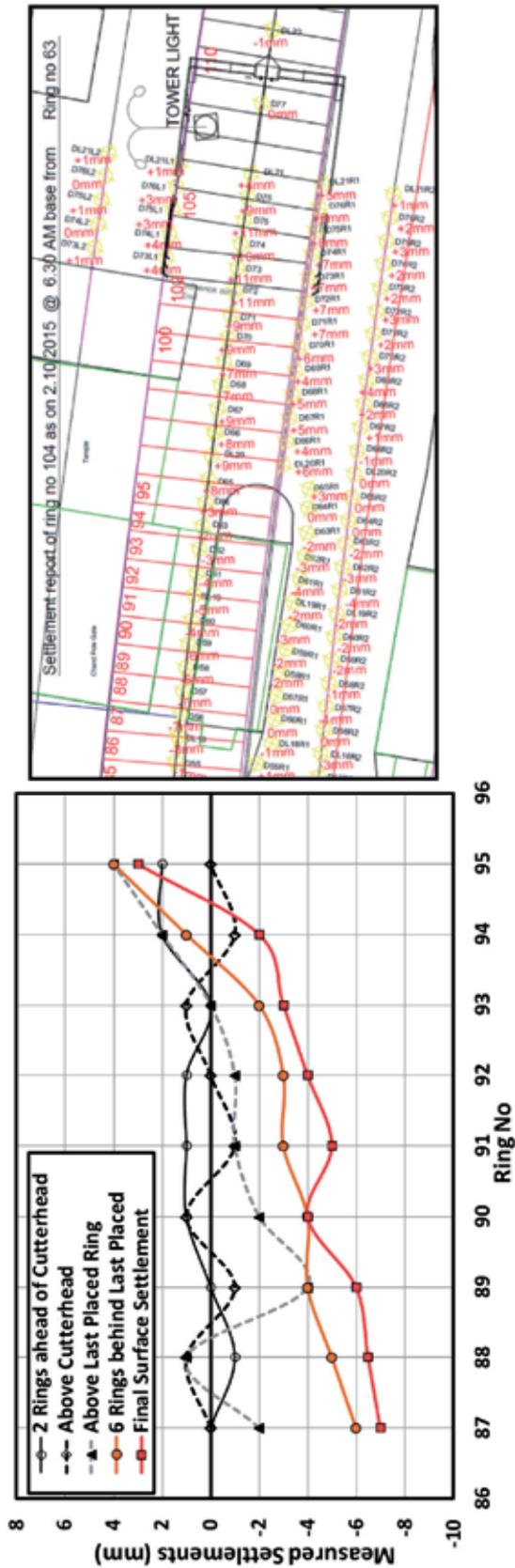
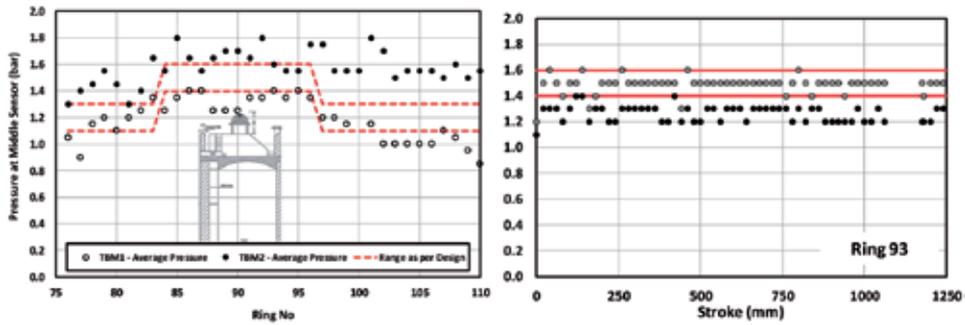
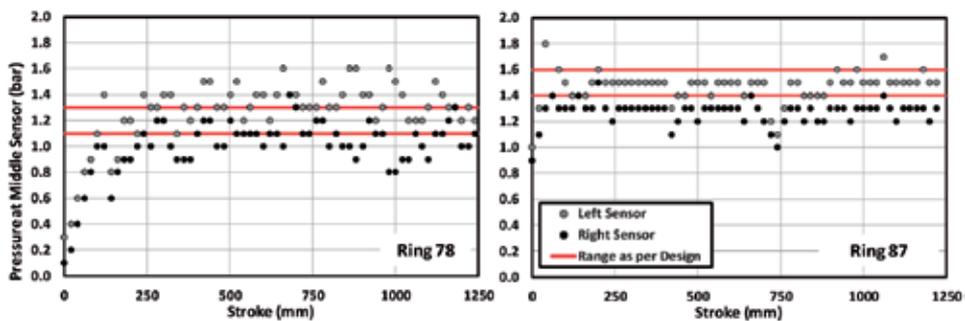


Figure 34. Variation of measured surface settlements below the Gate for various relative positions of the first TBM (1) versus the final surface settlement along the Gate (left) and snapshot of recorded settlement values for cutterhead at ring position 110 (approx. 20 m, i.e., 3D ahead of the Gate) [8].



**Figure 35.** Jaipur Metro Phase 1b. Variation of average face pressure per ring excavation versus the design range (as per section CH analysis), for TBM 1 (up line) and TBM 2 (down line) (left) [8].

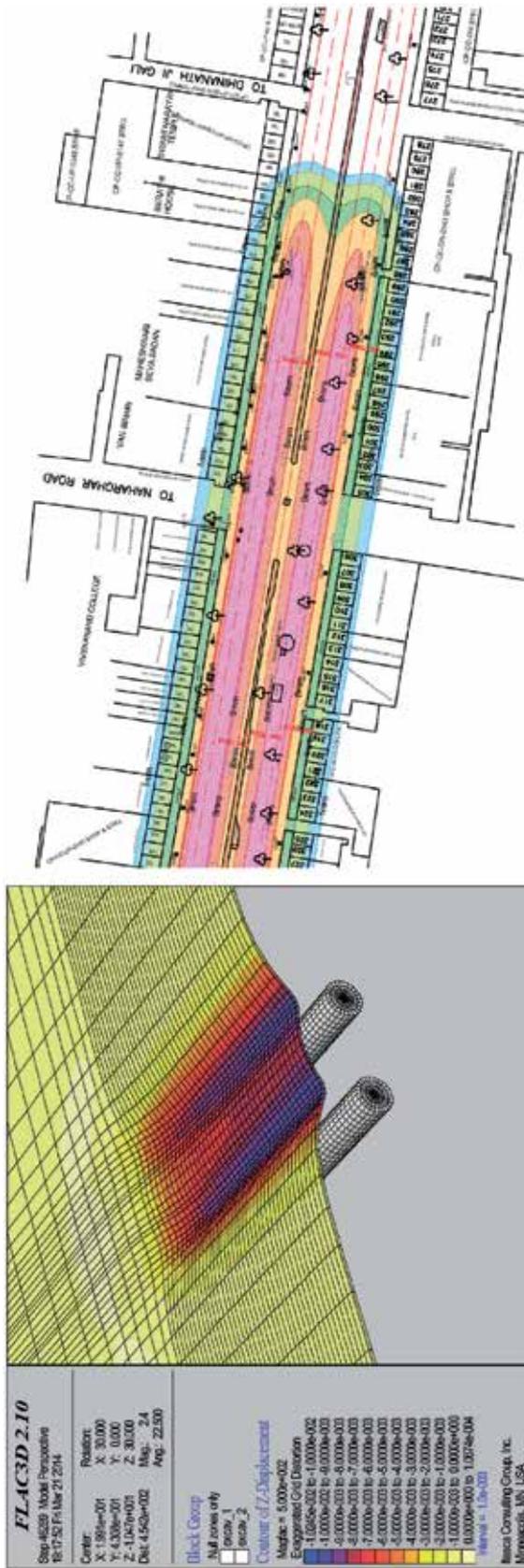


**Figure 36.** Jaipur Metro Phase 1b. Variation of face pressure of middle sensors along excavation of last ring of calibration drive (left) and first ring under the Gate (right) vs. the machine stroke [8].

Along the alignment of Jaipur Metro (3D FDM analysis J1–J3 as per **Table 7**), the maximum calculated surface settlements are in the order of 9–10 mm (**Figure 37**). As it is evident from **Figures 23** and **37**, all the buildings along the alignment are at 8 m distance from the tunnel bore axis. No structures are aligned above the tunnel centerline (except Chandpole Gate). Therefore, at the building location, the maximum calculated surface settlements are 4–5 mm. Maximum differential settlements are calculated 2 mm with an angular distortion of 1/2000.

For the case U1 of CC23 Delhi Metro, two analyses were performed. The first (as presented) includes the station structure, while the second is also performed without simulating the station. The purpose of the second analysis is to examine the effect of the station's stiffness on the development of the surface settlements. The presence of the stiffness of the existing tunnels and the Hauz Khas station affects the magnitude and the shape of the calculated surface settlements. In the first case, where the station is not simulated, the stiffness of the existing tunnels (Yellow Line) contributes to the development of the surface settlements by reducing their magnitude of approx. 10–15%.

This is evident in the diagram of **Figure 38** (left), where the maximum calculated surface settlements which are 30 m before the Yellow Metro Line is approx. 5 and 9 mm after the first and second bore excavations, respectively, while at a section just above and parallel with the up line axis of the Yellow Line, the surface settlements are calculated 4.5 and 8 mm, respectively. Therefore, a vertical shift on the settlement trough is observed, due to the stiffness of the existing tunnels, without affecting the overall shape of the trough but only the maximum value on



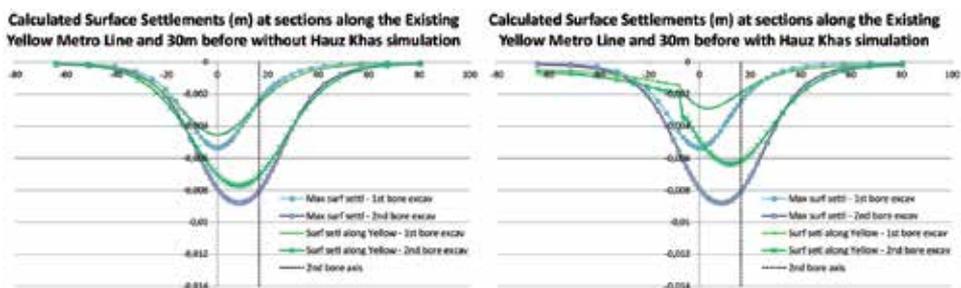
**Figure 37.** Jaipur Metro Phase 1b. 3D FDM calculated surface settlements after total excavation of both bores (values in meters) (left). Projected maximum calculated surface settlements along tunnel alignment (right).

the settlement curve. On the other examined case, the simulation of the station has a more clear influence on both the shape and the magnitude of the surface settlements. As it is evident from **Figure 38** (right), both vertical and horizontal shift of the settlements curve is observed. The vertical shift is affecting the magnitude of the maximum calculated settlements which is in the order of approx. 3 and 6 mm after the first and second bore excavations, respectively. Thus, a reduction of approx. 30–40% is observed from the maximum values (5 and 9 mm) of the surface settlements. After the first bore underpass simulation, vertical displacements are calculated at the crown of the existing tunnels, with maximum values within the range of 3 and 5 mm (for both examined cases). After the second bore excavation, a slight increase on the crown vertical displacements is calculated, and the final maximum values are in the order of 7 and 8.5 mm (for the cases with and without station simulation, respectively). Again, the stiffness of the simulated station reduces the actual values of the crown displacements by producing vertical and horizontal eccentricity in the calculated curves, almost in the same degree as in the surface settlements which are presented before.

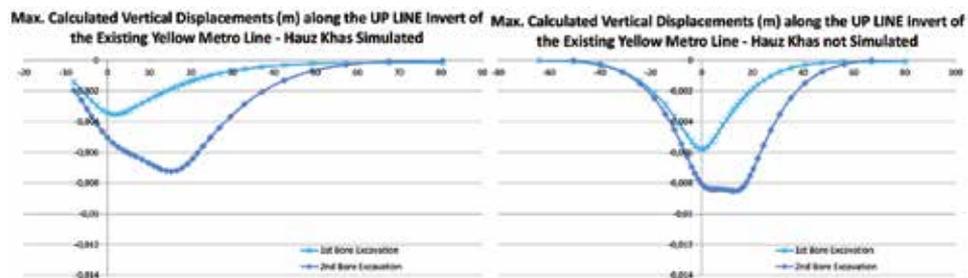
The maximum vertical displacements at the invert of the existing tunnels are calculated within the range of 3.5 mm (case with no station) and 6 mm (case with station) after the first bore excavation, with a maximum angular distortion in the order of 0.25‰ (no station simulation) having a peak value of 0.3‰ (station simulated) at the area which is in close vicinity with the station.

The advance of the second bore below the existing tunnels increases the invert displacements to 7 mm for the case where the station has been simulated and to 8.5 mm at the second case without the station been simulated, as it is evident in **Figure 39**. An increase on the differential displacements is also calculated with a maximum slope value reaching 0.4‰ (**Figure 40**). To sum up, for the case of U1 section, the maximum vertical displacements at the crown and the invert of the existing tunnels are calculated within the range of 3.5–6 mm with a maximum angular distortion of 0.3–0.35‰, after the first bore excavation simulation. The completion of the second bore excavation simulation gives rise to maximum vertical displacements at crown and invert equal to 7 and 8.5 mm, respectively, with a maximum angular distortion reaching the value of 0.4‰. The maximum horizontal displacements are calculated lower than 1 mm. The most critical aspect in U1 underpass is the execution of works with the existing tunnels under operation. Therefore minimum required displacements and differential settlements are acceptable at the track slab of the under operation tunnels. Details will be presented in the following chapter.

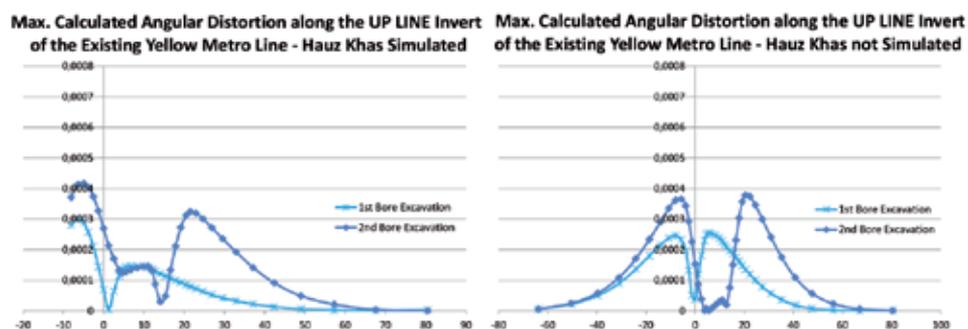
The case U2 of CC23 Delhi Metro was another critical underpass since in close vicinity with the tunnel launching shaft was the existing bridge and the water canal. The plastic zone width as it is evident in **Figure 41** is calculated less than 0.5 m width.



**Figure 38.** Calculated surface settlements (m) at sections along the existing yellow line and 30 m before—case without station simulation in the left diagram and case with station simulation in the right diagram.



**Figure 39.** Calculated max vertical displacements (m) at the invert of the existing up line of yellow line for both examined cases—with (left above) and without (right above) station simulation.



**Figure 40.** Calculated max differential displacements at the invert of the existing up line of yellow line for both examined cases.

The maximum calculated surface settlements are in the order of 8–9 mm with a maximum deflection of 1/1000. At the area of the existing bridge, the surface settlements and angular distortion are calculated 2–8 mm and 1/1000, respectively. At the foundation level of the bridge (~4 m above tunnels crown), the maximum calculated vertical subsurface displacements are 10 mm with maximum angular distortion of 1/700 (**Figure 42**).

### 3.3.3 Part C: building risk assessment

The permissible settlements and deflections in structures are commonly assigned according to their vulnerability which is the output from the precondition survey.

The relation between the categories of damage and the vulnerability index is given by Chiriotti et al. [16] in **Table 9**. Considering the results from part B of TID, the conclusive **Table 10** is prepared in order to be used in correlation with **Table 9**.

Following **Tables 9** and **10**, for all sections F1–F8 of CC23 Delhi Metro, the buildings within the tunnel influence area are expected with “Negligible” damage. Furthermore, for Jaipur Metro Phase 1b, all the buildings are not aligned above the tunnel centerline (except Chandpole Gate). For those cases, the derived results show also “Negligible” damage category even if the existing old buildings of “Pink City” will be categorized as “Highly” vulnerable. Considering that Chandpole Gate vulnerability is in category “Slight,” according to the precondition survey, “Negligible” damage is expected for settlements <6.7 mm and angular distortion <1/750. As already derived from the 3D analysis, the maximum calculated settlements and angular distortion are 5 mm and 1/1200, respectively, values which are

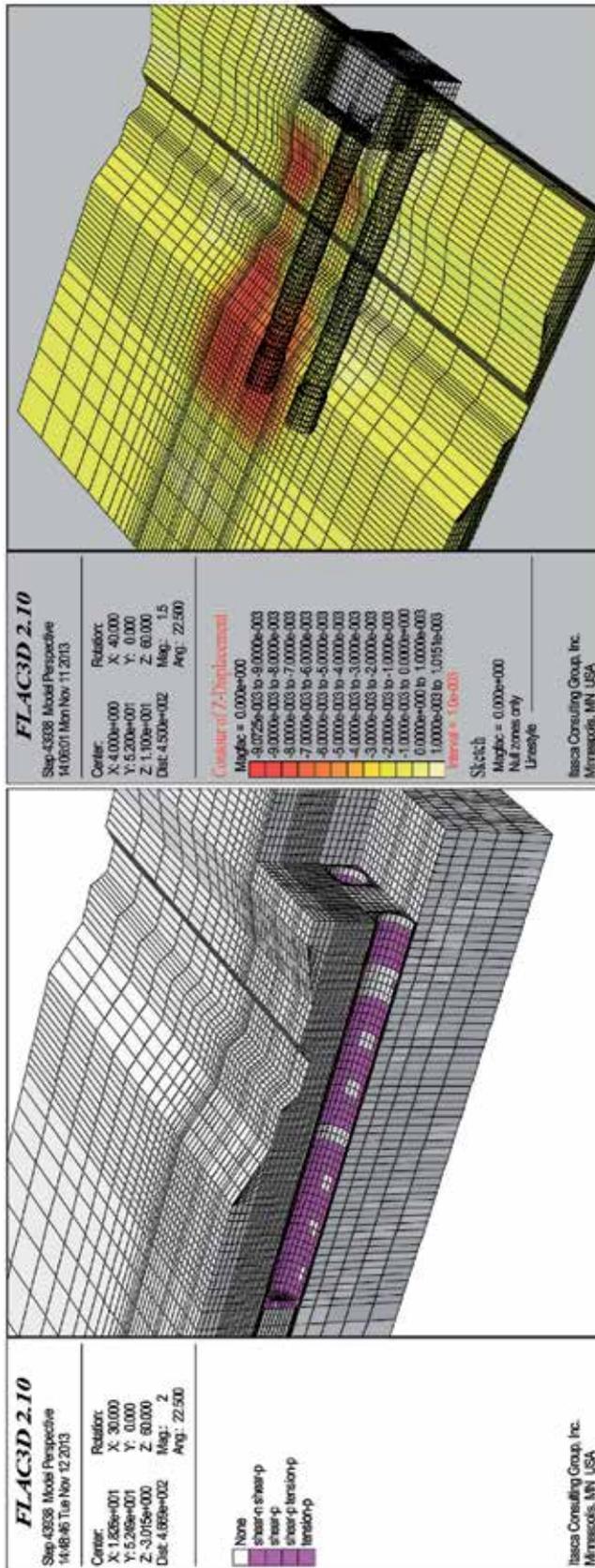


Figure 41. Calculated plastic zone width (left) and surface settlements (right) for case U2 of CC23 Delhi Metro [10].



Damage category	Negligible		Low		Slight		Medium		High					
	$0 < I_v < 20$	$20 < I_v < 40$	$40 < I_v < 60$	$60 < I_v < 80$	$80 < I_v < 100$	Fr = 1.0		Fr = 1.50		Fr = 1.75		Fr = 2.0		
	Smax (mm)	$\beta_{max}$	Smax (mm)	$\beta_{max}$	Smax (mm)	$\beta_{max}$	Smax (mm)	$\beta_{max}$	Smax (mm)	$\beta_{max}$	Smax (mm)	$\beta_{max}$	Smax (mm)	$\beta_{max}$
Negligible	<10	<1/500	<8	<1/625	<6.7	<1/750	<5.7	<1/875	<5	<1/1000				
Slight	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				
Moderate	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				
Severe	>75	>1/50	>60	>1/63	>50	>1/75	>43	>1/84	>37.5	>1/100				

**Table 9.**  
 Relation between buildings' categories of damage and vulnerability index.

related with “Negligible” damage even in the case of “Highly” vulnerable structures. For the case of the bridge (U2), “Negligible” damage category is expected as per the calculated results presented synoptically in **Table 10**.

Following the damage assessment of the buildings within the influence zone of the tunnels, the trigger and alarm levels are mandatory to be established in order to monitor the actual the surface settlements and compare them with the calculated ones. Examples from real established trigger and alarm levels are given in **Table 11** for the critical sections U1, U2, and CH.

Concluding, the derived support pressures as per part A of TID resulted in acceptable surface settlements (calculated in part B), and consequently the building risk assessment (part C) proved that no additional measures are required to be

Section	Overburden (m)	Support pressure (bar)	Smax (mm)	$\beta_{max}$
F1	25	1.5	11	1/2500
F2	22	1.4	11	1/2500
F3	19	1.2	11	1/1700
F4	17	1.1	11	1/1650
F5	15	1.0	11	1/1450
F6	13	0.8	12	1/1250
F7	12	0.8	12	1/1050
F8	11	0.8	12	1/1000
U1	Existing tunnel invert at 4.5 m	1.8	8 (at existing tunnel invert)	1/2500 (at existing tunnel invert)
U2	Bridge at 4 m	1.5	10	1/700
J1	7	1.1	9 (3 at building area)	1/2000 (at building area)
J2	10	1.4	10 (4 at building area)	1/1500 (at building area)
J3	12	1.5	10 (5 at building area)	1/1500 (at building area)
CH	Chandpole Gate/4.5	1.5	5	1/1200

**Table 10.**  
Synoptic presentation of calculated surface settlements.

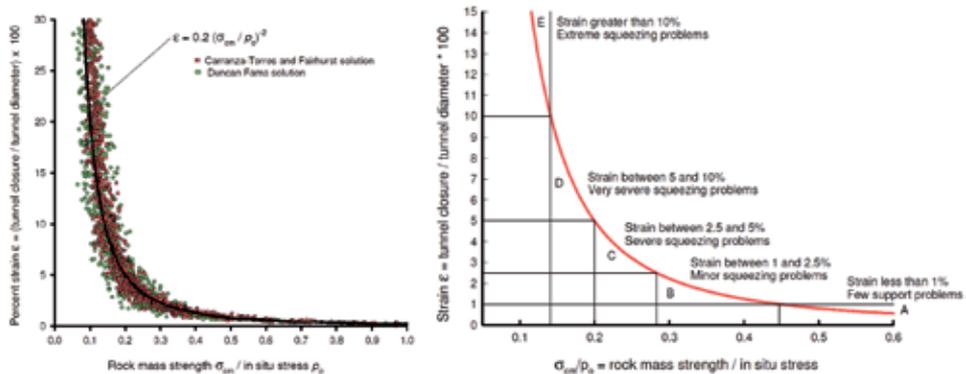
Section	Measurement	Trigger level	Alarm level	Limit values
U1	Deflection of track slab of existing tunnels	$\pm 3$ mm with gradient 0.20‰	$\pm 4$ mm with gradient 0.25‰	$\pm 5$ mm with gradient 0.3‰
	Displacement of existing tunnels' segmental lining	$\pm 4$ mm	$\pm 6$ mm	$\pm 8$ mm
U2	Settlements at bridge area	7 mm	9 mm	10 mm
	Angular distortion	1/1000	1/777	1/700
CH	Settlements	4 mm	5 mm	6 mm
	Angular distortion	1/1400	1/1200	1/1000

**Table 11.**  
Established trigger and alarm levels for sections U1, U2, and CH.

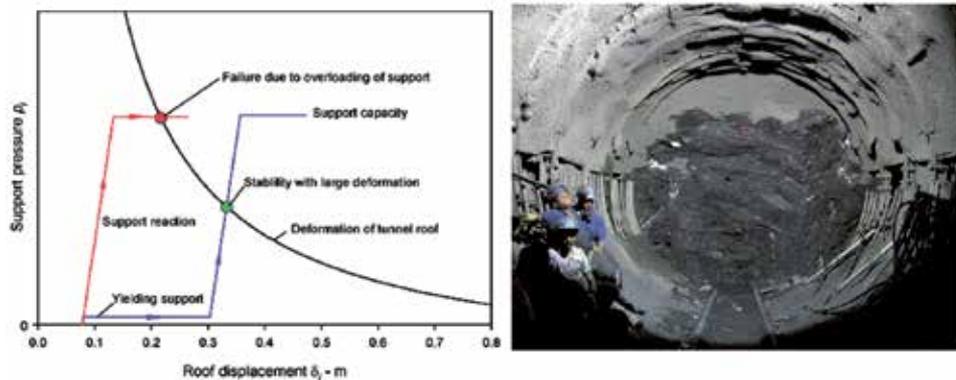
taken, in order to protect the existing surface and subsurface structures which are within the influence zone of the Metro tunnels.

#### 4. Deep tunnelling in squeezing conditions

As presented in the previous paragraphs, shallow tunnel design is mainly related with the control of the induced surface settlement, in order to protect the existing surface and subsurface structures (monuments, important buildings, tunnels, bridges, etc.), and this can be achieved by minimizing the plastic zone width around the cavity. On the other hand, for the case of deep tunnels, excavated in difficult geological-geotechnical conditions, the aim is to design the tunnel excavation and primary support by allowing the ground mass to get plasticized and redistribute the stresses around the cavity but at the same time to control the extent of the plastic zone in order to achieve equilibrium in each excavation stage. The excavation of the tunnel is a dynamic phenomenon, since the development of the ground displacement is taking place ahead of the excavation face (~ half to one tunnel diameter) and reaches its maximum value about one- and one-half tunnel diameters behind the excavation face. Whether or not the above deformations induce stability problems in the tunnel depends upon the ratio of ground mass strength ( $\sigma_{cm}$ ) to the in situ stress ( $P_o$ ) level (**Figure 43**).



**Figure 43.** Approximate relationship between strain and the degree of difficulty associated with tunnelling through squeezing rock, for unsupported tunnels [17].



**Figure 44.** Comparison between rigid (red line) and yielding (blue line) support [18].

As it is evident from the curve of **Figure 43**, the smaller the ratio  $\sigma_{cm}/P_o$ , the higher the strain, and thus critical in the tunnel design is the control of the rock mass deformations around the cavity. Therefore, in case of deep tunnels with high overburden and low rock mass properties, controlled plasticization is almost mandatory in order to reduce the exerted loads around the excavation. By intervening with application of proper yielding support measures, the rock mass is deformed plastically under controllable manner. This solution involves the introduction of deformable elements into the lining. These elements are allowed to deform by a predetermined amount, and when this limit is reached, the support system becomes rigid and starts to carry the full support load. This process allows progressive failure to occur, and a plastic zone is formed in the rock mass immediately surrounding the tunnel. This progressive failure results in a redistribution of the stresses in the rock, surrounding the tunnel, and in a significant reduction in the capacity of the support system required to stabilize the tunnel.

The concept is illustrated in **Figure 44**. Different behavior patterns of rigid (red line) and yielding (blue line) supports are compared. As the rigid support system is installed at a roof displacement of 100 mm, the rock mass loads are that high that the system fails. On the other hand, when a flexible yield support is installed at the same initial crown displacement (100 mm), the controllable deformation of the rock (to 300 mm) through the yielding joints reduces the loads transferred to the support after “locking” of the sliding joints (**Figure 44**, right), without failure of the system.

In the following paragraphs the case study from one of the most difficult tunnels around the world will be analyzed, and details from the design of critical sections will be presented.

#### **4.1 Yacambu-Quibor tunnel overview**

A characteristic example of a deep tunnel case, where excessive loads and deformations are recorded around the excavation, is the Yacambu-Quibor tunnel in Venezuela. The 26.4-km-long tunnel (**Figure 45**) was originally conceived as an unlined TBM-driven tunnel to convey water from the Yacambu dam in the wet Orinoco River basin to the dry agricultural region around Quibor. Construction commenced in 1976 in anticipation that the majority of the rock mass through which the tunnel would be driven would be silicified phyllite. This rock forms very steep and stable cliffs in the area of the dam site, and numerous unsupported exploration and drainage tunnels have remained stable for many years in the dam abutments. Unfortunately, it soon became apparent that the rock mass through which the tunnel had to be driven is composed largely of graphitic phyllite which, in contrast to the silicified phyllite of the dam site, is a very weak and tectonically disturbed material. Unsupported tunnels cannot be driven in this rock, particularly at the cover depths of up to 1200 m that occur in the center of the tunnel. In addition, the Bocono fault and the Turbio fault had to be crossed by the tunnel, and it was anticipated that these would cause significant stability problems. Floor heave in the original TBM-driven tunnel resulted in abandoning the tunnel in 1979, and all subsequent tunnel driving decided to proceed with conventional drill and blast and mechanical excavation. Steel sets embedded in a shotcrete lining provided the main support system with the use of forepoling, spiling, and rockbolting where required. Spalling of the shotcrete lining and floor heave has continued as problems whenever the rock mass deformation has been more severe than anticipated.

During the driving of the Ventana Inclinada (**Figure 45**), an intermediate access tunnel designed to provide early access to the Bocono Fault, severe squeezing was encountered. This is a phenomenon characterized by closure of the tunnel, and it

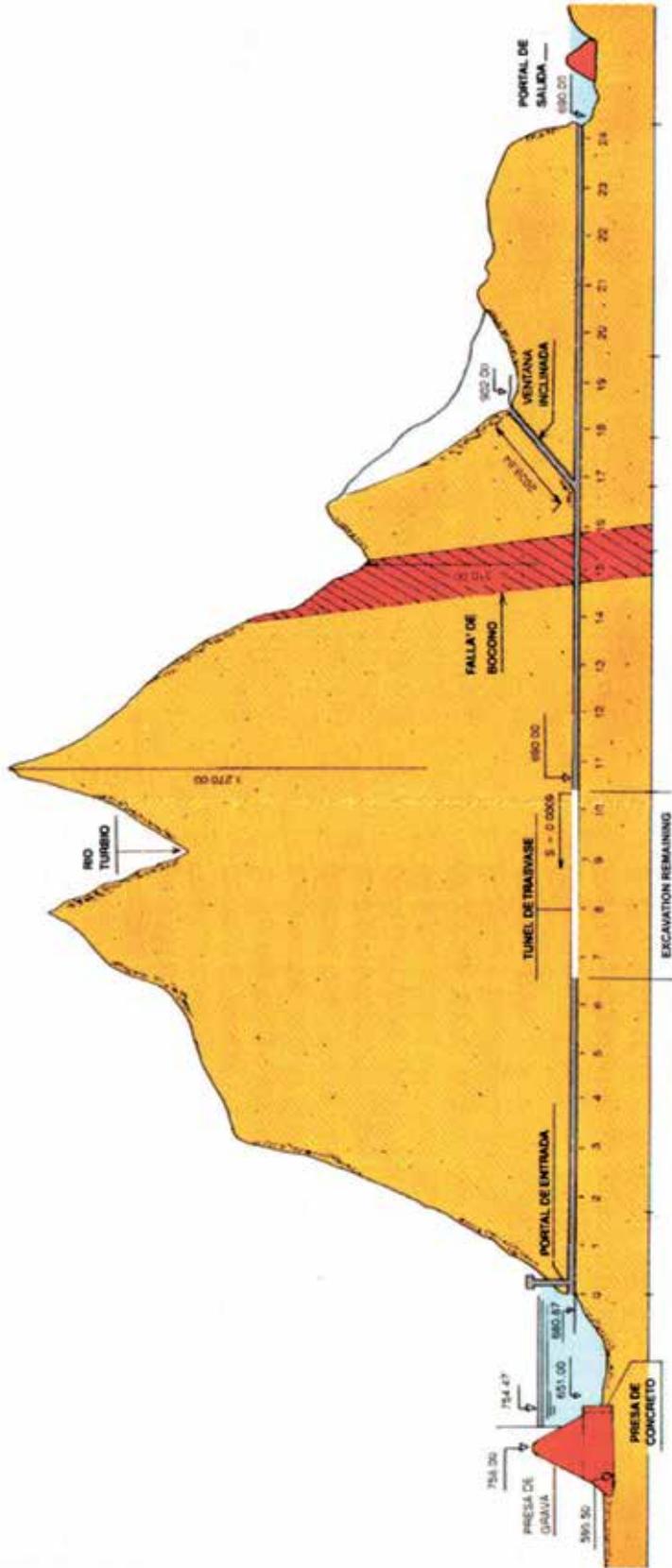


Figure 45.  
Yacambu-Quibor tunnel longitudinal section [18].

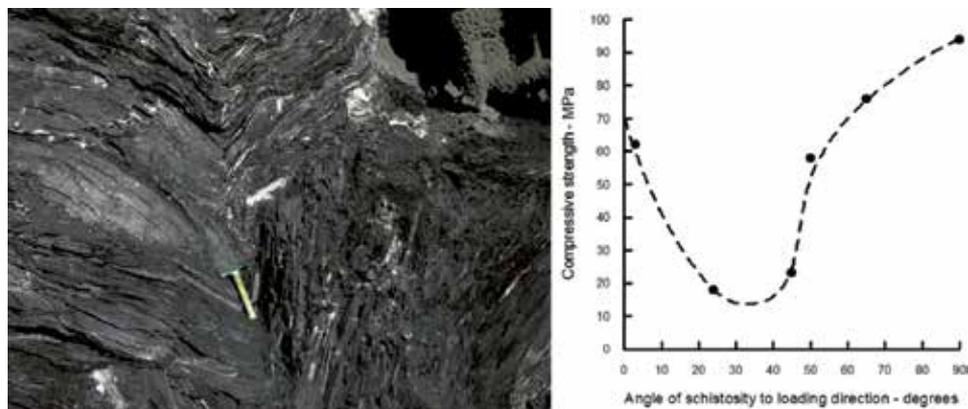
occurs when the rock mass surrounding the tunnel is overstressed and a “plastic zone” develops, as described previously. Unless adequate support measures are introduced, this squeezing can develop into uncontrolled closure and eventually collapse of the tunnel. The primary cause of squeezing is, as already described, a combination of a weak rock mass subjected to high in situ stresses due to a high overburden cover. The process is exacerbated by the presence of water and by gradual deterioration or “creep” of the rock mass.

## 4.2 Graphitic phyllite

In the case of the graphitic phyllite, shown at the Salida heading in **Figure 46**, the tectonically disturbed rock mass makes it very difficult to collect samples for laboratory testing, and, consequently, very little reliable rock mass strength data is available.

The choice of an appropriate intact strength from results such as those presented in **Figure 46** is a highly subjective process. Anisotropic and foliated rocks such as slates, schists, and phyllites, whose behavior is dominated by closely spaced planes of weakness, cleavage, or schistosity, present particular difficulties in the determination of the uniaxial compressive strengths. Salcedo has reported the results of a set of directional uniaxial compressive tests on a graphitic phyllite from Venezuela. It will be noted that the uniaxial compressive strength of this material varies by a factor of about 5, depending upon the direction of loading. Evidence of the behavior of this graphitic phyllite in the field suggests that the rock mass properties are dependent upon the strength parallel to schistosity rather than that normal to it. In the case of Yacambu-Quibor, the many years of experience of the behavior of the tunnel gives adequate information to calibrate the rock mass strength to a certain degree. It is recommended that the parameter that should be varied in this calibration is the intact rock strength  $\sigma_{ci}$  since this is both logical from a mechanics point of view and it has a very significant numerical influence on the estimated rock mass strength. For the good siliceous phyllites, the intact rock strength  $\sigma_{ci}$  was determined as 40–50 MPa, while for the more carbonaceous and foliated phyllites (graphitic phyllite), the intact rock strength varies from 15 to 30 MPa.

Approximately 5 km of tunnel remained to be excavated, and it is anticipated that a significant portion of this length will be in graphitic phyllite and that this will be under the high overburden cover (up to 1200 m). The Turbio fault, which is clearly defined on surface, may extend to tunnel depth, and provision had to be made for



**Figure 46.** Tectonically deformed graphitic phyllite exposed in excavated face of Salida heading of Yacambu-Quibor tunnel on 24 November 2003 (left) and influence of loading direction on strength of graphitic phyllite (right) [18].

excavating through and stabilizing the tunnel in this fault. It is also possible that high pressure water and methane gas could be encountered in excavating through the Turbio fault. Therefore, for designing the remaining un-excavated part (approx. 5 km), the information from two critical stations at KP 6+540 and KP 11+700 along the excavated tunnel (**Figure 47**) were used in order to design the most adequate support measures and excavation profile as well as the final lining of the tunnel.

### 4.3 Support types

The support types along the tunnel were divided into two main categories. Three full face horseshoe types S1, S2 and S3 for their application in silicified phyllite with GSI values greater than 50 (Rock mass types A, B and C) were foreseen, for fault zones and areas with large intrusions of graphitic phyllite at the excavation face (Rock mass types D1 and D2) with GSI values even low to 25 (**Figure 48**).

The flexible support types (S4 and S5) consist of shotcrete with total final thickness up to 60 cm (to act also as final lining), steel sets with sliding joints embedded in the shotcrete lining placed in axial distances of 0.6–1.0 m, and 10–12 fully grouted rock bolts installed every round length. Two sliding joints were foreseen for class S4 at the vaulted area (**Figure 49**), while three flexible joints were equally spaced in category S5. In both flexible support classes, the sliding joints gap was 30 cm as shown in the detail of **Figure 49**.

### 4.4 Tunnel station at KP 11+700

In order to investigate the behavior of the tunnel and the reaction of the flexible support class, to be used in the Turbio fault area, the particular case of the support near station 11+700 in the Salida heading was considered, and detailed 3D FEM analysis is elaborated. Near the station the conditions were as follows:

Rock mass classification: GSI 35

Tunnel cover: 1100 m

Horizontal-vertical stress ratio ( $K_o$ ): 1.15

Assumed radial deformation before final closing: 300–350 mm

An intact rock strength of 20 MPa is considered, and the following rock mass properties are considered in the simulation:

- Unit weight,  $\gamma$  (kN/m<sup>3</sup>): 25
- Cohesion,  $c$  (MPa): 1.28
- Friction angle,  $\varphi$  (°): 19.72
- Young modulus,  $E$  (MPa): 1905
- Poisson ratio,  $\nu$ : 0.30
- Rock mass strength,  $\sigma_{cm}$ : 2.4 MPa

It is well noticed that the ratio  $\sigma_{cm}/P_o$  ( $<0.1$ ) for the above case reveals extreme squeezing conditions. Following the above, support type S4 has been examined. For the simulation, SOFiSTiK [19] software was used. Large model is constructed

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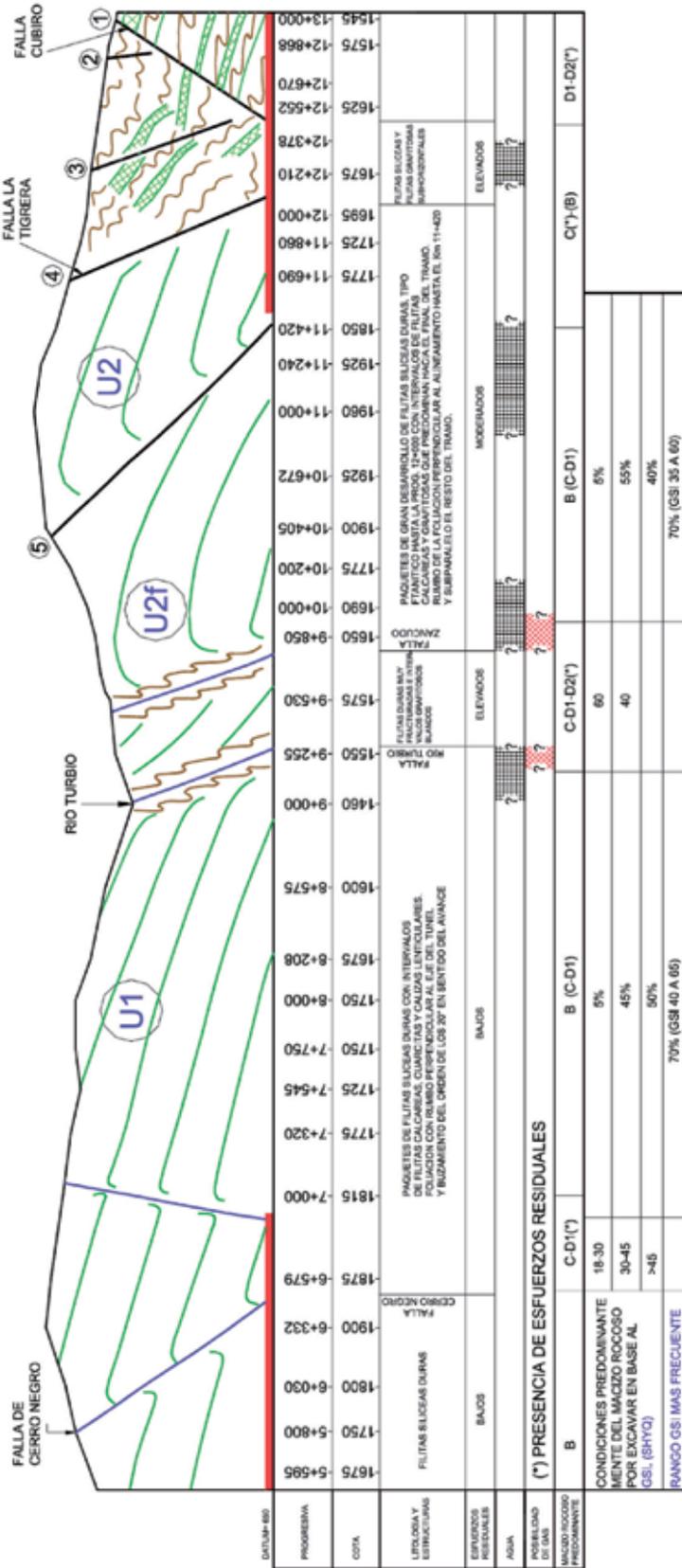


Figure 47. Geological-geotechnical section of Yacambu-Quibor tunnel [18].

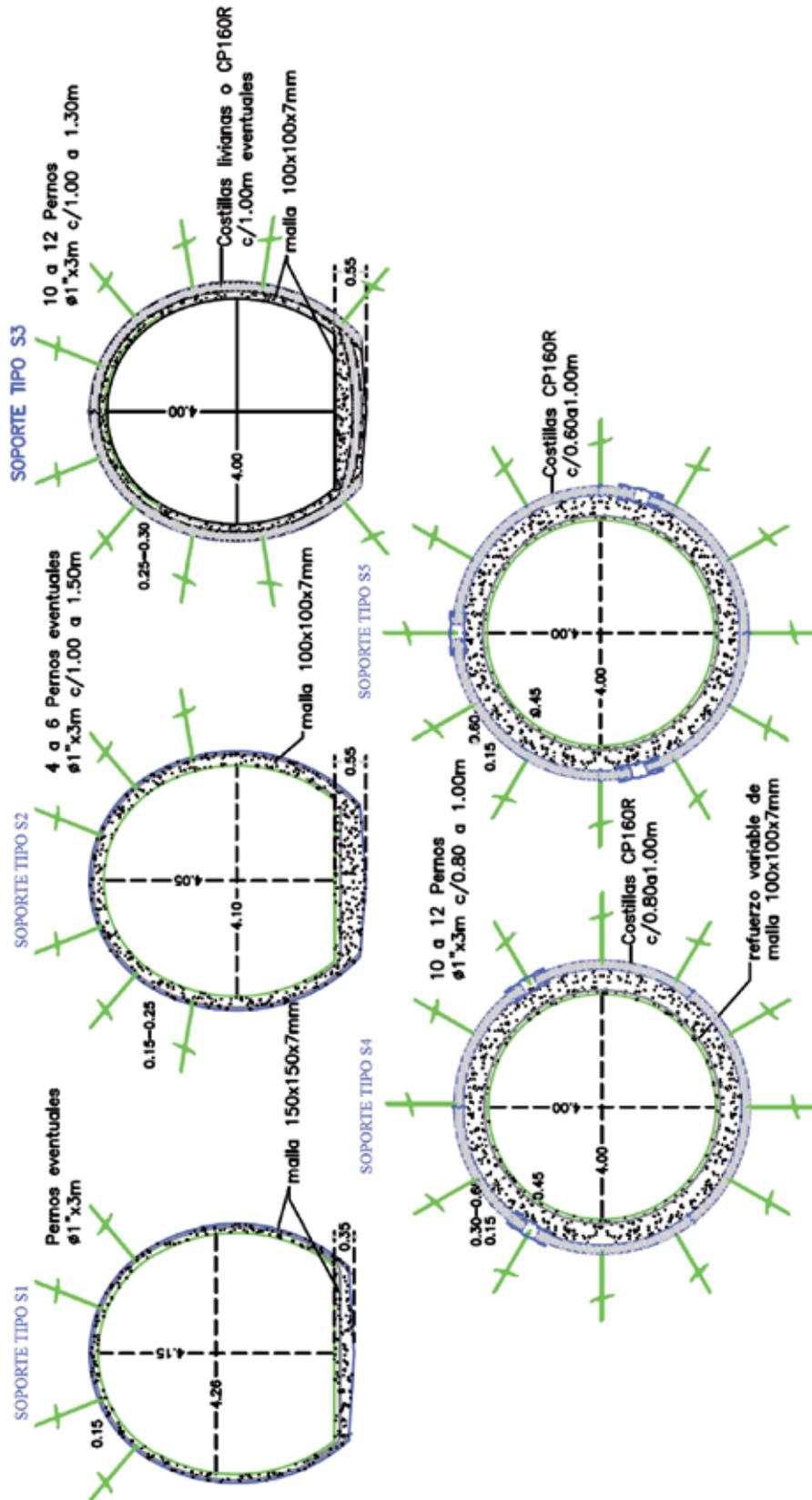


Figure 48.  
 Support types of Yacambu-Quibor tunnel [18].

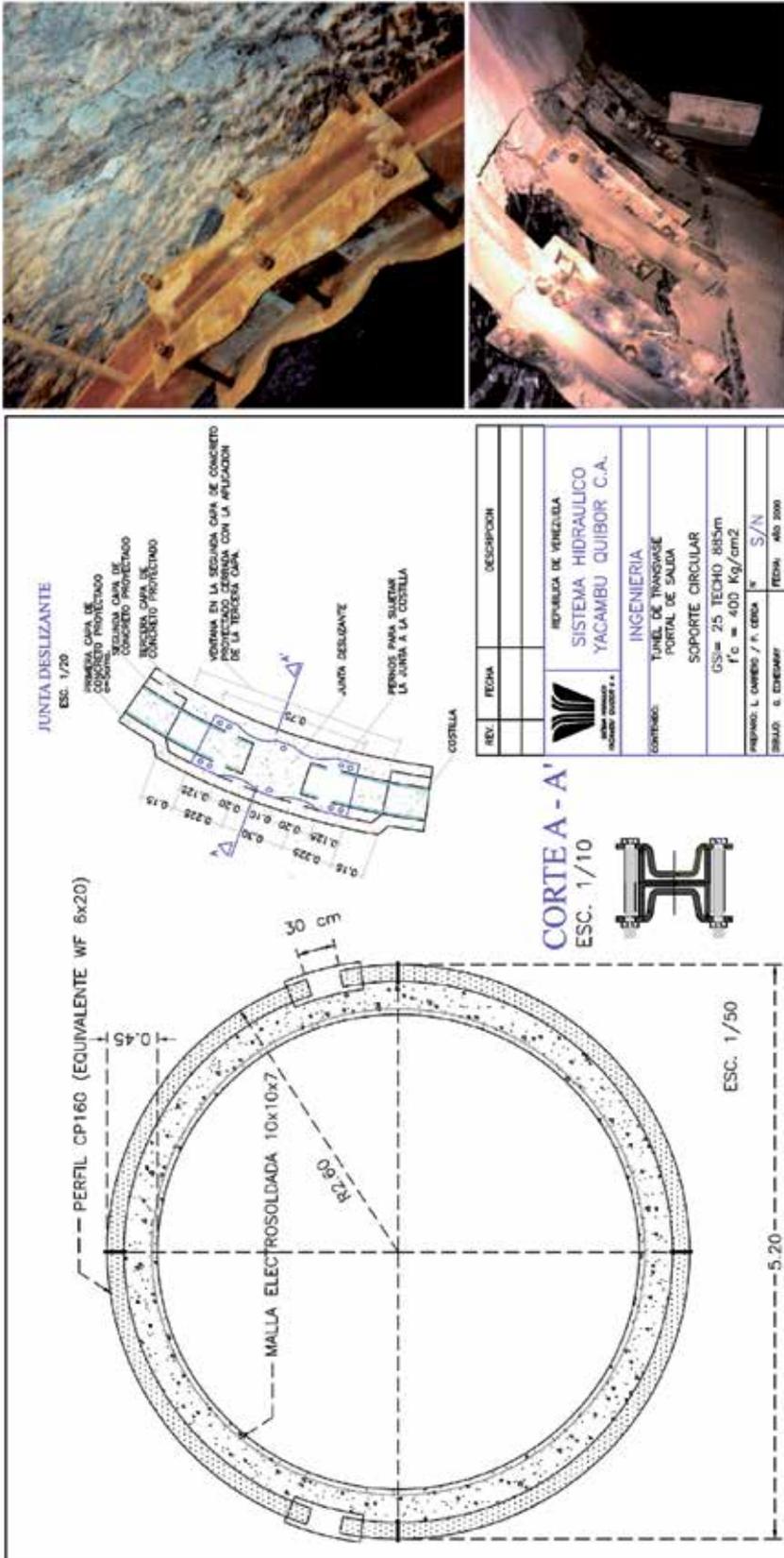
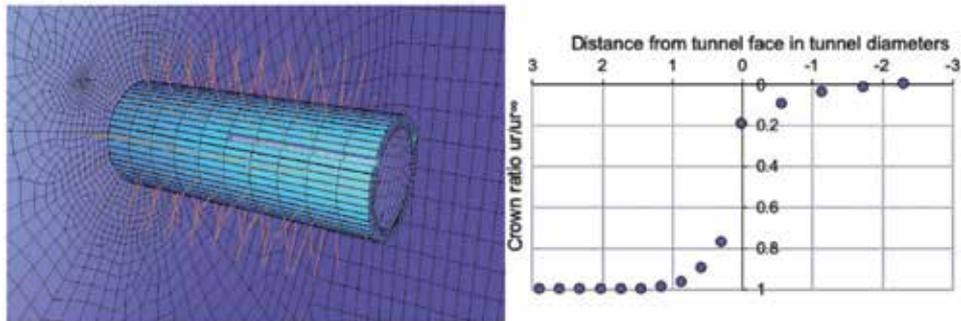


Figure 49. Support type S<sub>4</sub> with sliding joint detail and photos [48].



**Figure 50.**

3D finite element model of Yacambu-Quibor tunnel temporary support with sliding joints and roof displacement (ratio) versus tunnel advance [18].

consisted of octahedral brick elements. An excavation diameter of 5.2 m is simulated with application of shotcrete shell, simulated with quadrilateral 4-noded elements. For the simulation of sliding joints, spring elements were used with closure gap of 300 mm. Excavation length is considered 1.5 m. Typical view of the 3D model is presented in **Figure 50** (left).

The 3D analysis results (**Figure 50** right) showed that the crown vertical displacement of the rock begins approximately one diameter ahead of the tunnel face and reaches its maximum value about one- and one-half tunnel diameters behind the excavation face.

The total crown vertical displacement is calculated in the order of 200 mm. It is noted that the 200 mm gap closure gives a crown vertical displacement equal to ~60 mm for a tunnel radius equal to 2.6 m, while for the same radius but for a total closure of 300 mm, the crown displacement is calculated 100 mm.

Since the 3D modelling requires enormous computational time, due to large strain conditions, 2D FEM models were also constructed, and further parametric analysis was executed in order to confirm the findings from the previous 3D analysis. Using the convergence-confinement method (Panet 1995), for an unsupported deep tunnel, to assess the rock mass displacement ahead of the excavation face, two cases were investigated. In the first FEM model (S4-1), detailed simulation technique of the sliding joint closure is considered using spring elements. In the alternative FEM model (S4-4), a more simplified approach was used. The principle of the flexible support is that during the movement of the sliding joints, the stresses are released from the shotcrete shell, and only after the total closure of the joints, the temporary shell starts to carry the rock mass loads. Considering this principle, the support system (shotcrete shell) is activated when the crown displacement reaches the value which includes also the locking of the joints, thus rock mass displacement ahead of the face plus additional vertical movement due to joint gap closure. In **Figure 51**, the simulation steps for the two models are presented.

Considering the convergence-confinement method, a crown displacement equal to 150–200 mm is calculated 2–3 m ahead of the excavation face. In model S4-1 the resulted relaxation as per the convergence-confinement method is considered in LC1, and the respective relaxation value is applied on the face core. As per the analysis results, this led to a crown displacement of 211 mm, thus is matching the convergence-confinement calculations. Following the simulation of the sliding joints, at the end of LC2, the spring displacement (gap closure) is calculated 320 mm and the crown displacement equal to 305 mm. After the final excavation of the tunnel, simulated in LC3, the total crown displacement is calculated 315 mm. For the case of model S4-4, a total relaxation is considered in order to simulate the rock mass relaxation ahead of the tunnel face plus the

Load Case	Stage description	Core modulus reduction %	Figure
LC10	Initial stress field	0%	
LC1	Relaxation of heading due to proximity of the excavation face (Section analysis: 2-3 m in front of the face).	98%	
LC2	Installation of 10-15cm shotcrete, steel sets, and further relaxation of stresses around the excavation (Section analysis: 2-3 m behind the face)	99,9%	
LC21	Installation of rock bolts. Shotcrete thickness of 15 cm has reached its final strength. Heading excavation has advanced up to 15 m.	99,9%	
LC3	Full simulation of tunnel excavation. Mobilization 100% of the support measures (60cm of shotcrete).	Excavated	
LC10	Initial stress field	0%	
LC1	Relaxation of heading in order to simulate convergence of 30cm on the tunnel's crown (due to the yielding joints of HEB)	99%	
LC2	Installation of rock bolts. Shotcrete thickness of 30 cm has reached its final strength. Heading excavation has advanced up to 15 m.	99,9%	
LC3	Full simulation of tunnel excavation. Mobilization 100% of the support measures (60cm of shotcrete).	Excavated	

**Figure 51.** Simulation stages for S4-1 model (left) and S4-4 model (right) [20].

Location	Crown vertical displacement (mm) for S4-1/S4-4	Sliding joint closure (mm) for S4-1/S4-4	Shotcrete shell axial force (kN) for S4-1/S4-4
2-3 m ahead of excavation face	211/indirectly simulated	NA	NA
2-3 m behind excavation face	305/304	320/indirectly simulated closure of 300	6500/indirectly simulated
Final excavation with 60 cm final shotcrete thickness	315/309	NA	9000/8500

**Table 12.**

*Simulation results from 2D models S4-1 and S4-4.*

closure of the sliding joints. Therefore, a value of 300 mm is considered in LC1 at the crown area, followed by a value of 308 and 309 mm at the end of LC2 and LC3, respectively. In both simulations the maximum axial force on the shotcrete shell is calculated after the total closure of the sliding joints, followed by the total excavation of the tunnel; 8500–9000 kN is calculated in model S4-1, while in S4-4 a maximum value of 8000–8500 kN is derived. In **Table 12** the results from both analyses are presented.

For tunnelling in extreme squeezing conditions, it is preferable for the parametric design to be performed with 2D FEM or FDM computational analysis models. Parametric analysis is mandatory in such cases, to examine the behavior of the tunnel for a range of the rock mass properties, since a small change in the intact rock strength or the overburden height or the GSI value can lead to different results. 2D FEM parametric modelling is quicker and, with the guidance provided herein, can lead to proper dimensioning of the required support measures. Of course, 3D FEM analysis can be used as additional to the 2D calculations in order to examine the longitudinal development of the displacements.

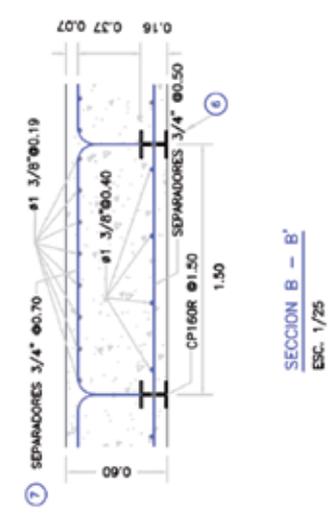
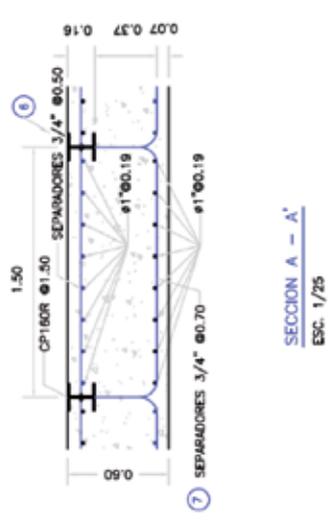
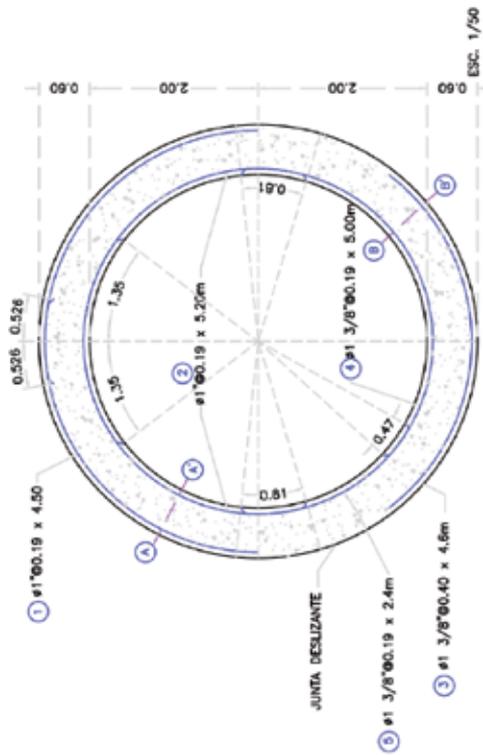
Following the results presented above, the S4 support class with 60 cm of shotcrete shell (which will be used as the permanent lining of the tunnel) and two sliding joints with 300 mm gap closure (**Figure 52**) proved adequate to be used for the un-excavated part of the tunnel. Further analysis of the results shows that possible reduction also on the final shotcrete shell thickness to 50 cm was also adequate, while 40 cm can also be utilized in special cases where the closure of the sliding joints was not fully mobilized (evidencing better rock conditions).

#### 4.5 Tunnel station at KP 6+540

Another also critical case that was mandatory to be examined in order to conclude on the support measures to be used for the un-excavated part was the findings near station KP 6+540 in the Entrada heading where a failure event was occurred.

After station 6+487, tunnel conditions were of GSI 50, 1225 m cover, and almost no ground deformation. For this situation a support type S3 (horseshoe) with 25 cm of shotcrete thickness was being used. In station 6+540 a tunnel slide occurred and was evaluated as a fault zone. While a geological, geotechnical, and support evaluation was in progress toward a heavier support, the contractor proceeded to place four steel ribs without sliding joints in S3 horseshoe geometry with 4.60 m excavation diameter. The measured deformations in sections before and after the main slide were of the order of 45 cm.

ACERO DEL SOPORTE DEFINITIVO		DIBUJO	SEPARACION	LONGITUD	ELEMENTOS ENTRE COSTILLAS	PESO (kg/m)	PESO (kg)	
N°	POSICION							
<b>ACERO SUPERIOR</b>								
1	EXTERNO		1	4.50	16	3.078	386.416	
2	INTERNO		1	5.20	16	3.078	330.079	
<b>ACERO INFERIOR</b>								
3	EXTERNO		13.8	0.40	4.60	4	7.913	145.599
4	INTERNO		13.8	0.19	5.00	8	7.913	316.820
5			13.8	0.19	2.40	8	7.913	151.900
<b>ACERO DE REPARACION</b>								
6	EXTERNO		3.4	0.50	1.70	30	2.237	114.087
7	INTERNO		3.4	0.70	2.24	20	2.237	100.218
						TOTAL	1445.759	



REV.	FECHA	DESCRIPCION
02	11/07/2003	ALTERNATIVA CON 1" DE MALLA
01	04/04/2003	REPARACION SUPERFICIE DEL ACERO

**SISTEMA HIDRAULICO YACAMBÚ HIDROR C.A.**  
 INGENIERIA  
 TALLER DE TRAZADOS  
 PUNTA DE SACA  
 SOPORTE 54  
 N° 1.15 650-36 TECHO 1100m P5= 1.50  
 P6 = 400 kg/cm2  
 escala: 1/25  
 modelo: 50-252-001  
 modificado:

Figure 52. S4 support class with 60 cm shotcrete thickness for station KP 11+700 [48].

The recommended excavation and support classes, according to Yacambu tunnel experience were:

- In the area of the fault zone, support type S4 with 40 cm of shotcrete in horse-shoe geometry with 4.60 m diameter (minimum internal diameter of 3.80 m).
- In the area before and after the fault zone, support type S4 in circular geometry.

The type S4 (horseshoe and circular geometry) has been examined in detail in this paragraph by 2D finite element analysis in order to investigate the implementation of type S4 section (horseshoe and circular geometry) during the long-term operation of the project as well as to use the findings for the un-excavated part of the tunnel.

Considering the event near the station KP 6+540, the conditions were considered as follows:

Rock mass classification: GSI 25

Tunnel cover: 1200 m

Horizontal-vertical stress ratio ( $K_0$ ): 1.0

An intact rock strength of 15 MPa is considered, and the following rock mass properties are adopted in the simulations:

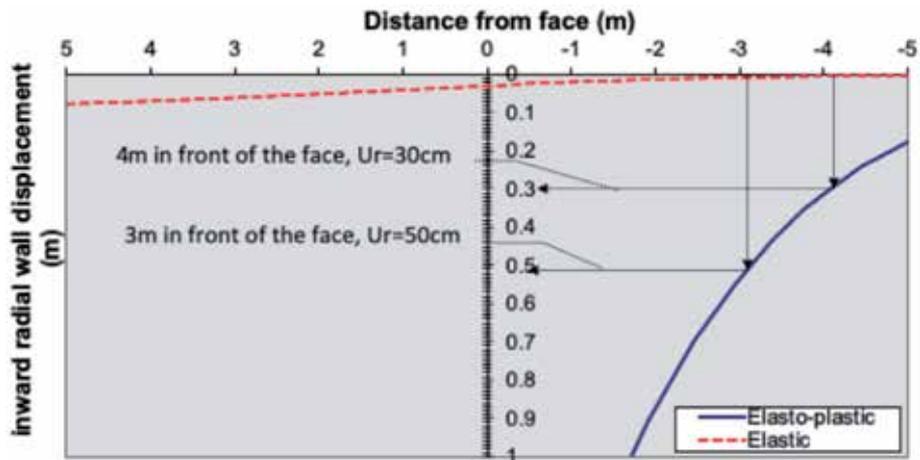
- Unit weight,  $\gamma$  (kN/m<sup>3</sup>): 25
- Cohesion,  $c$  (MPa): 0.963
- Friction angle,  $\varphi$  (°): 16.12
- Young modulus,  $E$  (MPa): 918
- Poisson ratio,  $\nu$ : 0.30
- Rock mass strength,  $\sigma_{cm}$ : 1.4 MPa

#### *4.5.1 Simulation stages in the area of the fault zone*

Using the convergence-confinement method (i.e., Panet) and elastoplastic response with Mohr-Coulomb law, the displacements 3–4 m ahead of the tunnel face are estimated 50–30 cm (see **Figure 53**).

Thus, before the tunnel sliding, the radial displacement is estimated about 30–50 cm. After the event, which happened in station 6+540 (tunnel slide) and was evaluated as a fault zone, additional displacements (45 cm), in sections before and after the main slide, were measured.

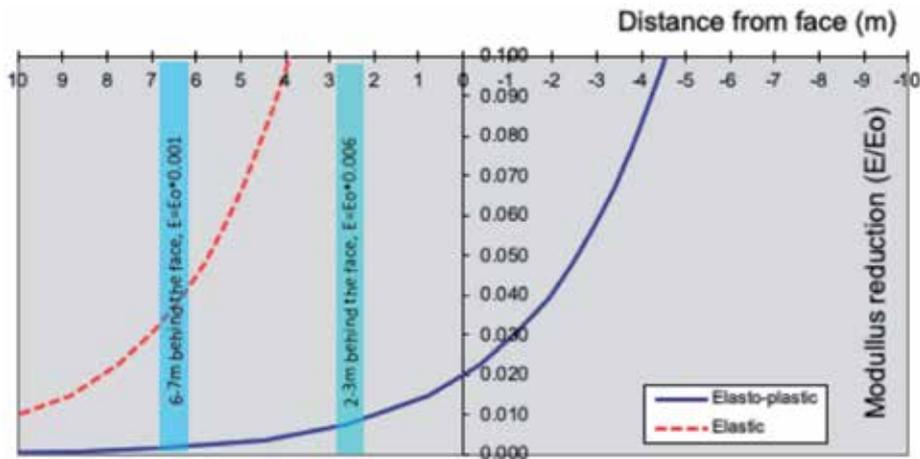
Considering the additional displacements (45 cm) due to the tunnel sliding as well as the radial displacements (30–35 cm) before the event, the total radial displacements in the area before and after the fault zone are estimated at about 75–80 cm. In the area of the fault zone, where the tunnel slid, the total radial deformations are estimated higher, about 80–100 cm. Following the above, four (4) 2D models are prepared using software SOFiSTiK [19] in order to check the adequacy of the shotcrete shell, as follows (**Table 13**).



**Figure 53.** Inward radial displacement as per convergence-confinement method for unsupported tunnel [21].

Model	Initial rock displacement considered (mm)	Tunnel profile	Diameter (m)	Shotcrete thickness (cm)
S4a-c	450	Circular	4.6	50
S4b-c	800	Circular	4.6	50
S4b-h	800	Horseshoe	4.6	40 crown 60 invert
S4c-h	1000	Horseshoe	4.6	40 crown 60 invert

**Table 13.** 2D FEM analysis for tunnel station KP 6+540.



**Figure 54.** Modulus of elasticity reduction as per convergence-confinement method [21].

The initial rock deformations 45 cm (S4a-c), 80 cm (S4b-c, S4b-h), and 100 cm (S4c-h) are investigated in Load Case LC1, by considering an initial reduction in elasticity modulus of the core of the order of 98% (45 cm), 99.3% (80 cm), and 99.5% (100 cm), respectively, as per Panet curves. In the next step, the installation of a temporary support shell with minimum stiffness is simulated

(10 cm shotcrete in tunnel periphery) with further reduction in elasticity modulus of the core (as it is presented in **Figure 54**) (Load Case LC2).

According to the tunnel construction sequence, fully cemented rock bolts (with diam. 25 mm up to 35 mm) are implemented in the previously erected steel set. Additional thickness of temporary support shell (5 cm) is simulated with further reduction in elasticity modulus of the core. Shotcrete thickness of 15 cm has reached its final strength (Load Case LC21). The simulation of excavation is fulfilled in Load Case LC3 by considering that the support measures will be mobilized 100% (full stiffening of the 50 cm shotcrete shell for S4a–c and S4b–c and of the 40 cm shotcrete shell in the crown and 60 cm in the invert for S4b–h and S4c–h). The produced stresses in the lining (by considering normal (N) and shear (V) forces, as well as bending moments (M)) are considered, and the appropriate radial and shear reinforcement is then calculated.

In a second step, the long-term behavior of the lining is investigated. A reduction of 15–20% of the rock mass properties is considered, and strain-softening material is used, as follows:

Young modulus,  $E = 918 \text{ MPa}$

Cohesion,  $c = 963 \text{ MPa}$

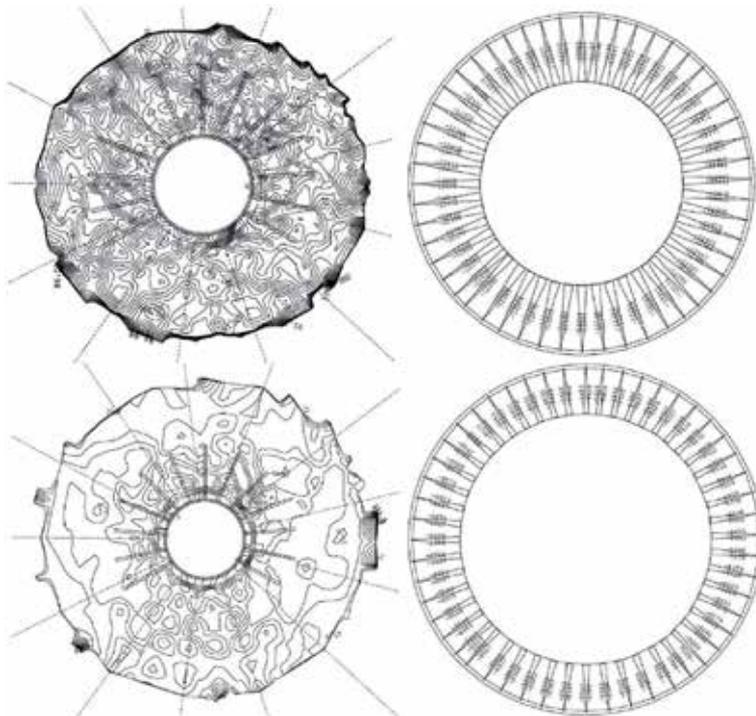
Friction angle,  $\varphi = 16.12^\circ$

Poisson ratio,  $\nu = 0.3$

Plastic ultimate strain,  $\varepsilon_u = 9.88\%$

Ultimate friction angle,  $\varphi_u = 15.30^\circ$

Ultimate cohesion,  $c_u = 0.871 \text{ MPa}$



**Figure 55.**

Model S4a–c and S4b–c, calculated plastic zone (top-left and bottom-left, respectively) and axial force (12,500 and 8500 kN, respectively) distribution (top-right and bottom-right, respectively) within the final shotcrete shell [21].

(Young modulus and ultimate shear strength are derived by the taken  $GSI = 25$ ,  $m_i = 10$ , and  $\sigma_{ci} = 15$  MPa).

Load Case LC4 is used as a long-term behavior calculation step. The thickness of the final lining is reduced to 40 cm for S4a–c and S4b–c and to 35 cm for S4b–h and S4c–h due to the risk of weathering of the external shotcrete layer. Rockbolts are considered as not functional.

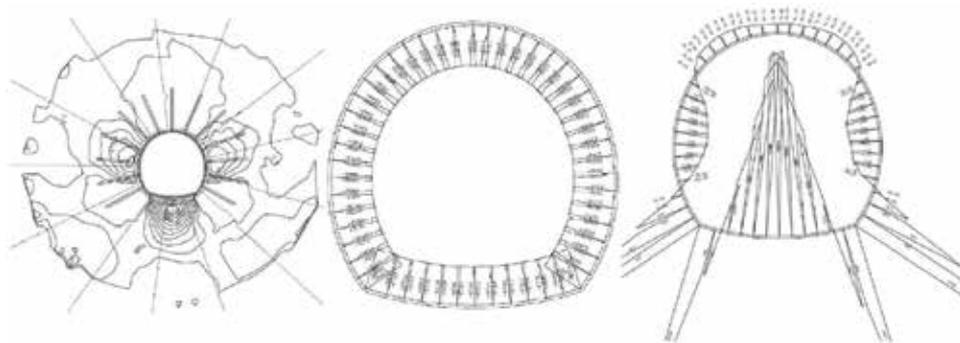
#### 4.5.2 Result presentation

Four (4) different models have been elaborated in order to investigate the sensitivity of the reinforcement results (for the final lining of S4, in either a circular and a horseshoe geometry), in comparison with the initial rock mass deformation (45, 80, 100 cm) due to the tunnel sliding that occurred near station at KP 6+540. In all models the detailed construction sequence has been analyzed along with the respective initial rock mass deformation. In the first model S4a–c the initial rock mass convergence that is simulated, before the installation of the temporary support, is 45 cm.

At the second model (S4b–c), it is assumed that the reported deformation of 45 cm (ING-03-52) does not include initial deformation occurred before the slide (estimated of the order of 30–35 cm), and therefore the total deformation is of the order of 80 cm. The resultant axial forces, regarding the first approach, are higher than the axial forces that are calculated with the second model (**Figure 55**).

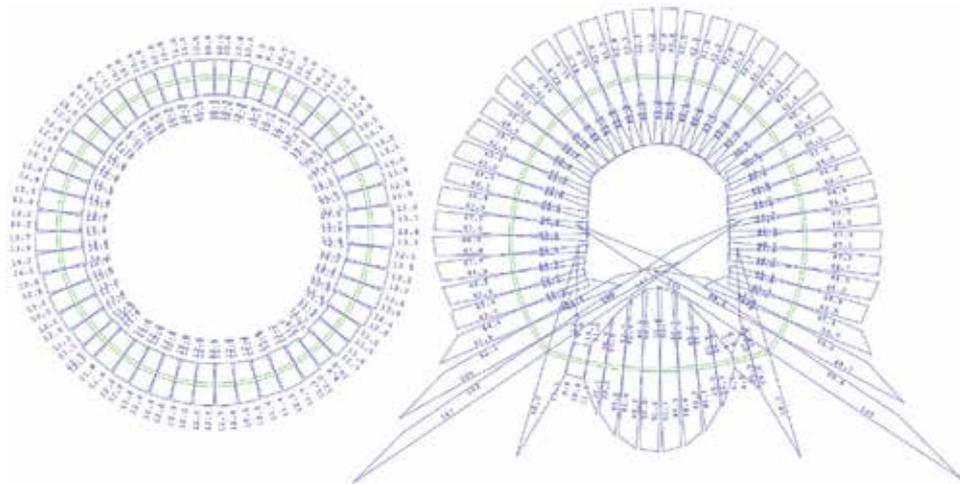
The second approach as mentioned above considers both convergences that are produced by (a) relaxation of the rock mass (30–35 cm) which is supposed to be occurred before the excavation and (b) additional measured displacement of the rock mass (before and after the fault zone) due to tunnel failure. The corresponding values of the axial and shear forces along with the moments that are developed in the flexible temporary lining is a result of the shell's displacement. Thus, the lower the displacement of the shell, the lower the axial and shear forces and moments. In a long-term basis, the stresses that were developed in the temporary lining are redistributed within the total thickness of the final lining. Considering the behavior of the tunnel near station 6+540, where no important shell displacements were occurred after the installation of the first 15 cm of shotcrete, it is assumed that the initial rock mass convergence before the installation of the shell was about 80 cm.

For models S4b–h and S4c–h, the radial initial displacement of the rock mass is estimated at about 80–100 cm, due to the fact that the area, which is investigated, is the fault zone, where the tunnel failed. The results for these analyses are unfavorable. In both approaches the bending moments and shear forces, in the connection area of the vault with the invert, are extremely higher than the previous model S4b–c (**Figure 56**).

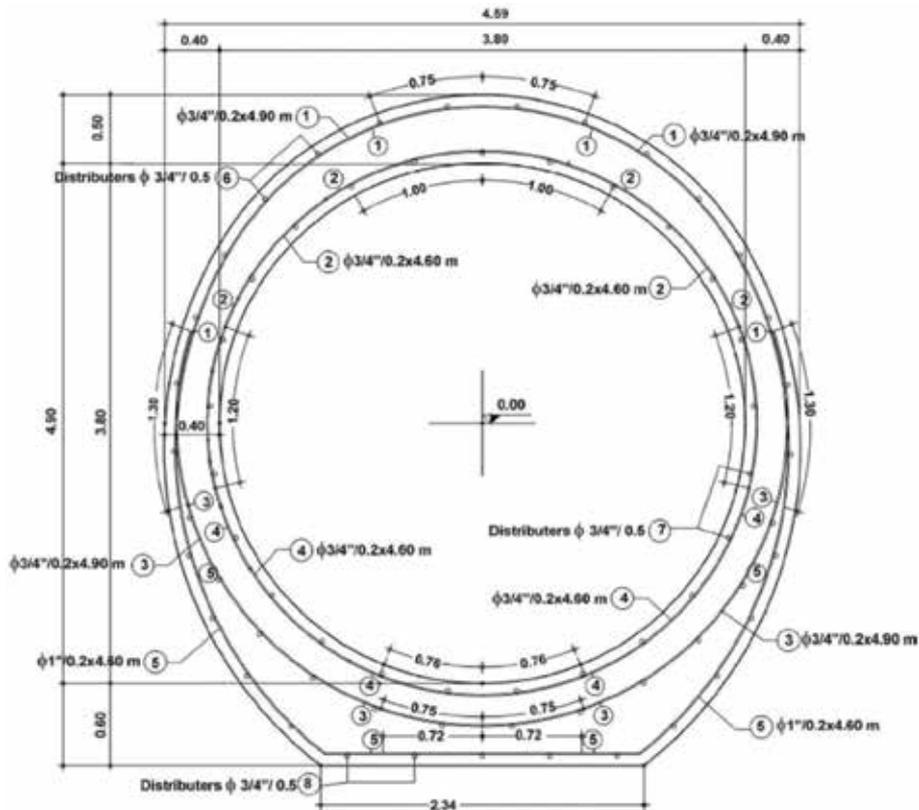


**Figure 56.** Model S4b–c, calculated plastic zone (left), axial force (9000 kN) distribution (middle), and bending moments (max 1300 kN m—right) within the final shotcrete shell [21].

The reason for the unfavorable results is the “bad” geometry of the horseshoe support class (see **Figure 48**). Even in cases that further failure has not been observed in the areas of adjustment to KP 6+540, where the horseshoe geometry was applied, a decision had to be made in order to secure the tunnel lining on a long-term basis.



**Figure 57.** Calculated required reinforcement for long-term tunnel operation. Model S4b-c (left) with max 14 cm<sup>2</sup> and model S4b-h (right) with 43 cm<sup>2</sup> at crown and max 140 cm<sup>2</sup> [21].



**Figure 58.** Conversion of horseshoe tunnel excavation profile to equivalent circular final lining for long-term tunnel operation [21].

Station	KP 11+700	KP 6+540
Overburden height, H (m)	1100	1200
GSI	35	25
$\sigma_{ci}$ (MPa)	20	15
Rock total radial closure (mm)	300–350	650–750
Sliding gap (mm)	2 × 300	3 × 300
Final lining axial force (kN)	8500	8500–12,500
Support class	S4	S5
Lining thickness (mm)	500	500–600

**Table 14.**  
Support classes and application criteria for GSI 25–35.

From the detailed FEM analysis results, it was clear that a circular geometry for the final lining during the long-term operation of the tunnel was able to withstand the rock load with a minimum required reinforcement of 14 cm<sup>2</sup> (Figure 57), while the horseshoe geometry was unfavorably affecting the capacity of the final lining. Therefore, special reinforcement arrangement was applied in order to convert the tunnel excavation profile to an equivalent circular final lining (Figure 58).

#### 4.6 Conclusive remarks for extreme squeezing conditions

The detailed analysis and back-analysis of the tunnel in the two critical stations at KP 11+700 and KP 6+540 describe the behavior of the excavation and consequently the support classes, under the worst geotechnical conditions which are governed by the presence of very weak carbonaceous and foliated phyllites (graphitic phyllite) with GSI <35 and intact rock strength from 15 to 30 MPa.

The tunnel excavation under maximum overburden height and in the presence of rock with GSI value of 35 and intact rock strength of 20 MPa can lead to a total radial closure of 300–350 mm. For the extreme case of GSI = 25 and  $\sigma_{ci}$  = 15 MPa, the total radial closure can be in the order of 650–750 mm. For both cases, the circular excavation profile of S4 and S5 support classes is suitable and needs to be implemented in order to secure the long-term operation of the final lining. Table 14 summarizes the findings from the parametric analysis.

As it is shown in detail in the respective chapters, the tunnel excavation under extreme squeezing conditions is only feasible by applying a flexible support category with sliding joints in a circular excavation profile.

## 5. Conclusions

In the present chapter of the book, the shallow and deep tunnel cases are explained in detail, and methods of designing are presented. For the reader and the tunnel designer, the following conclusions are summarized for the shallow and deep tunnel problems.

### 5.1 Shallow tunnelling in urban environment

Knowledge of the general macro-geology of the project area is always beneficial for the designer to establish a proper geotechnical campaign.

The general geological formations of the under-study area will be the governing parameter for establishing the density of the investigation boreholes and the density and type of laboratory tests.

Knowledge of the geological-geotechnical conditions will determine the type of the TBM shield to be used as well as the cutterhead design.

Combination of analytic solutions and computational advance modelling (FEM or FDM) is the most adequate method to be used in the design, since it reduces the time required for executing the calculations.

TBM tunnel interstation design is mandatory to be followed in Metro projects, due to the existence of surface and subsurface structures. The aim of the design to control the surface settlements and thus the operation of the TBM shields need to be determined by the designer.

Conservatism in determining the geotechnical properties of the soil must be avoided since it will result in overdesigning the permanent segmental lining of the tunnel and will also lead to unreasonably high support pressures with immediate detrimental effect on the operation of the machine.

The discrete parts A, B, and C determined the TBM TID and as such need to be strictly followed.

The application of the design support pressure (as derived from the TID) is the key factor along with the adequate tail shield grouting, in order to control the soil's settlements. The adequate application is to keep the chamber pressure constant in each stroke of the TBM, thus in each excavation step, as the shield is moving forward. Grouting from the tail shield of the TBM must be performed, and the amount and pressure of grout injected into the annular gap must be controlled by the pilot of the machine, and the consumed volume must always be monitored per ring along with the applied grouting pressure. Grouting amount in general should be 1.1–1.2 times of the interspace cubage. Secondary grouting from the segments should be performed based always on the live monitoring data.

## **5.2 Deep tunnelling in extreme squeezing conditions**

Anisotropic and foliated rocks such as graphitic phyllites, when met under high in situ pressure, can always lead to extreme squeezing problems.

For extreme squeezing conditions only, flexible supports can secure the stability of the underground excavation. Any other type of stiff support will result in failure.

Long-term behavior of weak rocks should always be investigated when permanent lining is designed.

The most adequate excavation profile is the circular geometry. Horseshoe geometries should be avoided since the resultant bending moments will lead to undesignable conditions. Larger tunnels (>5 m diameter) should be designed with a geometry as close as possible to a circle.

For larger tunnel diameters to be constructed in extreme squeezing conditions with increased number of sliding joints (or lining stress controllers (LSC)), 3D FEM or FDM computational analysis will be required with advance modelling of the sliding interface in order to investigate the closure sequence of the joints in relation to the advance of the tunnel face. Using the results from the 3D analysis, relaxation factors can be used in 2D modelling, and the temporary support measures dimensioning can be achieved.

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I would like to express my gratitude to Omikron Kappa SA for the more than 17 years of continuous cooperation that we have in performing designs for

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To download the chapter with high resolution images, please use the following link: <https://cdn.intechopen.com/public/docs/70605.pdf>

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Section 3

New Challenges in Tunnel  
Construction

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# Transit-Oriented Development Interactions on Existing Metro Systems: The Need for the Design of Adequate Structural Monitoring System and the Experience from International Projects

*Evangelos Astreinidis*

## Abstract

Contemporary metro transport systems present unrivaled efficiency for the commuting population. The development of the urban environment is interwoven with the metro transit systems. The transit-oriented development (TOD) is an upcoming topic in the design of the contemporary and of the future city and metro system alike. It entails the development of a microcell of the city centered around the metro station. Typically, bulky TOD buildings rise over and around the station and tunnel. The structural engineering aspect of these mega projects is highly complex. Major part of the complexity is due to complicated interactions between the oversite building and the underlying tunnel or station with its track-rail system. A significant number of issues arise, like methods to bridge over the tunnel or station, structural isolation, induced displacements to the track-rail system, tunnel movements and impact to tracks, vibration induction to the TOD building, and a plenitude of similar problems. It is highly important to design a structural monitoring system that will provide a validation tool of the structural-dynamic performance of the closed system TOD-tunnel/station. The distilled experience from international projects is presented.

**Keywords:** metro systems, tunnel stations, transit-oriented development, rail, track systems, displacement field, vibration field, structural monitoring, BIM

## 1. Introduction

This chapter is dedicated to a special engineering topic having to do with the structural interfaces of the metro to urban environment mega structures. The transit-oriented developments (TOD) are typically large real estate developments over or in the close vicinity of the metro stations. In the following, by the term metro is meant the whole metro installation including the tunnels, Stations and Switchboxes and all other structures like pop-ups and entrances. The social and financial importance of the TODs is very significant as they provide to the owner

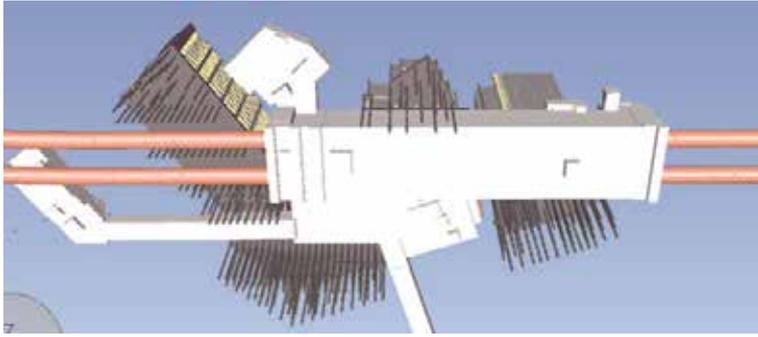
(usually the metro owner) great marketing privileges. The same time they are regarded as a major step toward the sustainable urban environment minimizing the use of cars (see for example [1, 2], also [www.tod.org](http://www.tod.org)). High rise buildings for office complexes, residential or hotel apartments, schools and hospitals, large malls and all other elements of the contemporary city life are built over or in walking distance to the metro. Thus, the technical concept of the TOD is threaded together with major structural issues. On one hand, the metro must bear TOD rising above and sustain the construction and service life loads and displacements envelopes safely. On the other hand the TOD shall be designed and constructed so that the inevitable effects on the existing metro shall be minimum and in all times within acceptable limits, without reducing the Design Service Life of it. The same time, the metro activities must not be blocked during the construction of the TOD or any case during the service life of it, while the noise and vibrations need to be filtered out on their way up to the residential areas of the TOD. In order to succeed in all the mentioned difficult tasks, special structural engineering considerations must be made and construction methods must be employed. The significant degree of uncertainty regarding the existing infrastructure is combined with the sensitivity of the metro and track rail system as well as the ambitious superstructure of the TOD giving a very cumbersome engineering undertaking. This makes unavoidable to employ methods to monitor the structural performance of both, the metro and the TOD. The monitoring system, as will be discussed in the following pages must be considered to have high specifications reflecting the important aspects of the analysis and design. It should never be considered to be a construction task left to the discretion of the contractor alone. It should be rather designed tailor made for the aforementioned engineering challenges and construction methods.

A typical TOD-metro combination is depicted in **Figures 1** and **2**. The figures come from a purpose made BIM exercise. In **Figure 1**, there is a bird's-eye view, while in **Figure 2** there is a bottom view, to show the distinct parts of the combined foundation of the TOD and the metro.

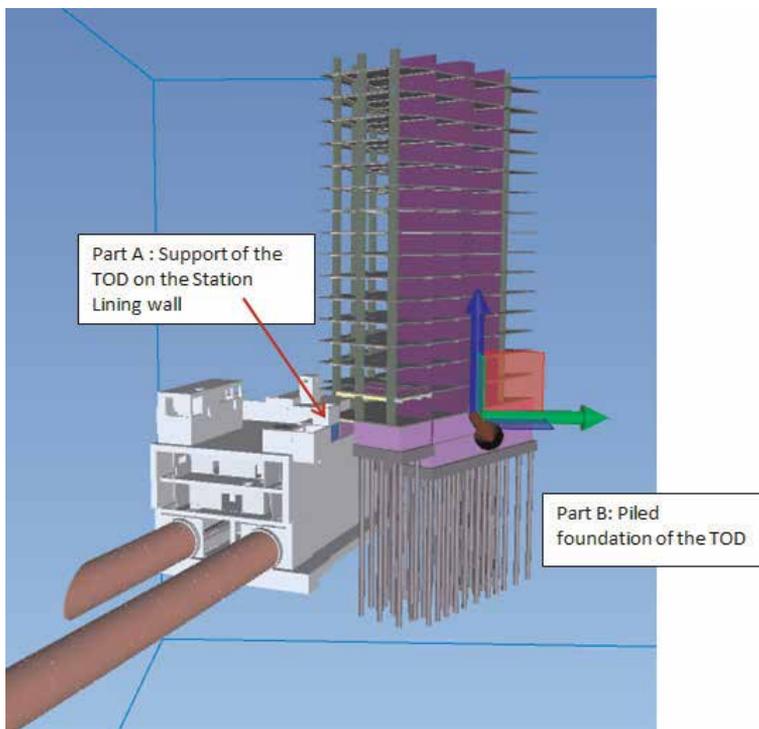
In the next pages some important structural considerations shall be presented, as they come from the experience of the TOD design over the metro of Qatar (**Figures 3** and **4**).



**Figure 1.** Typical BIM overview of the structural compound of TOD (in gray) and the metro station (in white) and tunnel (red).



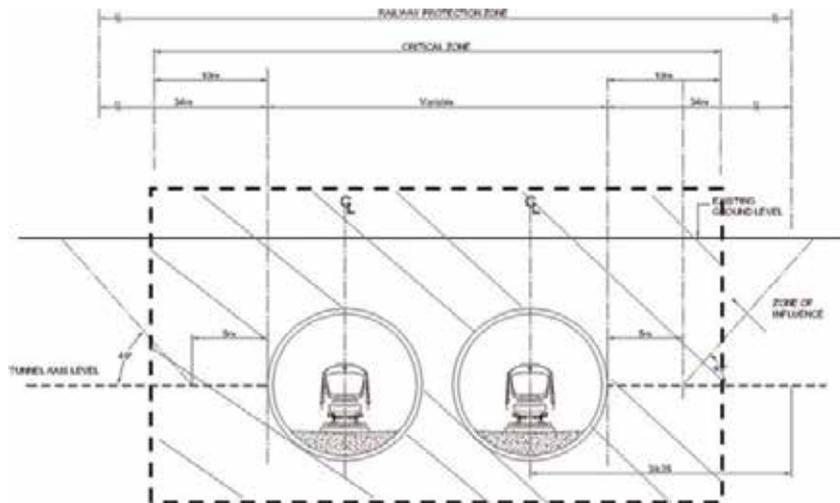
**Figure 2.**  
The same BIM model seen from a bottom angle to reveal the TOD piled foundation around the metro tunnels and structures.



**Figure 3.**  
Note the compound foundation system of the TOD. Part A is sitting on the metro, while part B is resting on a pile group.

More specifically, the following issues shall be discussed:

- a. Structural assessment of the existing metro structures (codes to consider, total structural behavior of the underground and superstructure and sub-structuring),
- b. Transfer systems (effects of transfer systems on the existing structures, configuration of pinned connection for the super structure),
- c. Control of the noise and vibration induced into the TODs.
- d. Displacement envelope induced in the existing track-rail system, either into the tunnel or at the Station-tunnel interfaces.



**Figure 4.**  
*Typical example of railway protection zone and critical zone around the metro tunnel.*

- e. Methods to insulate the metro from induced displacements.
- f. Structural monitoring and control of the construction and final TOD during its service life.
- g. Verification, validation and uncertainty quantification.

The challenges that are highlighted in this chapter lead to new unexplored areas which, as will be discussed below are:

- i. Simulation and computing demands

This includes issues regarding potential Static condensation capabilities in Finite Element structural software packages and numerical treatments like rigid link considerations

- ii. Multi-sensing monitoring—data fusion—situational awareness of Structural complexity

Special attention is given to the rail itself.

- iii. Corrosion monitoring—corrosion mitigation. Incorporation into the multi sensing monitoring
- iv. Special structural considerations like failure mode and effects analysis—fault tree analysis for critical structural members. There is a significant aspect of verification, validation and uncertainty quantification.

## **2. The framework of regulations for real estate development in the vicinity of the metro**

The starting point for the design of any structural intervention in the vicinity of the metro installations is the set of rail authorities regulations as depicted in technical guidelines and standards.

It is very important to make a reference of such regulations in this starting paragraph, as these constitute a prerequisite to any construction or structural activity near the metro and provide a glimpse of the structural issues of the metro-TOD interaction.

In most countries with advanced rail assets there are regulations regarding the permitting for real estate developments and in general construction activities by any third party in the vicinity of the rail-track.

Typically, there is a definition of the rail corridor which follows in a notional path on the ground surface the route of the rail track either, tunnel, at-grade or elevated.

Into the rail corridor, in general, there are conditions and requirements to be fulfilled in order to permit the construction activities. There is however a narrow zone, the critical zone, around the tunnel (in the case of underground metro) where the construction is in general prohibited. It may be allowed only after meticulous calculation of the effects on the metro assets, the tunnel and the rail-track system. In this case it seems that the most suitable analysis of effects may come from a Finite Element soil structure interaction analysis which should be very accurate in the calculation of the displacements of the rail.

In **Table 1**, a typical grading of the restrictions onto the railway corridor is depicted, depending to the proximity to the rail assets.

In general, into the Protection Zone there is a requirement for application for permit. During the review of the development all aspects of safety and operation of the railway are considered. There may be restricted construction activity, but the development is in general possible. The restrictions reflect the three major clusters of risks for the railway. The first is the risks related to potential damage to the structural part of the railway infrastructure like the tunnel concrete lining, the station, emergency exits, pop-ups of all kinds, etc. The second is the potential effects or blocking of metro operations due to construction activities close to entrances or pop-ups. A typical example is the risk of flooding to the existing metro installations due to excavations, earth moving and other construction site operations. But the most sensitive family of risks lays in the effects of the construction on the rail displacements. The tolerance to the rail displacements is always very small. The reader is referred to [3–7] for detailed presentation of the capacity of the track system to absorb displacements induced by the various construction phases. Especially the Deutsche Bahn standards are very sensitive to the lateral displacements at all expansion joints. For example, the expansion joints between the station to the switchbox, or even worse the tunnel to the station are very critical areas to check the induced displacements due to the construction program of the TOD.

It is the experience of the rail and metro authorities, that a soil structure interaction analysis will reveal the effects on concrete structure of the metro and the induced displacements on the rail. Moreover, some special construction restrictions

Zone	Permit	Restriction Level
Protection Zone (as depicted on surface)	Possible	Restricted
Influence Zone	Exception	High restriction (with limitations) – Special structural requirements
Critical Zone	Exception	Very high restriction (with limitations) – Special structural requirements

**Table 1.**  
*Grading of the construction activities depending on the proximity to the rail asset.*

or requirements should be imposed. For example the piling into the influence zone and especially into the critical zone may be required to include sleeves in order to reduce the development of pile skin friction. The pile sleeves are usually required down to approximately the depth of the tunnel invert, or to a depth under which there is no practical effect on the tunnel or the rail.

Monitoring of the tunnel or station and especially of the rail is a very important requirement in this case. The necessary accuracy and precision is important to be validated, but equally important is the real time character of the monitoring and reaction time for application of mitigation measures. A more detailed analysis of monitoring requirements shall be made in Section 3.9 of this chapter. For the purposes of the current paragraph it suffices to emphasize the real time character of the monitoring coupled with readiness of suitable mitigation measures.

Therefore, for any real estate development project within the rail corridor, the rail authority sets the following considerations regarding the existing metro structural assets:

Critical structures and issues:

- a. Station concrete shell
- b. Metro operation and construction restrictions
- c. Tunnel concrete lining
- d. Existing state of damage—cracked state of existing metro infrastructure and level of water ingress
- e. Existing reinforcement corrosion state—potential stray current
- f. Existing structure sensitive areas condition: sensitive items like the expansion joints, slender structural members or members with high utilization factor like columns, struts, pier heads working under very heavy punching loads.
- g. Induced displacements to the rail. Special focus on the interface between tunnel to station, expansion joints between station parts or station to switchbox, bridge to station, etc.

These considerations should be a prerequisite to any transit-oriented development structural design. Moreover, the structural monitoring requirements focused on the rail behavior, usually becomes the central part of the broader monitoring program of the TOD. The structural assessment of the existing Rail infrastructure becomes the starting point of the design phase. A more detailed presentation of the structural assessment of the existing structures shall be made in Section 3.2.

### **3. Transit-oriented developments over existing metro installations**

#### **3.1 Building extension over the metro station: the modeling issue**

In this section it is asserted that indispensable part of the structural analysis and design of the transit-oriented development (TOD) structures is the study of the effects on the existing metro structures (metro). Moreover, as the structural response of the TOD depends heavily on the underlying metro, the numerical simulation must include both of them. It should be highlighted that structural

interaction between the two structures TOD and metro exists even if the direct mechanical connection is partial.

Experience from TOD projects shows that it is a widespread belief among the various stakeholders that whenever there has been provision of a future Oversight building loading during the initial design of the metro, design work for the TOD could be eased and limited only to the superstructure part alone, i.e., the TOD itself. A widespread belief is that probably, the only care for the metro should be not to exceed the “allowable TOD reactions” for which it was initially designed. However, this is very rarely enough, as the existing metro forms part of the total structure. In other words the TOD and the metro are a combined structure. In this case, the stakeholders need to understand and consider the cost and risks that stem from the combined metro-TOD structure. More specifically, significant effort is needed to incorporate into the design of the TOD the metro as part of the total structure.

A dominant parameter for the design of the existing metro stations and tunnels is undeniably the future prospect of a major oversight building. This is reasonable and expected as a high rise superstructure connected to the underground metro installations, induces significant reactions and displacements. This is primarily due to gravity loads, but lateral loads (due to wind and earthquake) exerted on the building may also produce dominant loading combinations. Critical construction stages which may include one side excavations, leading to unbalanced lateral earth pressures, also turn out to be very demanding tests for the metro. Depending on the construction program the loading on the metro may be highly non-uniform, leading to structural loads for which it is not designed. At the time of the design of the existing metro structures, the actual architectural configuration of a future oversight building or buildings may have not been entirely known. Far less would be known the construction stages necessary for the actual TOD. Therefore, gross assumptions may have been taken into the structural design regarding the prospect transit-oriented developments. These assumptions comprise an initial necessary condition for the structural design as far the bearing capacity of the metro is concerned. The satisfaction of the necessary and sufficient condition for the metro to undertake the effects of the TOD is checked by the simulation of the precise configuration of the complete connected structure of the underground metro installations and the TOD superstructure, following the real construction phases. Both the existing metro and the TOD along with their structural connections and TOD independent foundation elements should be simulated as accurately as possible.

The ideal method to perform this simulation would be to incorporate in one model the whole metro and the TOD superstructure. The continuous modeling of the total structure, underground part and superstructure, in one model provides an uninterrupted displacement and stress flow. Arguably, this leads to safer results than any set of separate analysis of the metro, the TOD and their subassemblies. The benefits of the unified approach are pronounced in analysis for lateral loads from wind and earthquake. The computational cost, however, becomes an unsurpassed obstacle for this. The trivial method is to subdivide to two substructures, the underground metro box and the superstructure TOD and perform separate analyses. In this case the risk lurking is the intermediate and tedious phase of manual insertion of reactions from one substructure to the other. Apart from being a very tedious procedure which leaves large room for error, the risk of inadequate simulation of the total-unified response is always there. The soil-structure interaction in this case proves to be far more complicated, especially when the construction phases of the station box and the TOD building are distant to one another. A comprehensive study of this subject has been elaborated by O’Riorden in [8].

A substructuring method, if available, using static condensation to form super-elements for the underground part and the superstructure would provide better

control of the modeling work and better approximation of the structural response of the total structure.

### **3.2 Structural assessment of the existing metro assets**

It is highly recommended to perform an initial analysis of the metro as it is, prior to the introduction of TOD or its construction phases. This should be part of an initial structural assessment of the existing structure. This initial analysis may help to understand the structural response of the initial structure and reveal sensitive areas and critical failure modes.

Significant parameters to consider at this phase of the design have to do with the standards with which it was designed, the foreseen and the actual loading conditions but also with the current structural condition of the metro. Especially the crack formations and potential water ingress through cracks, voids, and construction joints, need to be taken seriously into account. The corrosion level of the reinforcement should be considered in case there are signs of corrosion or even better if there are measurements for the corrosion potential. It must be born in mind in this case that an allowance for the structural capacity should be made to simulate the cracking and the reinforcement section loss. The designer should carefully consider the actual stiffness of the metro. Potentially, a reduction of the modulus of elasticity should be considered.

In the structural monitoring section of this chapter, it will be mentioned that corrosion monitoring and mitigation should be introduced and should become part of the holistic structural health monitoring schedule of the total structure (metro and TOD).

In order to confront such structural assessment and modeling issues, appropriate sets of standards should be employed (see for example [9–12]). The standards for the assessment should deal with the necessary investigative works for the potential concrete deterioration, precondition surveys, etc. as well as the bridging of the design standards used for the existing structures and the current ones with which the TODs are under design. It is very important also that the design code employed will provide guidance on special modeling aspects pertinent to the special structural system of the combined TOD and Station box. American set of standards like ASCE-41 [9] and FEMA 356 [10] are more adept to provide guidance for existing structures of such importance. They also provide significant guidance for the numerical modeling. In the Euronorm family of standards, one can seek limited guidance in EN1998-05 (although this standard is dedicated to earthquake engineering) and the Greek Code of Structural Interventions [11]. Qatar Rail has compiled a necessary set of investigation practices as well as structural assessment best practices in Ref. [12].

Realistic reanalysis of the structure must not be over conservative. If it's conservative it'll prove the tunnel or the station inadequate to withstand the loads or the induced displacements. Careful selection of assumptions must be made in order to depict the real structural condition. Typical example is the simulation of the massive concrete joints of the existing structures. For example the “knee” and “tee” joints of the station box and also the joints of the columns to the massive bottom and top slabs require careful design of a rigid link.

### **3.3 The transfer systems**

The challenge of having a transfer system spanning over typical metro stations has been detailed in many significant publications, with some key references can be found in [13, 14]. The typical transfer slab is characterized by massive concrete and usually by a small span in comparison to its longitudinal direction. In what follows,

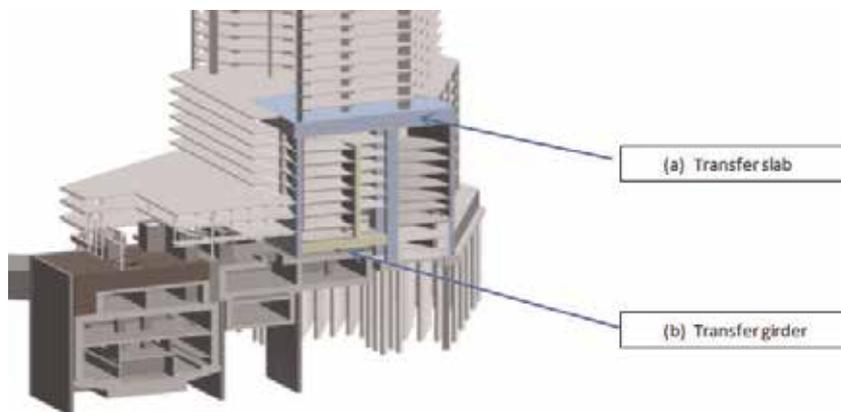
structural members made of massive concrete (MC) shall mean members with thickness more than 1.5 m and which are prone to exhibit high temperature gradient across the thickness during concrete hardening.

**Figures 5 and 6** depict characteristic configurations of transfer slabs spanning over stations.

In the case of direct connection of the TODs to the station box a secure path of stress flow from the superstructure to the station box is needed. The station box acts to a great extent as a foundation to the TODs, as a multi-cell concrete box circulating the stress flow between the lining walls and the main slabs (base slab, concourse and roof slab). However, there are sensitive structural members of the interior of the station box, like internal columns and stress concentration points due to openings that set limitations to excessive deformation due to the effects of the TOD superstructure. The most efficient way to transfer the TOD's actions on the station



**Figure 5.** Typical configuration of a transfer slab over a metro station. Note the oversite development (OSD) building transfer columns and their load redistribution role.



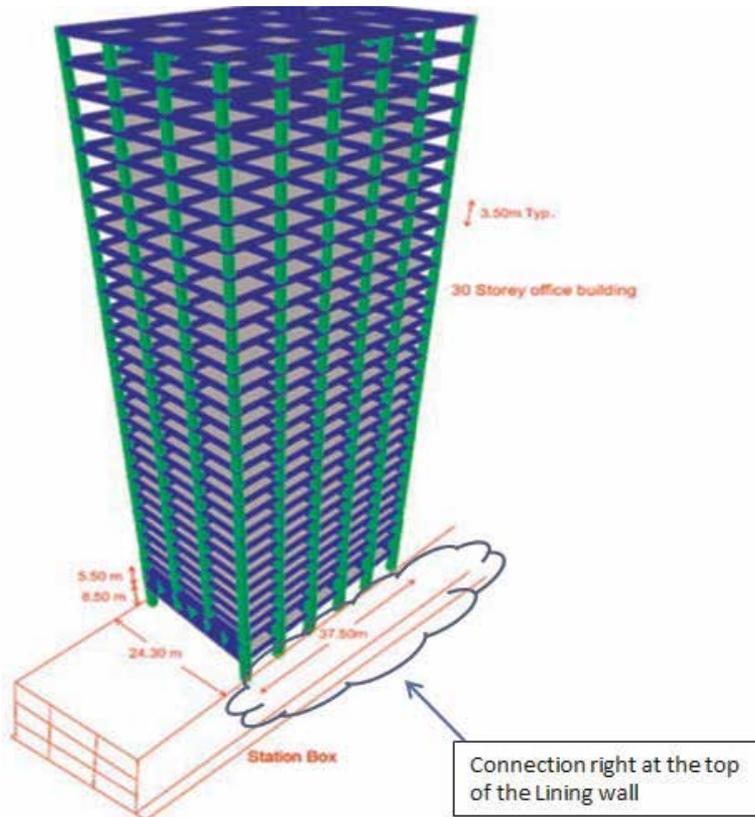
**Figure 6.** Transfer slab (a) at elevated position and (b) transfer girder.

box's hard points is the utilization of a transfer system that passes the reactions of the TOD's columns and cores at selected anchor points of the lining walls of the station (see for example **Figures 7 and 8**).

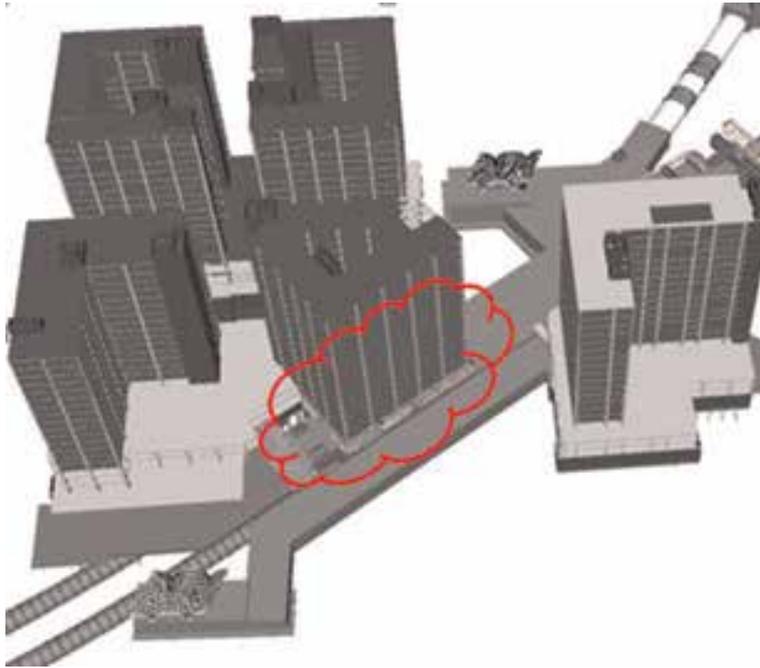
This transposition of vertical forces can be done either in the form of a massive concrete slab resting right on top of the lining walls and spanning over the roof slab of the station, or at any level above and well into the structure of the TOD (see for example [14]). Massive concrete considerations must be performed, including thermal stress, early age and long term cracking, etc. Heavy post tensioning sequence steps must be considered given the span of the slab over the station. The massive character of the slab helps to absorb vibrations from the station before they enter into the superstructure of the TODs, performing thus as the first line of defense for vibration isolation. In the case of the transfer system at elevated position, a hanging system is formed, usually employing deep steel trusses.

From the structural system point of view, the method of the massive concrete transfer slab resting in close proximity to the station box seems to have great benefits. The center of gravity is close to the station-foundation and provides small lever arm to the top of the "knee" joints of the station box. The connection to the lining walls can be made to behave as a pin with little effort. A potential draw back of the position close to the soil surface is the exposure class for the durability design, which in this case is harsher.

On the other hand the pop-ups of the station box and other surface utilities may obstruct the position of the transfer slab at that level. The architectural and mechanical engineering requirements are many times such that the position of the transfer system at an elevated position is one way to go. The lever arm of the mass



**Figure 7.**  
Idealized "module" of TOD "loading" all along the lining walls of the station box.



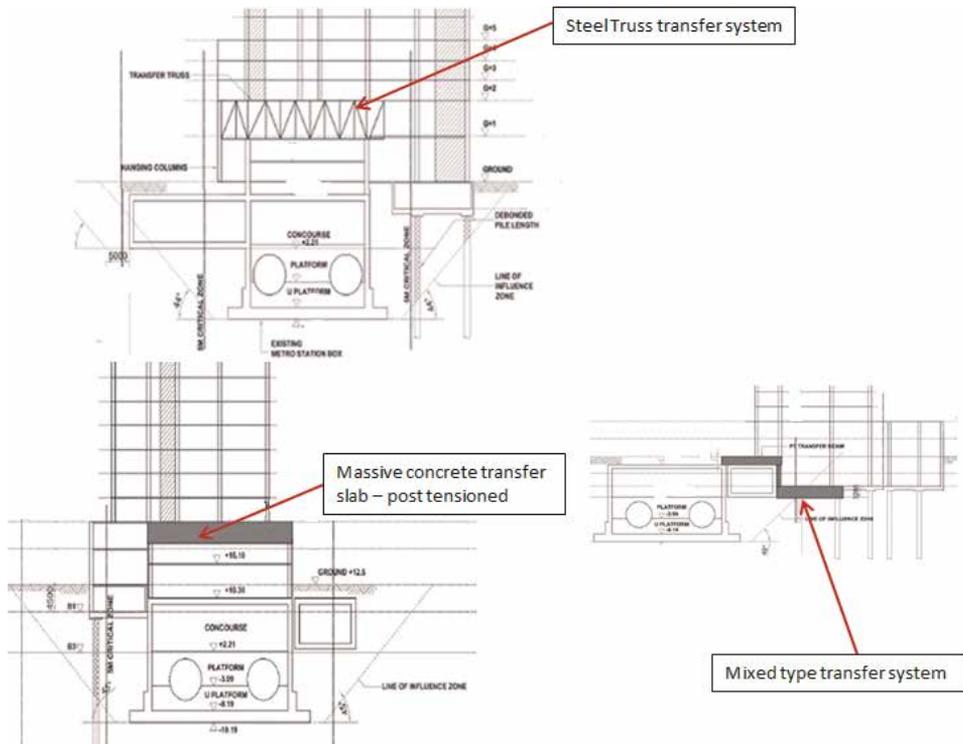
**Figure 8.** Actual TOD building configuration on top and besides the station box—in contrast to **Figure 1** of the initial concept. The reader should notice in the cloud the half-treading of the building only on some part of the station.

of the transfer slab is then very great, producing significant bending moments during lateral loadings. The construction of the slab at an elevated position is by itself a significant construction task, involving difficult logistics and heavy false-work the design of which is a cumbersome undertaking (**Figure 9**).

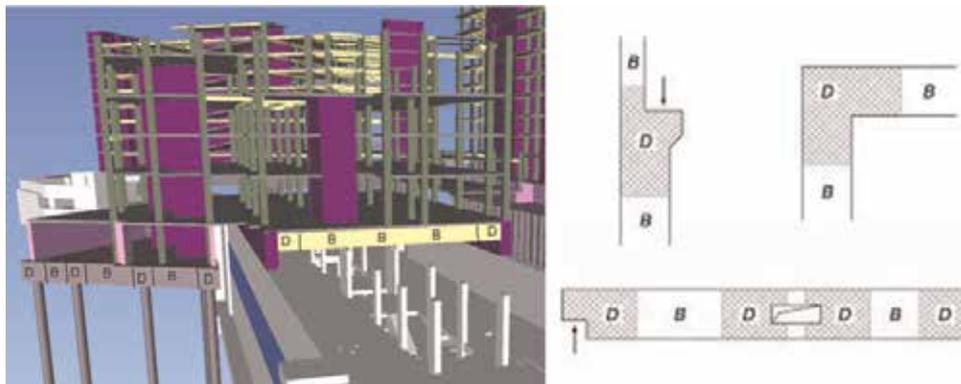
### 3.4 D and B regions in concrete beam analysis

Flexural behavior of beams and therefore slabs is well coded in international standards providing specific conditions apply which signify a more-or-less known stress flow pattern. This behavior is generally known as Bernoulli or Beam type (B type) flexural behavior as it complies with Bernoulli conditions of flexural behavior. In contrast, areas of beams or slabs that do not fulfill the necessary conditions for a Bernoulli type flexural behavior present a more complex, unpredicted and spurious stress flow, which necessitates delicate computational mechanics analysis methods. They are usually designated as D regions, named after the disturbed or discontinuity regions (see **Figure 10**). The only closed-form analytical method alternative to expensive Finite Element methods accredited by international codes is the strut and tie method. The success of the strut and tie method relies upon the experience and “art” of the modeler. Usually, it is employed for validation purposes rather than for the main design procedure. Certainly, the criticality of the transfer slab and the risks/impact involved requires a fool proof method, capable for representation of the full stress field in every region. These requirements are very high, well above the capacity of the ordinary Finite Shell element method or the strut and tie method.

The transfer beams and slabs differ from the ordinary, so as to say, everyday beams and slabs due to the ratio of their thickness to their length. There is no doubt that these slabs contain areas that belong to the D regions. Their thickness to span



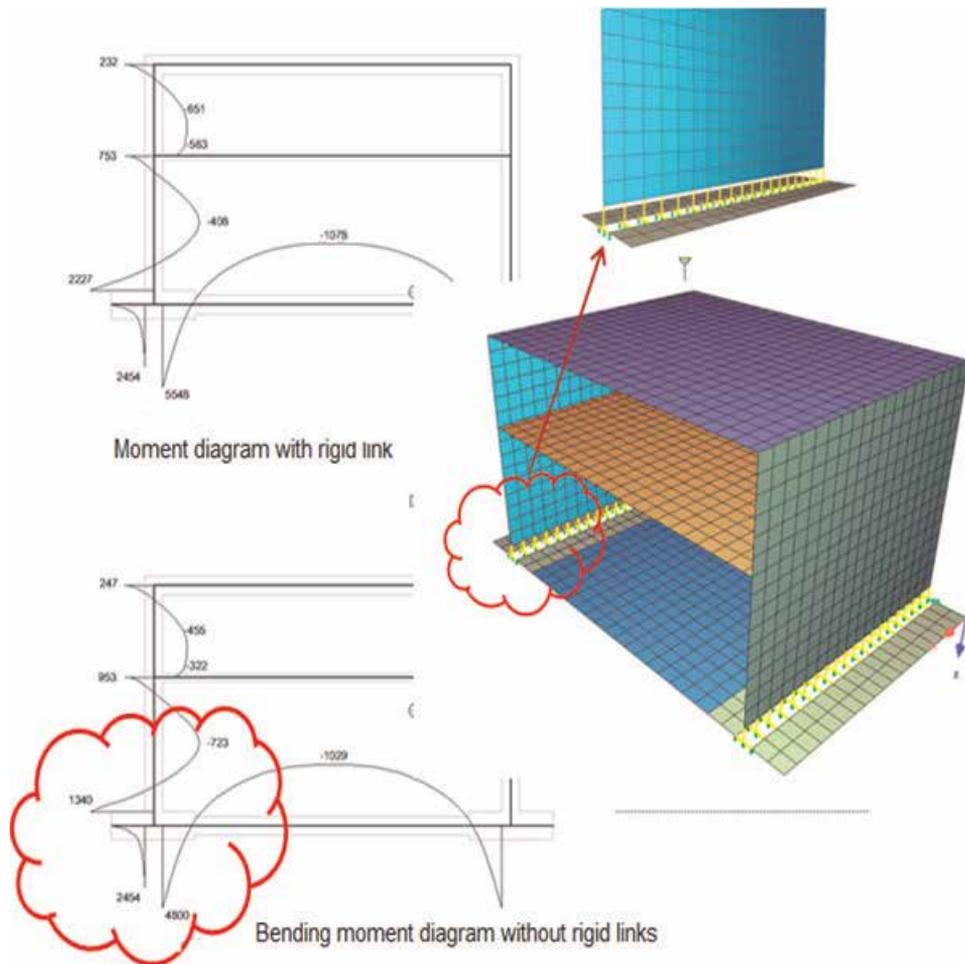
**Figure 9.**  
 Typical examples of the diverse methods of transferring the load through slabs-girders—trusses.



**Figure 10.**  
 Typical examples of D and B regions in transfer systems.

ratio and the MC joints produce stress flows that do not comply with Bernoulli beam bending theory. The classic Bernoulli beam bending theory may only be valid under certain conditions for it to be applied. Thick shell Finite Element methods can in some cases be a remedy, but the problem may persist in cases where the geometry does not allow discretization with shell or line elements anyway. The consideration of appropriate numerical treatment of D-regions, or in other words a non-shell-element FE area may be decisive for the correct analysis (**Figure 11**).

Moreover, all the mentioned D regions and concrete joints require dedicated methods of analysis, apt to simulate the Saint-Venant principle due to the



**Figure 11.** Characteristic case where FE shell elements without treatment of critical areas is inappropriate. The bending moment at the connection of the wall to base slab is 66% higher when rigid link is used.

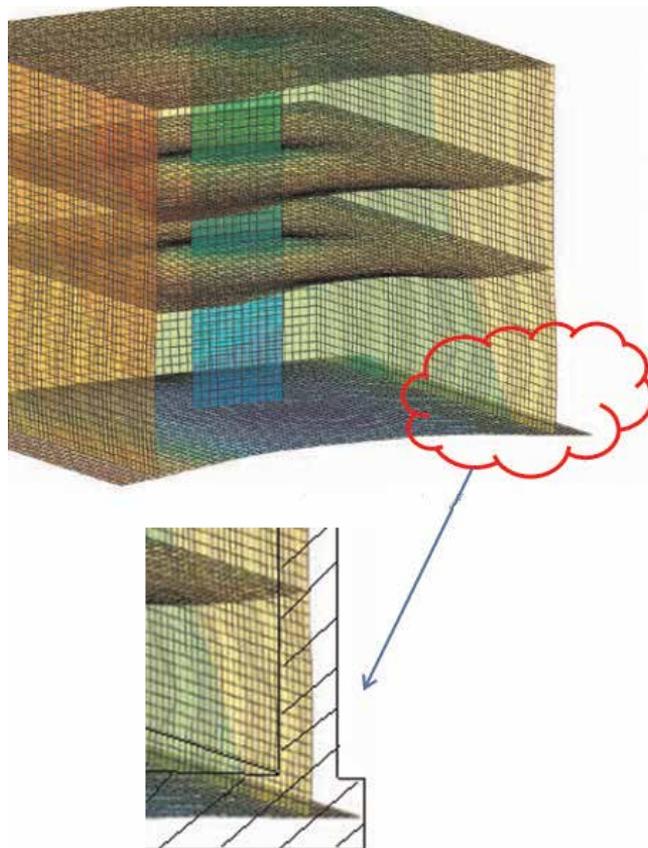
complicated stress bursting there. Posttensioning of the slab or girder contributing to strong stress bursting exacerbates the analysis complexity and increases the finite element requirements.

Methods capable to perform accurate simulation of the stress-strain flow of the transfer slab are either numerical simulations based on solid-3D elements or laboratory tests. The most accurate Finite Element methods to treat the MC and D region problems incorporate 3D and non-linear analyses. Thus, the work load and numerical effort needed increases significantly as well as the assumptions log.

The transfer slab analysis must take into consideration the massive concrete nature of the particular structural element, especially at its connections. The classic linear analysis, as this is offered by linear shell element analysis, may be inappropriate. The linear beam analysis assumptions, and especially the Bernoulli assumption of plain section rotations, usually are not applicable. The nature of this structural element is a thick shell. The infusion of stresses into the transfer slab is following a Saint-Venant bursting stress pattern that is not possible to be simulated with a mere rod-like line element approximation. Either an efficient local Finite Element analysis should be employed, or an appropriate strut and tie local model should be used to simulate the stress distribution in the joint.

To this point it is important to note that the concrete joint analysis of both the underground structure and the transfer system or other similar TOD massive structures must take into account rigid link analysis, wherever appropriate. As far as line-type elements, ASCE [9] and FEMA 356 [10] provide guidelines for the estimation of the length of the rigid link, with the ASCE standard being somewhat more sensitive to the actual concrete configuration. Birely et al. [15] offer a more delicate approach to this critical FE analysis issue (see also [16, 17]). Similar approaches should be followed for the MC joints formed by the surface members of the metro and TOD. The MC joints formed by the slabs and walls of the metro and the TOD transfer slabs and their connections are similar to the line element joints for which the mentioned literature is dedicated, although the actual engineering mechanics problem may be somewhat more complicated due to the influence of the stress components in the cross direction to the joint. It is to be noted that there aren't enough experimental or numerical analysis results published for this type of concrete connectivity.

In any case, neglecting the rigid link at the massive concrete connections may result in significant underestimation of the developed stresses and consequently the reinforcement requirements. Apart from the pure engineering mechanics fact that the core of the MC joint behaves differently from the field of the converging wall and slab, there is also a simulation necessity to incorporate a type of a rigid link. In **Figure 12**, it is evident that a refined FE discretization is not appropriate to simulate



**Figure 12.**

*A typical FE discretization of part of a station box. The actual concrete joint cannot be represented properly by the selected FE modeling. In the current configuration, which does not have any modeling treatment like a rigid link, the FE nodes that are located into the actual concrete joint do not represent the geometry or the stiffness of the joint. Obviously the shell-like representation chosen is inappropriate there.*

the core of the joint with the same type of FE as with the fields of the slab or wall. A similar geometrical problem is evident at the connection of the columns to the MC slabs—with more important the thick base slabs. If no rigid link is introduced, then the size representation of the column may be largely different. Potentially, an envelope of a lower and upper bound stresses corresponding to analysis with and without rigid link may provide a conservative approach.

### 3.5 Connection of the transfer system to the metro structures

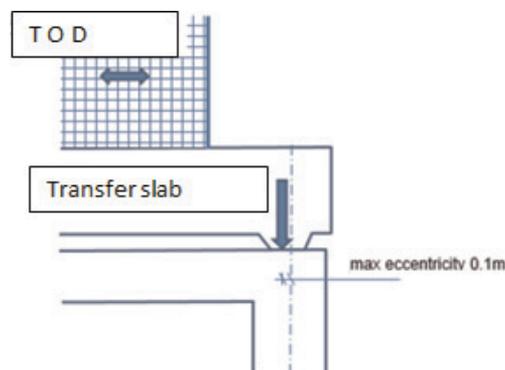
In the case of transfer system at an elevated position, the connection of it to the station box has a further complexity. Necessarily there will be vertical elements connecting it to the lining walls, but in this case significant bending moments will be induced into the “knee” joints of the roof slab (**Figure 13**).

This is unavoidable, because of the lever arm of the transfer slab and the slenderness of the vertical connection elements. The analysis of the existing “knee” joint must incorporate the joint stress flow appropriately. A pure and genuine pin and bracket connection is usually prohibitive, due to the magnitude of the gravity loads of the TOD. The requirements for water tightness and durability add greater difficulty in properly designing and constructing such configuration.

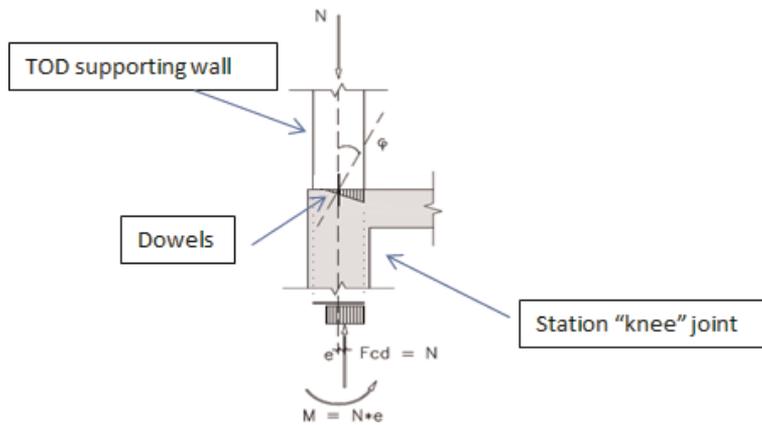
On the other hand an existing concrete station box may not have been designed for such additional bending moments, as, in the usual structural provision for future extensions, only vertical and horizontal components are considered. Therefore, the structural assessment of the existing structure must include the investigation of the “knee” joints of the existing metro structure.

A typical approach for reducing the bending moment development on top of such “knee” joints of the existing structures, involves a doweled connection configured primarily for shear (see **Figure 14**). It is noteworthy, that such a configuration may be simulated by a pure pinned connection for the TOD building with sufficient accuracy. This however, does not apply for the underground structure. Parasitic bending moments shall be developed, which may prove to be significant for the concrete shell of the existing structure.

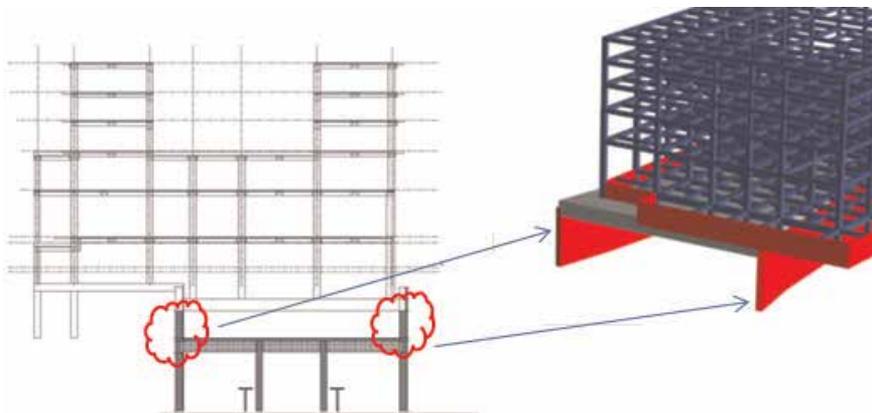
In such a case, it is necessary to investigate the actual rotational spring acting at the interface of the new wall or column resting on the existing “knee” joint. Either the actual moment rotation diagram of the concrete cross section may be used, or even a potential works analysis of the structural element rising above new structure may be used, to provide the rotational spring at that support of the TODs. A



**Figure 13.** Typical configuration of the transfer slab concept on top of the station roof. Note the close position of the rigid transfer slab to the top of the lining wall. In this case the concept of pinned connection may work sufficiently.



**Figure 14.** Typical development of parasitic bending moments due to dowel articulated connection between TOD supporting wall and station “knee” joints over the lining wall.



**Figure 15.** Connection of the TOD to the station “knee joints” of the station roof. The connection is meant between retaining walls and lining walls.

subsequent building analysis shall provide the parasitic bending moment, which in most such cases proves to be not negligible at all (**Figure 15**).

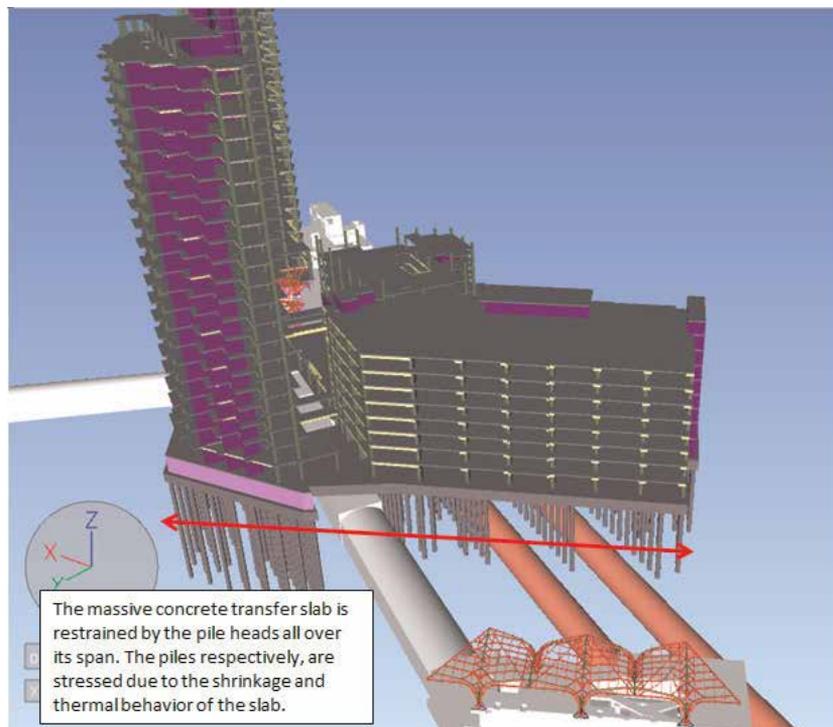
The usage of the transfer system smooths out the concentration of vertical forces induced by cores and arbitrary distribution of vertical elements in general. If, however, there is partial occupation of the plan view of the station area, then there is necessarily a bending effect on the station box that the transfer slab cannot mitigate. This is further aggravated by the lateral loads (wind and earthquake, if applicable) that will induce a torsional deformation shape to the station box. Openings and entrance interfaces will develop significant stress concentrations that need to be checked. A critical point may be internal columns. These may already have a significant utilization factor, due to the complex behavior of the station stress flow. Given the bending and torsional deformational behavior of the station box, due to the TOD effects, buckling and punching at the pier heads shall be checked thoroughly (see also [17]).

### 3.6 Thermal dissemination due to massive concrete hardening

One major challenge of MC is the significant thermal loading developed during hardening and the corresponding thermal strain gradient throughout the cross

section of the member. As the thickness of the MC is significant there are great temperature variations between the core and the outer skin of the member. It is noted that, this type of temperature loading is a highly non-linear and depends heavily on the thickness of the structure and the construction phases and joints provided. In the overall performance of the member, post tensioning analysis should take into consideration the residual stresses and strains due to hardening thermal loads. When highly aggressive environments must be considered, like for example in Qatar and in the Arab gulf states, in order to combine the necessary mechanical strength required and also to withstand the local weathering effects by corrosive agents and sulfate attack, a typical choice is to use high strength concrete mixes that provide solutions to many of the mentioned challenges. Concrete mixes with high percentage of binder like Grand Granulated Blast Furnace Slag (GGBFS) becomes often the choice. Nevertheless, there is still high risk for internal cracking of the MC, and this requires careful thermal dissipation analysis, shrinkage restraint analysis and construction joints design. Multiple layer construction and post tensioning increase the analysis and design demands.

The restrains to the slab or beam contraction may also be an additional analysis task. This is especially true in the case of MC transfer slabs acting as pile caps as shown in **Figure 16**. The pile head rows provide multiple restrain lines as they act as anchor points. The shrinkage and thermal loads may impose to the slab significant contraction strains and consequently the piles may experience very high shear loads. Crack development of the slab becomes the major design concern. FE analysis of the restraint of the slab due to shrinkage should rather be made on conservative assumptions. As far as the early age thermal cracking, CIRIA 660 Report [18] on early age thermal crack control offers some guidance on the type of restraint



**Figure 16.** Arrangement of the TOD over the tunnels and the pedestrian walkway. The transfer slab resting on the heavy piling in this case has a significant dimension. The massive concrete thermal considerations must be combined with the restraining effect of the piles.

formed. The analyst however, should be very cautious in his simulation of the boundary conditions whether there are conditions favoring end restraint to the notion of CIRIA 660. Stress concentrations due to abrupt stiffness changes, thickness, holes, may favor localized cracking rather than a somewhat evenly distributed crack pattern similar to the edge restraint pattern implied by CIRIA 660.

It is very important to perform mock ups of the transfer systems for calibrating the whole concreting procedure. Fruitful results also become available for the detailed design. Even so, structural monitoring of the performance of the actual transfer system is necessary. This is further discussed in the later section of this chapter regarding monitoring. Here it suffices to note that the transfer system is probably the most critical structure of the TOD-metro complex. The actual final condition of it will define its own structural performance, but also the structural performance of the total structure. The challenges due to massive concrete concreting, the actual stress flow into the massive concrete joints, the post tensioning, and the laminated concreting make the structural monitoring an indispensable validation tool.

### **3.7 Noise and vibration issues**

The train noise and vibration emissions due to the metro operation are a major concern for the near-by Real Estate developments in general. This is a highly specialized technical topic that deserves a dedicated presentation which unfortunately falls outside the main scope of this chapter. The main effort in this work is toward the more traditional structural analysis and design issues. It must be said however, that the TOD configuration dictates to a great degree the transmissivity of noise and vibrations coming from the operation of the metro. It is indeed very possible that the structural configuration as well as a wide range of material choices may be defined by it. Therefore, careful cooperation between architectural and structural design should incorporate also the noise and vibration transmission analysis.

It must be noted that the international experience shows that the vibration transmission can take place through the piles as well. Therefore, soil-structure interaction for the vibration transmission must take place.

The transmissivity of the vibrations through the station to the TOD is much different to the one from the tunnel to the nearby buildings, at the time of the tunnel boring. Therefore, even if there may be analysis and recording of measurements from the tunnel construction phase new measurements are necessary for the new compound structure TOD-metro.

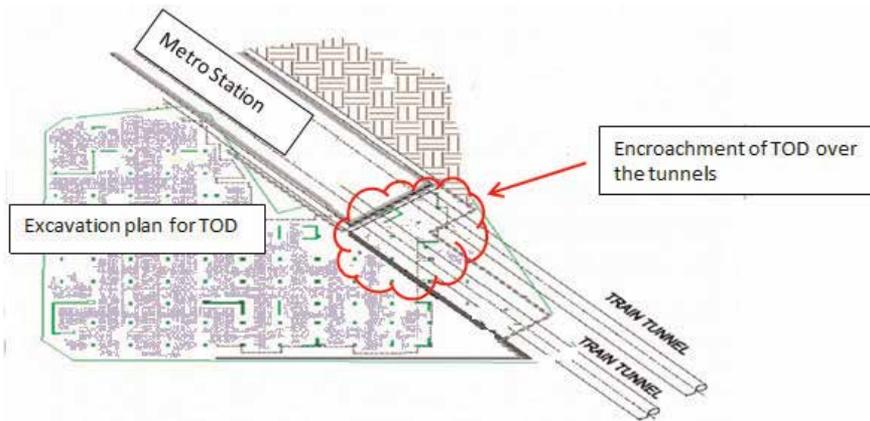
The most widely accepted set of acceptance criteria is probably FTA Report *Transit Noise and Vibration Impact Assessment* [19] (see also [20, 21]) .

There are no effective mitigation measures known to amend a vibrations inflicted development a posteriori. The vibrations prediction must be complete and influence the structural/architectural design.

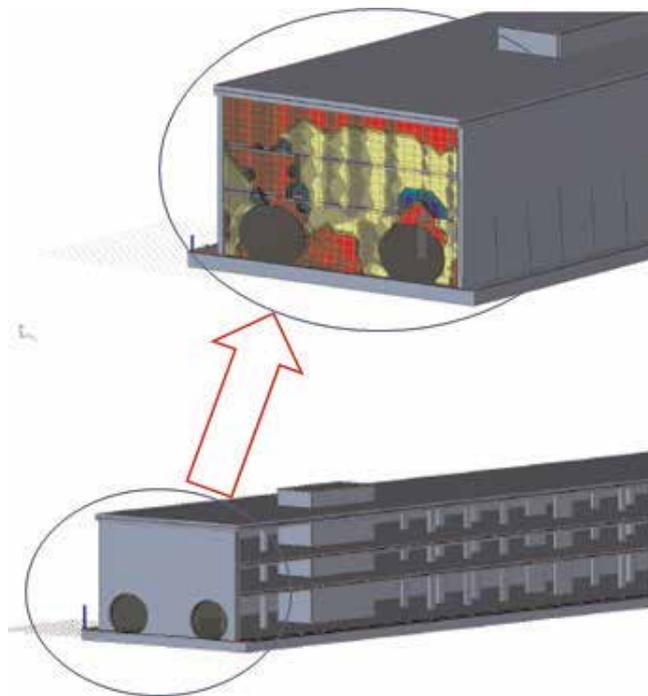
In general, a Finite Element model may be used for this. In some cases, the FE model is possible to play a dual role for structural-strength considerations and also for structural vibrations analysis. However, the discretization for vibrations analysis follows different and more stringent rules than the one for structural analysis. It might be practical however, to utilize the FE structural modeling for the vibration modeling as well.

### **3.8 Special geotechnical issues**

The soil structure interaction may be so great that the structural effects could be savaging to the metro structural infrastructure. In **Figures 17** and **18** below the

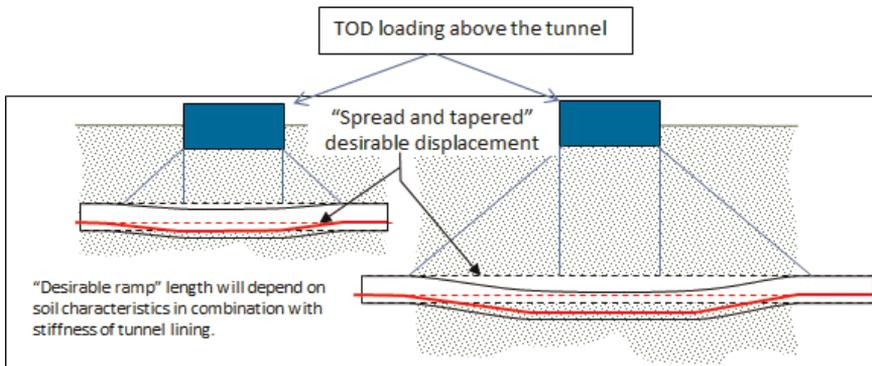


**Figure 17.** Plan view of excavation next to the metro, station-tunnel. The unbalanced lateral earth pressures need to be considered for each construction phase.



**Figure 18.** Due to excavation only from one side of the station box, significant unbalanced earth pressures raise high stress distribution at the head walls. Other distressed areas include the column pier caps and the tunnel to station expansion joint displacements.

effects of the excavation next to the station concrete box have been analyzed to show the unbalanced lateral earth pressures significant effects. The otherwise resilient thick concrete shell walls of the station box are experiencing significant stress development which needs to be checked for the main failure modes (cracking—water ingress). Nonetheless, slender structural members like columns, need to be checked for their predominant failure modes, like buckling or punching through their pier caps. Due to the unbalanced earth pressures, an overall box torque-like

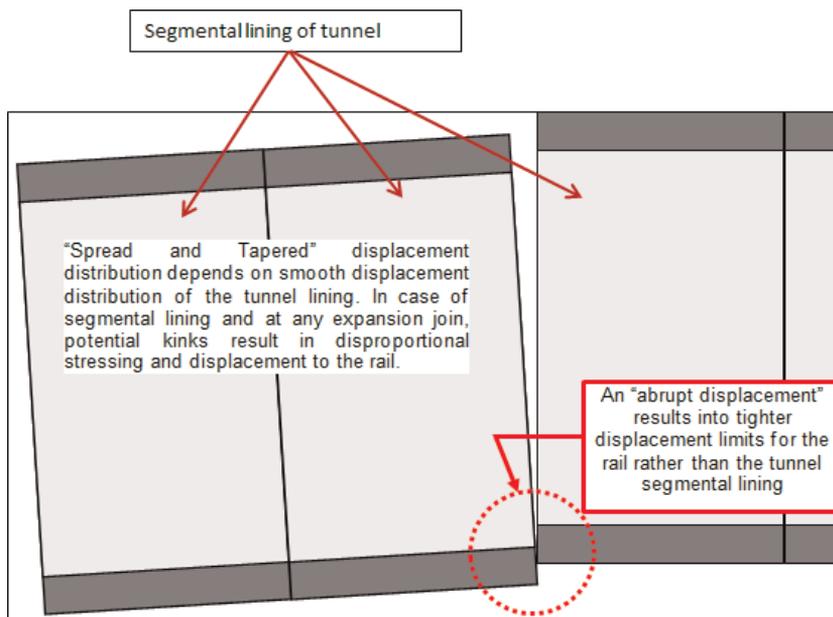


**Figure 19.**  
Desired “ramping” of the displacements.

displacement field may be induced which, among other stresses, increases the bending moments at the pier heads, making the punching check more demanding.

Regarding the very sensitive rail track displacement issue, **Figure 19** is very depictive about the tight tolerances. In practical terms, the expansion joint opening tolerances may be limited to a tight 10 to 15 mm depending on the joint capacity and the waterproofing or gasket allowance. The allowable displacement envelope induced to the rail however is far more restrictive. Refs. [3–5, 7] cited in Section 2 should be considered regarding the rail displacement. However, the Rail authorities concern is usually, to grade the induced displacement to enough length. In general displacements of the order of 5 mm are tolerable, but the most important characteristic of the desired displacement profile is the spread over a significant length of the rail in order to achieve a tapered deformation profile (**Figure 20**).

Typical allowable requirements for the rail into the tunnel and at the expansion joints are displayed in **Table 2**.



**Figure 20.**  
The expansion joint opening tolerances may be limited to a just a few millimeters.

Horizontal displacement of lining	20mm
Settlement of lining	10mm
Warning Level: Horizontal and vertical displacement measured on the rail	5mm
Alarm Level: Horizontal and vertical displacement measured at the rail	10mm

**Table 2.**  
*Suggested warning and alarm levels of displacements into the metro tunnel.*

The tunnel concrete shell is highly susceptible to induced displacements because of any third party construction activities, let alone the TOD loadings. Moreover, the tolerances of the rail-track system are an even greater challenge. In contrast to the concrete shell which can sustain displacements of the order of 15 mm, the requirements regarding the rail are far more restrictive. Depending on the Rail organization, approximately, a mere 5 mm are tolerable and that should be spread over a span of 10 meters over the rail length (see for example [22, 23]).

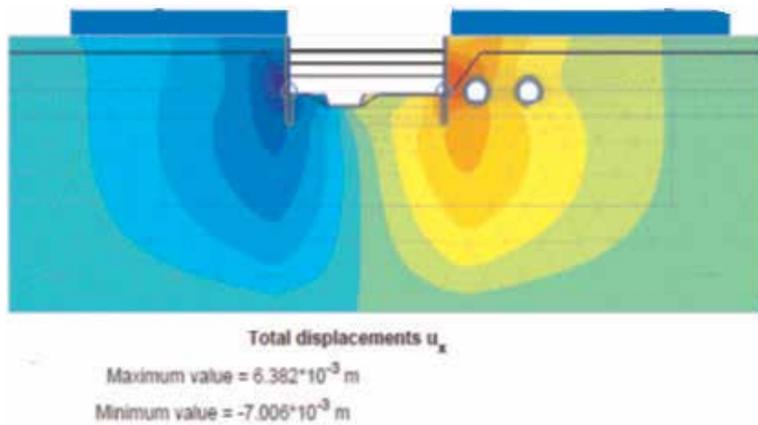
At the interfaces of the Station/Switchbox—Tunnels, or any other type of expansion joint, there may be a sharp rise of differential displacements due to the TOD. Significant design effort must be put to make sure that the differential displacements of the rail are within the allowable limitations.

Furthermore, in the case of ring segmental tunnel, it must be preserved at all times in compression, in order to avoid tension cracks or unacceptable movements at the segments' interfaces. Therefore, significant effort should be provided in FE analysis to accurately detail the effects of the TOD on the tunnels and especially at the interfaces to the station. If piles are needed to pass close to the tunnels, significant effects are expected to the tunnels in terms of displacements and stresses. Single or double sleeved piles could be used in order to reduce the pile-soil friction and avoid stressing the tunnel to the degree possible. A comprehensive numerical study of the simulation of the sleeved piles in such cases can be found [24]. It must be always remembered however, that the friction reduction methods could not eliminate completely the friction, allowing therefore, a significant stress field to affect the tunnel or even the station box.

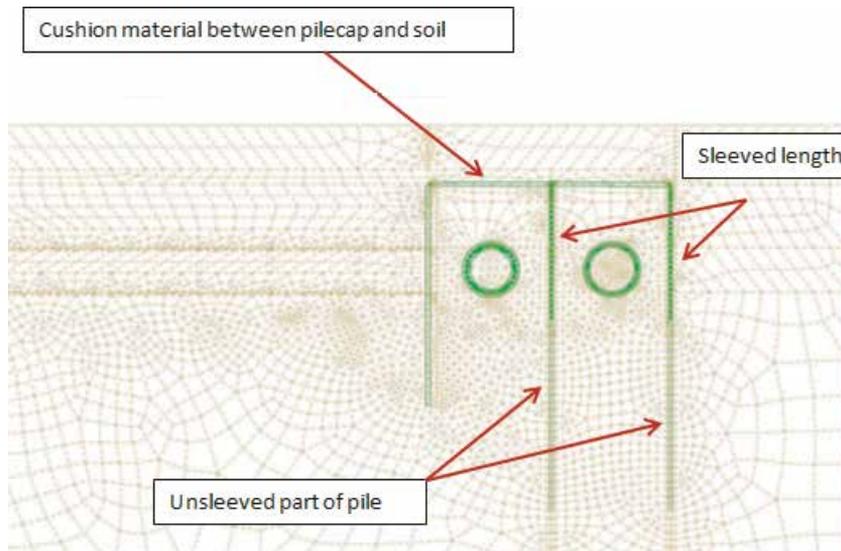
The arrangement of the foundation system of the TOD alongside the tunnels poses a significant challenge. In order to avoid stressing the tunnel lining or inducing unacceptable displacements, it often required to use pile skin friction reduction systems. The sleeved system offers a reduced pile skin friction but does not eliminate it. It must be remembered that it does not get reduced to more than 50% or maximum 30%. It is very difficult to verify what exactly has been achieved during the construction of the pile. Therefore, it is strongly recommended to follow a very conservative approach in the design, especially regarding the friction developed at the sleeved part of the pile. The vertical springs that simulate the friction should be appropriately treated in the model, while the lateral ones, which simulate the lateral resistance, should not be forgotten (**Figure 21**).

The pile cap will be a major source of bearing pressure to the soil, if in full contact, despite the axial resistance of the piles that they connect. Therefore, if a separation cannot be achieved, a cushion material should be designed and suitably inserted to detach the slab from the soil, as shown in **Figure 22**. This is not an easy task at all, either to design or to implement in construction. The degree to which this detachment is achieved is difficult to be verified.

Since there are so many sources of uncertainty, it may be wise to consider in the design envelopes of upper and lower bounds of induced stresses to the tunnel,



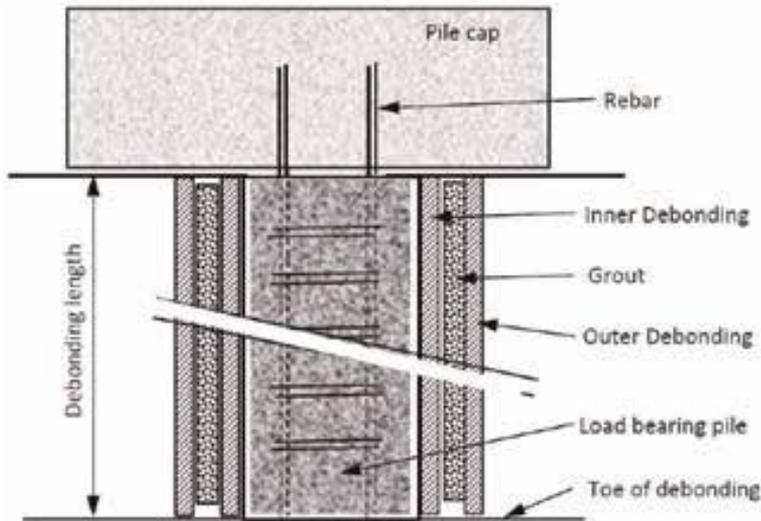
**Figure 21.** Excavation for the TOD next to the tunnels. Experience shows that even at hard soils, like the Simsima Limestone of Doha, there may be significant displacements to the tunnels. And while the effects on the lining may be tolerable, the displacements to the track system may be prohibitive if no stabilization methods for the excavation are not in place.



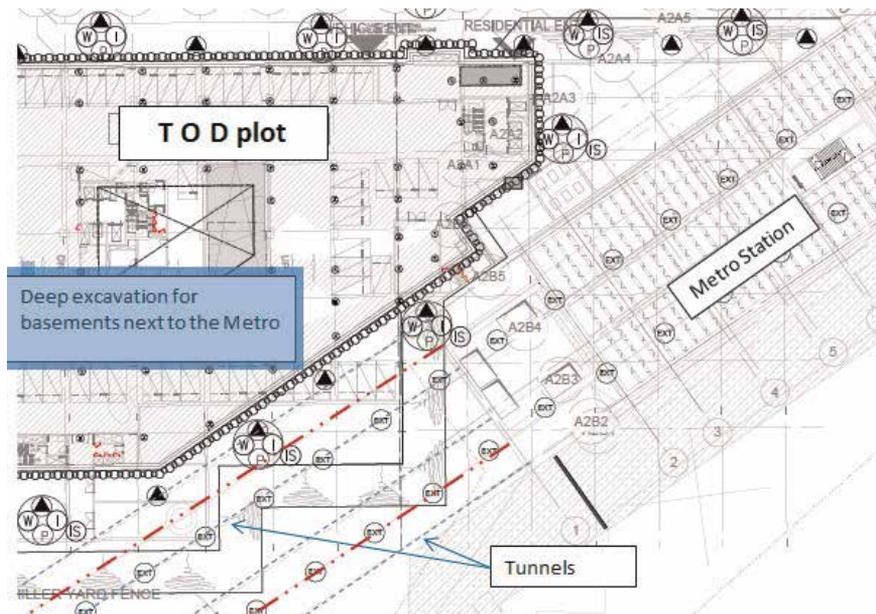
**Figure 22.** FE analysis of effects of progressive excavation and TOD construction over the tunnels with segmental lining. As the piles are sleeved all the way down to the level of the tunnel invert, their representation into the FE model includes removal of the pile skin friction to a specific degree. This reduction is not suggested to be more than 30%.

representing the maximum and minimum friction sleeve isolation and soil pressure by the pile cap (**Figure 23**).

In **Figure 24**, a typical arrangement of geotechnical instrumentation as is shown. With such an arrangement it is possible to monitor the soil mass, and probably this may lead to reliable conclusions regarding the stress-strain conditions of the tunnel lining or the station (see also Refs. [25–27]). This can be achieved by comparing the readings of the instruments to the values of the displacement—stresses from the soil structure interaction analysis. But it becomes entirely indirect in the case of the more sensitive entities like the rail itself or even the expansion joints. In order to obtain the picture regarding the actual stress-strain conditions of the tunnel or the station it is important to have a direct instrumentation attached on carefully

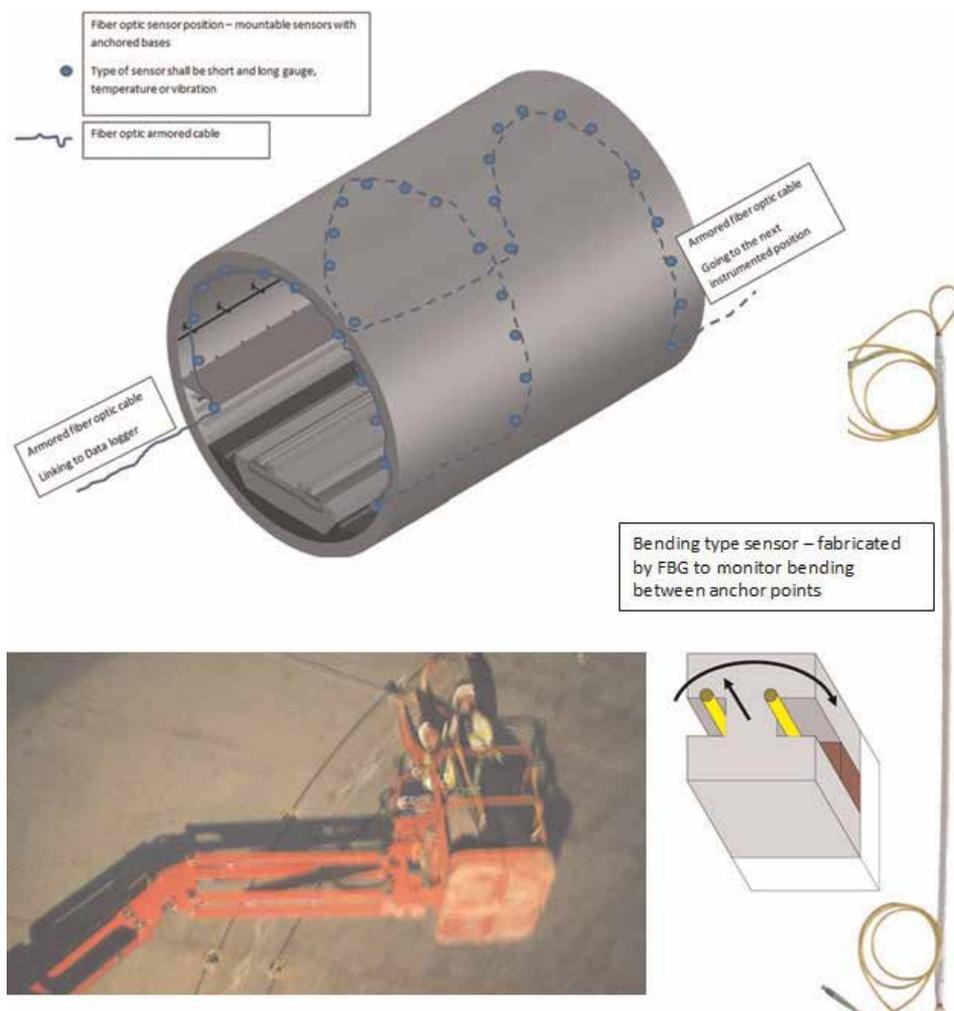


**Figure 23.** Typical arrangement for pile debonding. Notice that the lateral support is not eliminated and should be considered in the simulation accordingly.



**Figure 24.** Plan view of new construction next to the metro station and tunnels. A typical distribution of geotechnical instruments to monitor the surrounding soil of tunnel and station. Notation: EXT: extensometer, I: inclinometer, W: water pressure meter, P: piezometer.

selected positions into the structures. A more direct strain-stress distribution insight can be gathered from the instrumentation of **Figure 25**, where a fiber optic instrumentation is shown to “wrap around” the tunnel lining for a critical length. Such strain instrumentation is useful to provide the picture of the tunnel lining distortion at a critical area of interaction with the TOD or other third party construction activities. But the cost in this case becomes too high. On the other hand, if a mountable type of sensor is used, then the instrumentation can be retractable and is



**Figure 25.** *The original FBG sensor, which is fabricated to operate in axial tension, has typically excellent characteristics in terms of accuracy and repeatability. When used into a compound sensor, to monitor other degrees of freedom (like rotation) may have dramatically reduced its fine characteristics. Testing and calibration are indispensable in such cases.*

not “spent” in the specific section of the tunnel. It then can be used for a period of time to measure the effects caused and when requested can be transferred to another section. Only the bases to which the sensors are fixed are spent. The drawback in this case is the accuracy of the mountable sensor. It must always be remembered that intrinsic effects of the sensors are the gauge length, the attachment and the fixation, even the rotation of the attachment to the desired direction.

### 3.9 Structural monitoring

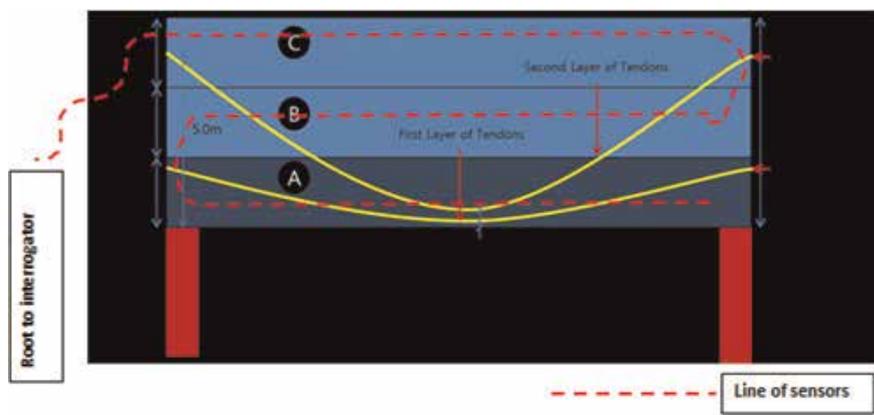
The structural monitoring of buildings although not entirely rare does not take place often. The experience built up until now is not comparable to the one of the geotechnical monitoring. A comprehensive presentation of recommendations and best practices can be found in Ref. [28]. It is the intention of this chapter though to support that structural monitoring is a necessary and indispensable part of the design of the TOD-existing metro compound. It is not an issue to be left to the

contractor alone, as it seems to be a widespread belief. There are many reasons why the designer should specify the structural monitoring instrumentation and its monitoring targets. These include verification of critical design parameters (and how these evolve in due service life time) and construction loading parameters which affect the existing metro structures and for which there may be many unknowns. In general, in the consideration of the compound TOD-metro structure there is accumulated significant uncertainty mainly due to the long assumptions list for the behavior of both parts but more importantly of their interaction. Tolerable displacements of each part of the compound structure may mean significant change in the boundary conditions of the other part. Moreover, the interface between TOD and metro, as configured in the transfer systems may have itself significant uncertainty built up. To give a few examples, concrete deterioration of the transfer slabs or girders may mean not only reduced service life of the members themselves or of the TOD but also altered reaction envelope for the metro, which receives them. Reversely, concrete deterioration of the metro may ignite change in the boundary conditions for the TOD. Concrete deterioration may be of limited or controllable character for the structural member itself, but may produce disproportional alteration to the stress distribution over the TOD-metro compound.

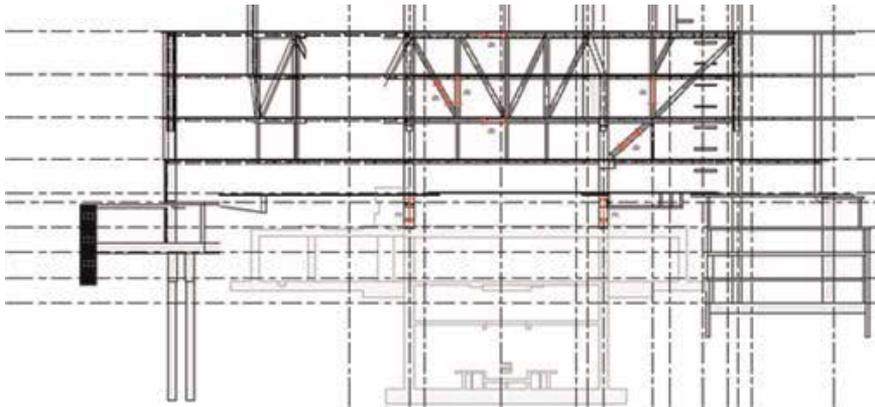
Exactly because such reaction changes at the interface of TOD-metro due to any damage accumulation or structural performance are unpredictable it is necessary to monitor the condition of the critical structural members. The transfer systems in this endeavor are high in the priority list.

The nature of the transfer systems makes the employment of advanced monitoring instrumentation necessary. The target of the monitoring should be the validation of the stress-strain levels, the displacement field and also the structural condition of the members monitored. In order to make possible to synthesize the stress-strain gradient picture throughout the total section of the member the instrumentation should have robust characteristics (**Figures 26–28**).

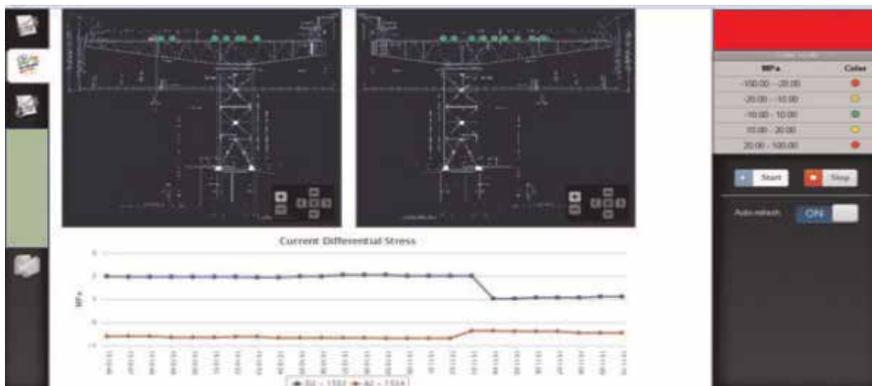
- Robustness and redundancy of sensors: As a basic rule, it's highly preferable to have not only robustness in the sensing systems but also redundancy of the sensors, despite the additional costs. Potential damage of the cabling of the sensors during the construction shall reduce drastically the life span of the instrumentation. Therefore, significant budget for SHM is dedicated to cable protection in the form of conduits, extra protection for cable splices, sensor



**Figure 26.** A triple layer construction sequence envisaged for thick slab transfer system of a TOD over the metro tunnels. Note the post tensioning tendons and proposed arrays of strain-thermal sensors.



**Figure 27.**  
*In this transfer steel truss system the members marked with red symbol are of greatest concern to the designer. He therefore designed a strain monitoring system to validate the stress-strain development during the erection of the structure and building up of the loads.*



**Figure 28.**  
*A situational awareness visualization tool is necessary to obtain the overall picture and the same time to be able to locate on the BIM the section of interest to get the sensor analysis in detail. In the figure each green dot on the top of the structure represents a set of two sensors. Their stress flow history is shown in the bottom of the screen. On the right hand side of the screen their stress analysis level is compared to the green-to-red chromatic scale of alarm.*

covers, etc. Moreover it's highly recommended to pass alternative or additional cable routes with redundant sensors at least at the most critical members/areas.

- **Complementarity:** For the sake of validation, complementary verification systems should be specified in order to verify the results and validate the method. (E.g., strain sensors should be complemented by displacement sensors and if possible with vibration monitoring, displacements complemented with pressure cells, etc.)
- **Coverage of the area of interest:** Orthonormal grids of sensors are recommended to be installed. They should follow the principal stress flow in surface elements. In case of line elements (columns-beams) multiple sensors should be employed for monitoring the rotation of neutral axis.
- **Inspectability of the system:** In all cases, appropriate armored conduit should be used, especially in the cases were the cables are exposed and not within the structural member.

In order to achieve such characteristics, the designer should specify wisely the instrumentation characteristics. Parameters to be considered should include:

- Specify the stress range
- Specify the gauge length
- Specify the Degrees of freedom of the sensor
- Specify the attachment method and stiffness of the attachment in relation to the substrate
- Specify the cover of the sensor

Investigate:

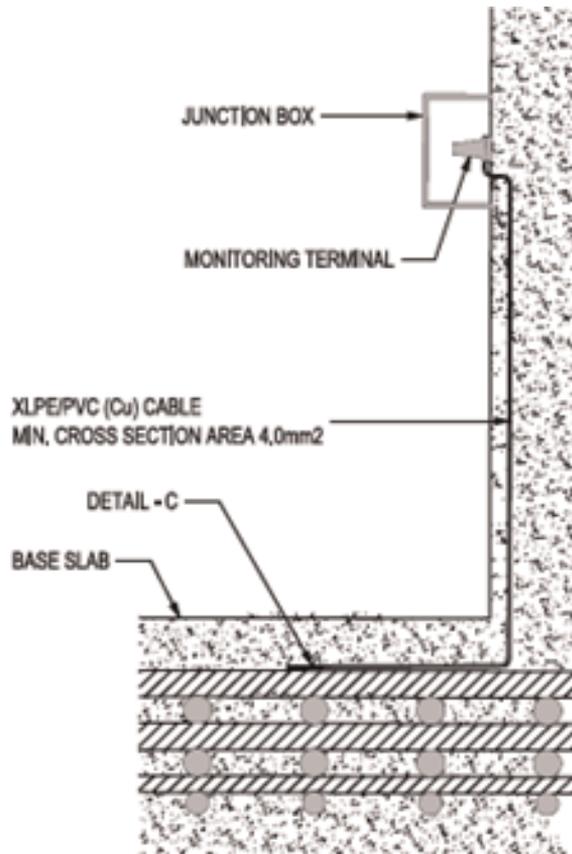
- a. The Modulus of Elasticity of the structure
- b. Temperature compensation
- c. Moisture and drift

At the sensor points a member cross section analysis can be performed to provide the distribution of stresses and strains to compare with the monitored strain distribution. It follows that careful positioning and attachment of the sensors is necessary. In order to obtain a clear picture of the stress condition of the structural member and the notional rotation of the neutral axis it's rather necessary to use more than one sensor at every cross section.

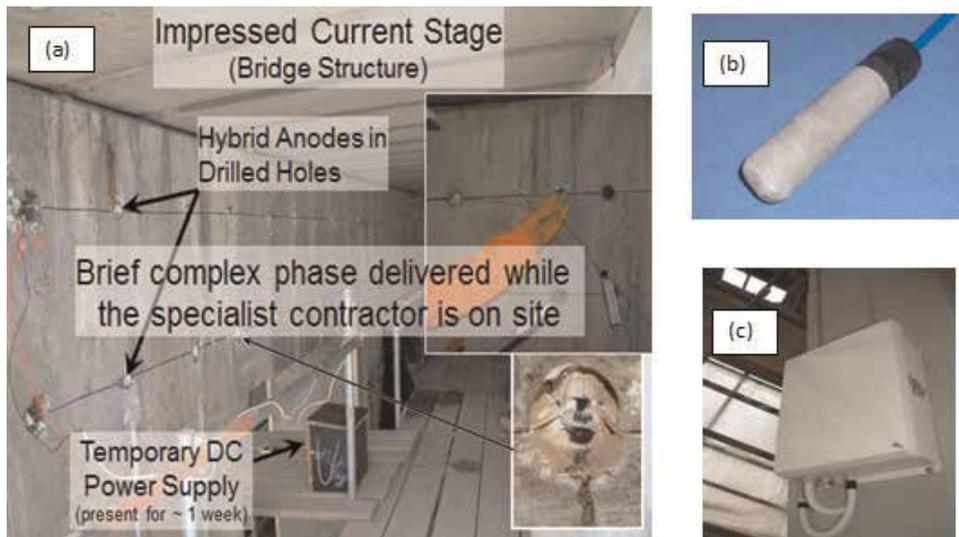
A significant part of the structural monitoring, which deserves separate analysis is the corrosion monitoring. Corrosion of reinforcement and especially of tendons is critical concrete deterioration which must be early detected and mitigated. Instrumentation for corrosion monitoring may in some cases be combined by cathodic protection infrastructure as shown for example in **Figure 29**. The inspection port is meant to be used for incipient current cathodic protection. Otherwise, separate apparatus may be installed. This may be comprised by embedded half-cell potential wiring, reference electrodes, or chloride ingress detection devices. The risk of corrosion of the post tension tendons is especially high in structures close to the ground and to ground water. In regions where the soil is rich in chlorides like in the Arabic Sea states, even underwater there is significant prospect of corrosion, through formation of macrocell development. This is noted to happen in underground heavily reinforced concrete boxes of stations and switchboxes (see also [29, 30]). Macro cell corrosion, where the cathodic reaction taking place in an aerated zone of the structure and the anodic process taking place the depassivated area may produce non-expansive products making the phenomenon undetected until it will be too late. This has been found to be occurring in cases of post tensioned tendons and especially when the duct grouting is problematic.

Active corrosion protection is by far a more prudent way to secure the Design Service Life of the structure. The high costs for distributed anodes or incipient current protection pay off as a major concrete deterioration risk is removed. The hybrid type of anodes has shown excellent characteristics while they offer the advantage of periodic chloride ion removal (see also [29–32]) (**Figure 30**).

The application of such a method for active cathodic protection all over the foundation system of the TOD may be very costly. The critical structural elements however, should be seriously considered to be protected, especially when post tensioning is present.



**Figure 29.** Cross section of base slab to lining wall concrete joint. A monitoring terminal is shown. It is used for connectivity check, but can in theory serve for corrosion monitoring and incipient current.



**Figure 30.** (a) Distribution of hybrid anodes, (b) reference electrode, and (c) transmitter for remote sensing.

#### **4. Verification, validation, and uncertainty quantification**

From all the major points discussed in the preceding sections of this chapter it must have been evident by now that the structural compound of the TOD and the metro is a highly complex system. It is a complex system in the sense that it presents emergent properties: i.e., properties that are not apparent from their structural members when these are studied in isolation but which result from the relationships and dependences when they are finally constructed [33–36]. The assumptions log of the total structural analysis is being increased too much because of the interaction of structures and structural elements so different to each other. The predominant Failure modes of metro are different when the TOD is built over it, as the ones of the structural elements of the TOD when considered on a fixed foundation rather than on the metro. The displacement limits of the rail-track measured at millimeter scale are in significant contrast to the large scale Finite Element model of the metro-TOD compound which produces so great reactions in magnitude, has enormous computing requirements, it has many assumptions pertinent to large scale structural systems rather than the one comprised by the delicate rail-track system. Let alone the inherent uncertainties regarding the existing metro structural condition. The same time there are critical structural elements like the transfer systems that require a thorough verification of their functionality.

In order to keep the Uncertainty contained in acceptable levels, a rigorous Verification and Validation system is necessary to be employed. As far as design is concerned, it should not focus only on producing reliable simulations, in terms of Finite Element analysis and BIM, but it should focus on accumulation of evidence of the credibility of the simulation results. This should be an important element of the risk management plan for both the TOD and if possible of the metro.

It is asserted in this chapter that the laboratory testing of critical structural members and also data collection from structural monitoring are vital parts of uncertainty quantification procedure and that validation is achieved through compound simulation, testing and structural monitoring.

As has already been said in Section 3.6, mock up tests for concrete are very important. Mock ups of massive concrete would provide information about the actual heat dissipation flow of the specific concrete mix to be used. Strength build up, thermal gradient and thermal stressing are vital pieces of information that are needed for the most critical structural members of the TOD, like for example the concrete transfer slabs or girders.

Unfortunately, the uncertainty regarding the performance of them is deemed to be built up with the size (not only the thickness), the reinforcement quantity, concreting conditions, weather conditions, concreting layers, construction joints, post tensioning and stress flow into the joints. Therefore, a good solution to cast light into all these complicated and intrinsic procedures, series of laboratory tests could be organized. The same time, data collected from structural monitoring are necessary to provide the picture of the actual developments of the monitored parameters.

A great benefit would be to identify the failure modes of the transfer systems. The laboratory testing should focus on each failure mode and organize precautions for them. The precautions should have the form of simulation calibration parameters for further analysis during the final Detailed Design phase, construction, construction sequence, maintenance and monitoring during the service time.

The risks from concrete casting in layers should be investigated. Thermal issues, de-bonding and post tension passing through the layers should be thoroughly studied. Again, guidelines for the casting and tensioning should be drafted.

The key attributes of simulation credibility are evidence of completeness and correctness, which must be communicated in an understandable and straightforward manner. In conjunction with simulation governance, the technical processes to build and assess correctness in simulations are verification, validation and uncertainty quantification.

This issue should be closely monitored with the structural monitoring means, to cast light in areas that will be critical in the actual TOD behavior.

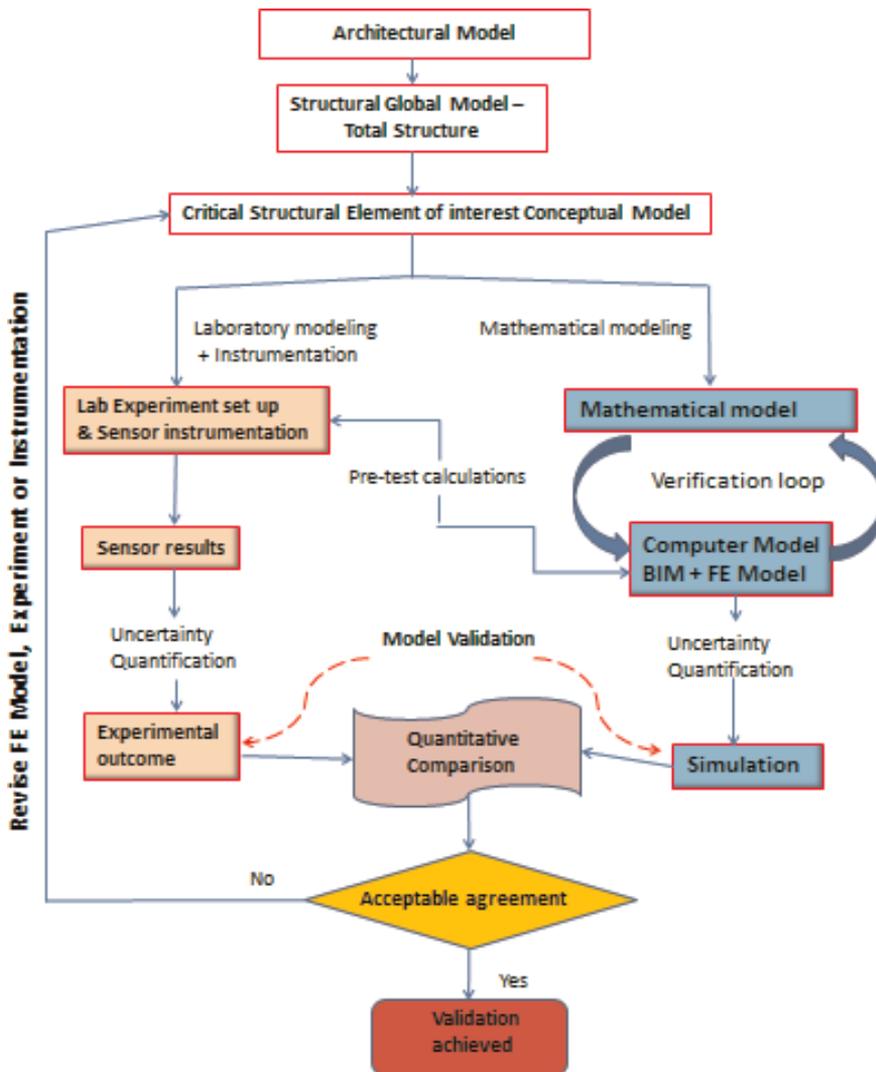
The scope in this case of monitoring is not only the ultimate capacity, but also potential deterioration/change with time, as this would inevitably lead to magnified effects on the rest of the building or even the QR assets (stations-tunnels). Therefore and because of the massive concrete character of the transfer system, a well-studied instrumentation strategy needs to take place. The performance of the instrumentation depends on the position of the sensors, the type of the sensors, the attachment method and many other factors. Fruitful results will lead to decisions about complementarity of the sensing systems, redundancy, accuracy of local measurement and overall structural response.

The simulation results are thus compared with sub-scale systems or portions of the complete system, such as subsystems and components of the total system. Formulation and approximation errors are commonly intertwined with uncertainties in the input data for the mathematical model. If for example, initial conditions, boundary conditions, or system parameters cannot be measured independently, then uncertainties in these quantities are entwined with model formulation and approximation errors. Therefore, model calibration becomes necessary. As a result, model parameter calibration and model validation involve an iterative process for cases where experimental data and data from structural monitoring are available. A potential procedural flow chart for Experimental and numerical simulation cross validation is shown in **Figure 31**.

There is one more thing however to consider regarding the VVUQ. This has to do with the method of the structural analysis itself. In order to utilize the VVUQ character described in the previous paragraph, it may be found practical to perform types of analysis resembling to Failure Mode and Effects Analysis, or even Fault tree analyses. This has to do of course with the critical structural members, the load paths utilizing them, and potential alternative load paths. A good paradigm of such approach is offered in Refs. [37, 38].

Ordinary structural analysis as ruled by international standards, (like the Euro Norms—EN1992 for example), does not categorize the structural members depending on their importance. It does not categorize or classifies potential failures, as the aim is to avoid either.

However, some structural elements are apparently more critical, or their integrity may be vital for the whole structure. Obvious examples in the case of the metro-TOD compound are the rail-track system and the transfer slab/girder respectively. But the complexity of the compound structural system pronounces the criticality of other structural systems as well. For example, a long span post tensioned beam is far more critical and should obtain a higher degree of structural importance than an isolated beam at a part of the structural system with large structural redundancy. Or equally, and to put it closer to the FMEA, a potential failure of a structural member that leads to progressive collapse is more important and deserves higher severity ranking than a potential failure of a structural member that will cause moment redistribution achievable by a robust and highly redundant structural system. The predominant failure modes of the transfer system of the TOD should be examined thoroughly from the perspective of progressive collapse, activation of alternative load paths, limitation-containment of collapse/failure and especially, away from the Station box and tunnel and at least away from the rail-track system. Obviously, the



**Figure 31.**  
 Flow chart for cross validation of experimental and numerical simulation.

hierarchy of structural assets leads to prioritization of failure mode analysis and effects analysis.

In a potential FMEA there should not be a limitation only to controls for the typical Ultimate and Serviceability Limit states. Durability deterioration should be included in such investigations as well. Potential introduction of importance factors attributed to structural members should take into account the risk factors for damage or failure: Occurrence, Severity and Detectability.

## 5. Conclusions

This chapter is an effort to provide a glimpse of the multi-faceted design efforts for analysis and design of transit-oriented developments over existing metro structures. It has been pointed out that the existing metro stations and tunnels are very sensitive structures with very tight limitations for the displacement and stress envelopes induced by the TOD. The Structural interventions needed to

accommodate the transit-oriented development must be considered following a comprehensive structural assessment of the existing structures. The effort needed to model properly the combined TOD-metro structure is very significant. Careful procedures should be adopted for the modeling of the boundary conditions of the two distinct parts and of all critical structural members. Special Finite Element—numerical treatments should be employed to approximate massive concrete. The importance of the Transfer system is highlighted, along with the potential detrimental factors that may affect its performance and that need special attention. The combined TOD-existing metro structure has got a long assumptions log which makes inevitable the structural monitoring and active corrosion monitoring and protection. The structural monitoring should be also considered as a vital tool for the validation of the design and construction assumptions and procedures.

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# Innovative Concepts in TBM Tunnels

*Silvino Pompeu-Santos*

## Abstract

Tunnel boring machine (TBM) tunnels are increasingly used in the construction of transport infrastructure, allowing for reduction of the environmental impact and cost and time of construction. Despite these advantages, TBM tunnels still face major challenges such as further cost reduction, the structural safety under earthquakes, and the improvement of safety during operation in the case of traffic tunnels (rail and road tunnels). To overcome these challenges, three innovative and very cost-effective concepts for the construction of TBM tunnels were recently developed by the author: the tunnel of improved seismic behavior (TISB) concept for improving structural safety of tunnels on soft ground in seismic areas and the tunnel multi-floor (TMF) and tunnel multi-gallery (TMG) concepts for road and rail tunnels, respectively, which allow an even greater cost reduction and improvement of safety in operation. In this paper these concepts are presented as well as their application in some specific cases, emphasizing the obtained added value.

**Keywords:** tunnels, TBM, innovations, TISB, TMF, TMG

## 1. Introduction

Tunnels are increasingly used in the construction of traffic infrastructure for both rail and road networks.

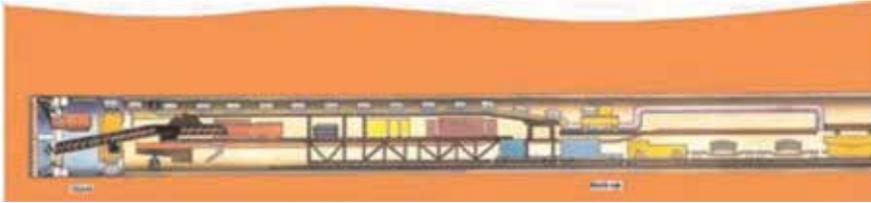
The “tunnel boring machine” (TBM) technique is nowadays the most common, allowing for reduction of the environmental impact (in underwater tunnels allows for non-disturbance of the natural bed) and significant savings in costs and time. It has become usual to build more than 0.4 km of a TBM tunnel per month on average, depending on the specific ground conditions.

Despite great progress observed in recent times, the construction of traffic (rail and road) tunnels with the TBM technique still faces significant challenges.

Big issues are the improvement of the structural safety of the tunnels when built in soft ground in seismic areas, the decision on the number of separate tubes to form the tunnel with cost analysis, and the measures to provide safe evacuation of users in the event of an accident or fire inside the tunnel.

## 2. TBM tunnels

In the construction of tunnels bored with TBMs, the circular cutter head of the front shield of the machine excavates the ground as the erector mounts precast



**Figure 1.**  
*Schematic view of a TBM.*

segments around the excavated surface, which are clamped together, forming the circular wall (lining) of the tunnel (**Figure 1**). TBMs are of different types, depending on the characteristics of the ground to be bored (EPB, mix-shield, hard rock, etc.) [1].

The precast segments are made of high-strength concrete (C40 or higher), reinforced with steel bars or fibers (in steel or synthetic). The number of precast segments in each ring will be appropriate to form complete circles with pieces of a given weight, according to the capacity of the handling equipment; commonly medium size tunnels have 6–8 segments per circle. The thickness of the precast segments will depend on the surrounding acting stresses and on the thrust forces applied by the TBM; in common situations it corresponds to about 1/25 of the internal diameter of the tunnel.

After the execution of the tunnel wall, a fill is installed at the bottom of the tunnel, creating a platform for the circulation of the vehicles: trains in the case of rail tunnels or cars and trucks in the case of road tunnels.

### 3. Challenges

As is known, the TBM technique is suitable for the construction of tunnels in stiff ground (rock, stiff clay, compacted sand, etc.), since the tunnels built in this way have their stability ensured by the resistance of the surrounding ground (the precast segments mainly work as a finish), so they do not need to have significant resistance in both the transverse direction and the direction of the tunnel axis.

In the case of soft soil (mud, soft clay, loose sand, etc.), the TBM tunnels can be unreliable because, as the connections between precast segments are very weak (it is a kind of LEGO), the strength and the ductility of the tunnels are low, so there is a risk of sinking, even collapsing, particularly during earthquakes. Soil treatments, sometimes used to aid seismic behavior, are very expensive and often do not guarantee the reliability required for the tunnels.

Regarding the number of tubes, in the case of rail tunnels in order to meet the international safety requirements [2, 3], the installation of both directions of traffic placed side by side in the same tube is only possible in short tunnels.

In long rail tunnels (tunnels over 1 km in length), for safety reasons [2, 3], two separate tunnels are usually built, each for a direction of traffic, and a complex system of cross-passages connecting the two tubes. In the event of an accident or fire inside the tunnel, users will leave the affected train and move to the other rail gallery to be rescued later by another train. In some situations systems of safety galleries and shafts are also built for local access of the emergency personnel.

As an example, the Gotthard Base tunnel (recently built in Switzerland), with 57 km in length, the longest in the world, is formed of two tubes [4]. Cross-passages regularly spaced along the tunnel and access galleries to outside in some locations were adopted (**Figure 2**).

Where it is not possible to build access galleries or shafts (e.g., in underwater tunnels), three tubes are usually adopted. This is the case of the Channel tunnel (between the UK and France) or the basic layout of the Gibraltar Strait tunnel, where two tubes are used for rail traffic and a third tube, placed between the two, is used for access to emergency services and rescue of users, using special vehicles with wheels (**Figure 2**) [4]. The three tubes are also interconnected by a number of cross-passages, regularly spaced.

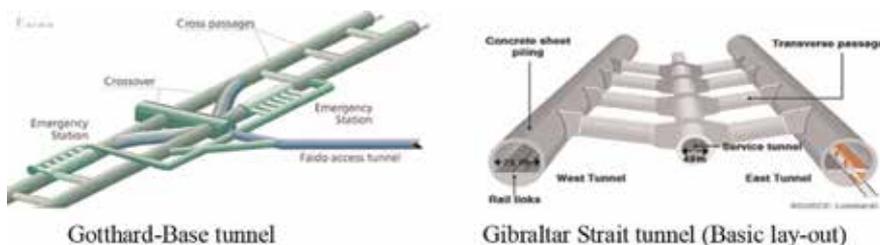
Also in the case of road tunnels, the installation of the two traffic directions in the same tube is only possible in tunnels with a single lane in each direction. When there are two or more lanes in each direction, the required diameter of the tunnel would become so large that it would be impractical. In any case, in long road tunnels (longer than 0.5 km in length), in order to satisfy safety requirements [5], placing bidirectional traffic side by side in the same tube is quite problematic. Hence, two separate tubes, each one for a direction of traffic, are nowadays usually built, so that, for ventilation and smoke removal purposes, air will circulate in one direction, the direction of traffic.

Generally, the two tubes are interconnected by cross-passages, regularly spaced, so that in the event of an accident or fire, users will leave the incident tube to the other, from where they will be evacuated by conventional buses, such the Westerschelde tunnel, in the south of the Netherlands (**Figure 3**) [6].

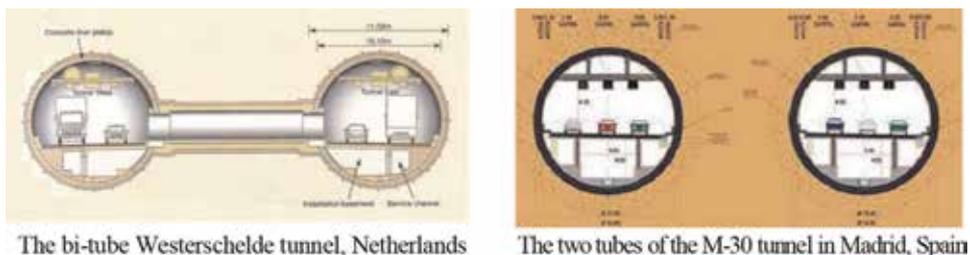
Where possible, instead of cross-passages, access galleries and evacuation routes are built along the tunnel, to allow local access and evacuation of users from the tunnel. This is the case of the tunnel of the south bypass of the M30 motorway, in Madrid, Spain, in which two large diameter tubes were adopted (**Figure 3**) [6].

Building of two (or three) tubes and the systems of cross-passages or access galleries makes the construction of the tunnels very expensive. In addition, although such layouts represent the most advanced solutions at present for rail and road tunnels, the long time necessary for rescue services to reach the scene may be too long, as has been seen in the recent past.

In order to overcome the abovementioned limitations, the tunnel of improved seismic behavior (TISB) concept for TBM tunnels on soft ground in seismic areas



**Figure 2.**  
 Common layouts of rail tunnels.



**Figure 3.**  
 Common layouts of road tunnels.

and the tunnel multi-gallery (TMG) and tunnel multi-floor (TMF) concepts for TBM rail and road tunnels, respectively, were recently developed.

## 4. The TISB, TMG, and TMF concepts

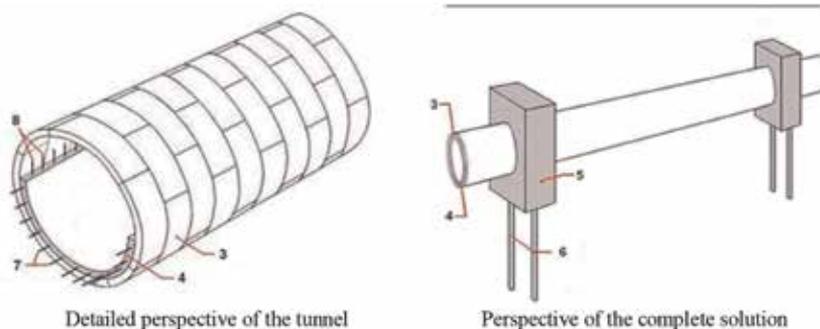
### 4.1 The TISB concept

The “tunnel of improved seismic behavior” concept is an innovative solution for TBM tunnels, when the referred tunnels are built in soft ground (e.g., mud) in seismic areas, allowing the tunnel to be provided with the adequate resistance and ductility. It also allows the strengthening of existing TBM tunnels, using them as external formwork for the execution of the internal strengthening [7]. The TISB concept is a Portuguese patent [8] and is illustrated in **Figure 4**.

In the TISB concept, the tunnel is formed by two concentric tubes; an external tube (3), which is a conventional TBM tunnel, and an internal tube (4), which is subsequently executed, inside the external one. The external tube (3) is thus formed by precast segments mounted by the TBM, while the internal tube (4) is later cast inside the external tube (3), using the latter as exterior formwork. Within the thickness of the internal tube (4), longitudinal reinforcement bars (7) and transverse reinforcement bars (8) are laid, both in two layers, which are confined by confinement bars, so as to provide the tunnel with adequate strength and ductility.

Where the vertical actions in the tunnel can have a significant variation (e.g., due to the increase or decrease in the height of the overburden in underwater tunnels), the tunnel will be provided with supports, regularly spaced along the tunnel axis. Those supports are composed of groups of piles with great horizontal deformability and ductility, arranged in the longitudinal and transverse directions, which are anchored at the top in large blocks of jet grouting (5) surrounding the outer tube (3) and at the base in the stiff ground below, so to resist vertical loads, while allowing horizontal movements of the tunnel during earthquakes, functioning as a kind of “movable bearings.”

The TISB concept thus leads to the obtaining of monolithic structures (there are no joints) with appropriate resistance in both longitudinal and transverse directions and great ductility under earthquakes. It will also be very effective if liquefaction and cyclic mobility phenomena occur. In addition, the structures obtained will present great structural redundancy, which can be useful in the case of unforeseen scenarios during the design phase.



**Figure 4.**  
*Illustration of the TISB concept.*

## 4.2 The TMG concept

The “tunnel multi-gallery” concept allows, with a single TBM tunnel, the creation of rail tunnels with completely independent directions of traffic and the installation of appropriate means that provide a dedicated and very reliable system for local access of the emergency personnel and the evacuation of users, in the event of an accident or fire inside the tunnel. The TMG concept is a Portuguese patent [9] and is illustrated in **Figure 5**.

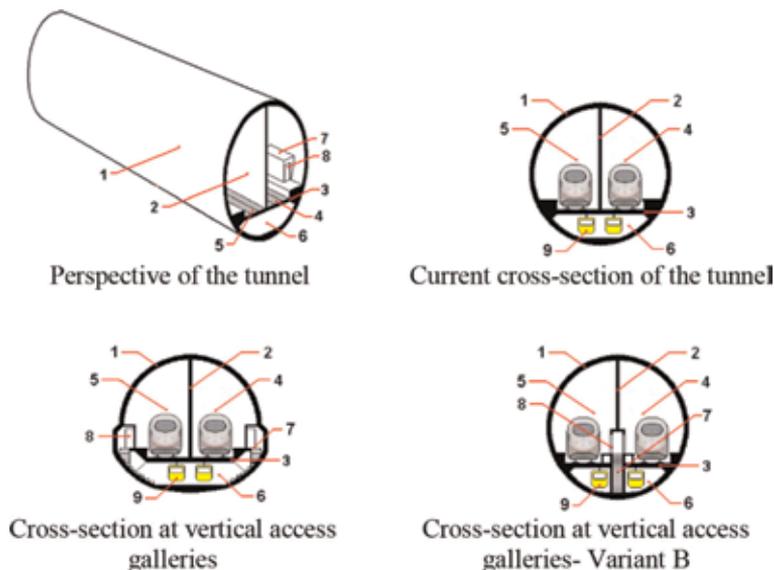
In the TMG concept, the tunnel is constituted by the external wall (1) made by the TBM, a slab (3), placed slightly above the bottom of the tunnel and the entire width, and a separating wall (2), placed in the middle of the tunnel and its entire height, so as to form two independent rail galleries, disposed side by side (4) (5), one for each track, and a service gallery (6) below.

In both sides of the tunnel, vertical access galleries (7), regularly spaced and provided with escape doors (8) in both rail galleries, are also created, allowing for the safe passage of people to the service gallery (6), in the event of an accident or fire inside the tunnel. Inside the service gallery (6), emergency vehicles (9) of monorail type are installed, to provide local access to the emergency personnel and the evacuation of users to outside.

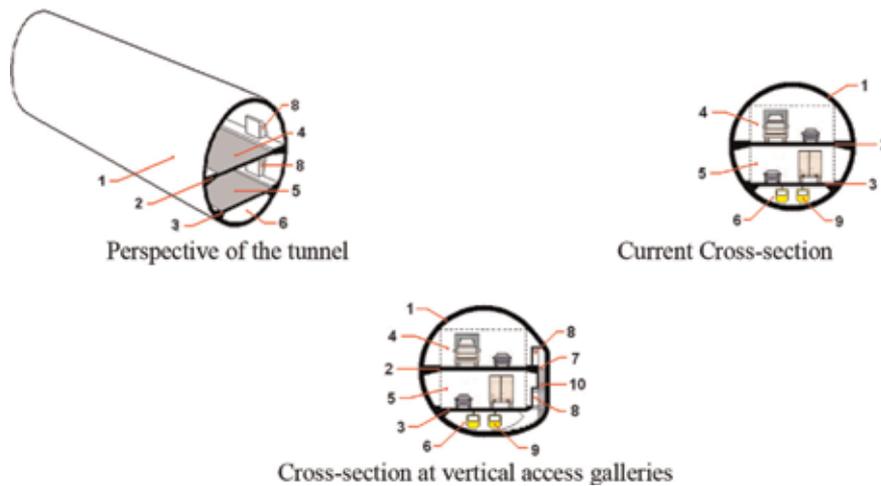
A variant B of the basic solution may also be adopted, in which the vertical access galleries (7) are placed in the middle of the tunnel, in the separating wall (Figure 4). Although there is a local slight reduction of the cross-section of the rail galleries, it avoids the need to make openings in the external wall of the tunnel.

## 4.3 The TMF concept

The “tunnel multi-floor” concept allows with a single TBM tunnel the creation of road tunnels with two identical road galleries, isolated and independent, and the installation of appropriate means that provide a dedicated and very reliable system for local access of the emergency personnel and the evacuation of users, in the event of an accident or fire inside the tunnel. The TMF concept is a Portuguese patent [10] and a European patent [11] and is illustrated in **Figure 6**.



**Figure 5.**  
*Illustration of the TMG concept.*



**Figure 6.**  
*Illustration of the TMF concept.*

In the TMF concept, the tunnel is constituted by the external wall (1) made by the TBM and two slabs (2) (3), built at its full width, one placed roughly at half the height of the tunnel and the other placed slightly over the bottom of the tunnel, so as to form two superimposed two road galleries (4) (5), one for each direction of traffic, and a service gallery (6) below.

In one of the sides of the tunnel, vertical access galleries (7), regularly spaced and provided with escape doors (8) in both road galleries, are also created, allowing for the safe passage of people to the service gallery (6), in the event of an accident or fire inside the tunnel. Inside the service gallery (6), emergency vehicles (9) of monorail type are installed, to provide local access to the emergency personnel and the evacuation of users to outside.

#### 4.4 Specific matters

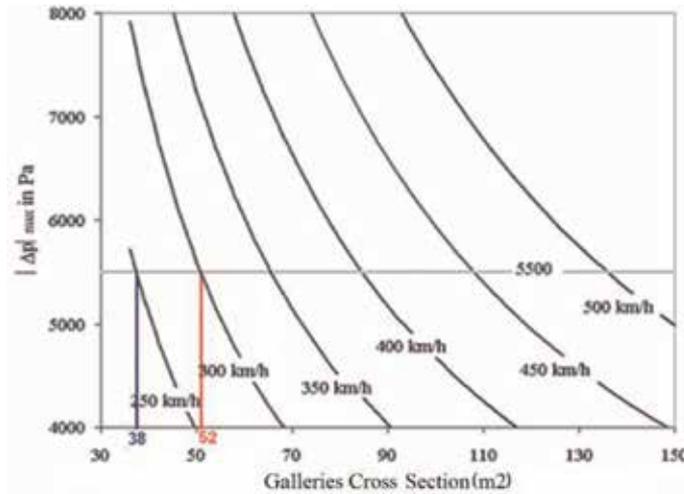
The application of these new concepts in the construction of traffic tunnels raises specific issues that require the adoption of appropriate measures.

**Tunnel cross-section.** Regarding the rail tunnels, the internal diameter will depend basically on the speed of the trains and the permissible pressure variation inside the trains. **Figure 7** shows relationships between the cross-section of the rail galleries of bi-tube tunnels and the permissible pressure variation inside trains of  $12.4 \text{ m}^2$  of the front area (a common value in high-speed trains), for different speeds of the trains [12].

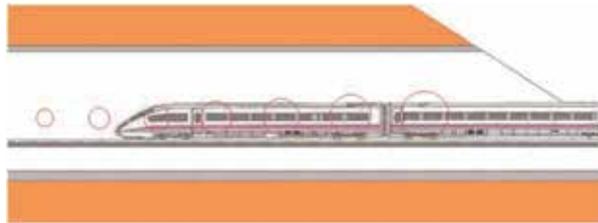
Admitting as acceptable a pressure variation of  $5.5 \text{ kPa}$  (appropriate value since there is no clash of the piston effect of the trains), for example, for speeds of  $300$  and of  $250 \text{ km/h}$ , cross-sections of  $52$  and of  $38 \text{ m}^2$  will be required, respectively, in each railway gallery.

However, as this effect is only sensitive in the portal zones of the tunnel (at the entrance and exit of the trains), it can be overcome by adopting special arrangements in those zones, namely, creating openings in the separating wall, whose area decreases from the outside to the inside, acting as pressure relief (**Figure 8**), which allows a significant reduction ( $10\text{--}15\%$ ) of the cross-sectional area of the railway galleries [13].

When variant B of the TMG concept is used, the placing of the vertical access galleries in the middle of the tunnel will cause slight localized decrease of the



**Figure 7.**  
*Relationships between the cross-section of the rail galleries of the tunnels and the speed of the trains.*



**Figure 8.**  
*Openings in the separating wall at the portal zones of the tunnels.*

cross-sectional area of the railway galleries on those areas and thus an increase in the pressure variation inside the trains. However, as the vertical access galleries are inside the tunnel, outside the portal areas, this has no influence on the comfort inside the trains.

Regarding the road tunnels, the internal diameter will depend essentially on the number of lanes in each traffic gallery and their width and the permitted height of the vehicles. In Europe, where the height of the vehicles is limited to 4.0 m, a minimum clearance of 4.8 m will be, in general, adopted.

**Execution of vertical access galleries.** When the vertical access galleries are placed on the external wall of the tunnel, they will be built by locally dismantling the precast segments mounted on those areas and casting new concrete walls in situ.

In situations where there is water pressure around the tunnel, injections of cement grout will allow for the development of the works in safe conditions. In those situations special steel segments will be mounted on those areas, provided with holes to allow for the execution of the injections.

Although there are some risks, they are similar to those of execution the cross-passages in twin-tube tunnels.

**Firefighting.** The traffic galleries of the tunnels will be equipped with active detection and attack devices, acting jointly, instead of relying on conventional systems of attack by fire trucks. Heat sensors and smoke detection systems automatically activate high-pressure water mist nozzle systems, regularly distributed along the tunnel and grouped in sections, lowering the temperature on site. After this action, firefighters (who come through the service gallery) can then extinguish the fire.

**Rescue of users.** In the event of an accident or fire inside one of the traffic galleries of the tunnel, users will leave that gallery through the emergency walkways to the nearest escape door, from where they reach the service gallery below, down the stairs inside the vertical access galleries. Inclined platform lifts running along the stairs provide access to handicapped people.

Emergency vehicles of the “emergency monorail electric vehicle” (EMEV) type that circulate inside the service gallery will rescue the users to outside of the tunnel.

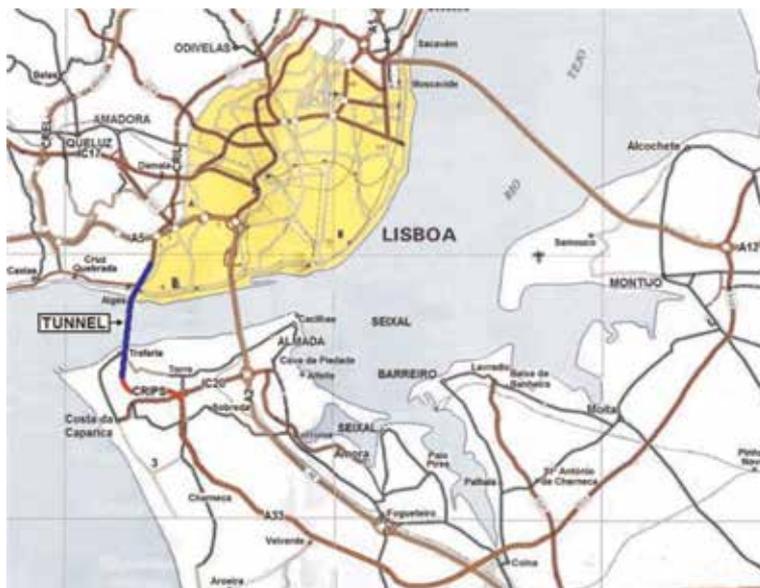
**Emergency vehicles.** The EMEVs are autonomous vehicles that receive wireless signals from the tunnel control center. They are battery powered, so as not to be dependent on the reliability of the electricity network inside the tunnel. They circulate suspended from the ceiling slab, in general, in two parallel lines. They are grouped in “trains,” in numbers according to the needs. They will be parked at one or both of the tunnel portals, where users will have outbound exits.

## 5. Application of the TISB and TMF concepts on a proposal for a road tunnel crossing the Tagus River in Lisbon

### 5.1 Introduction

The Algés-Trafaria road tunnel, crossing the Tagus River in Lisbon, Portugal, aims the decongestion of the road traffic of the suspension bridge, which is currently 50% higher than its capacity. It will allow the closing of the inner ring motorway of Lisbon, constituted by the CRIL in the north bank, the Vasco da Gama Bridge at east, and the CRIPS (A33) in the south bank of the river. It will be located west of the suspension bridge (**Figure 9**).

The location of the tunnel is characterized by the existence of thick alluvial deposits along the riverbed, composed of various complexes of mud and sand, extending from elevation  $-29$  m (the deepest level of the river) to elevation  $-75$  m. Underlying the alluvial deposits, there are bedrock formations constituted by basalt and limestone that extend by the north bank. On the south bank, there are Miocene



**Figure 9.**  
*Location of the Algés-Trafaria tunnel crossing.*

formations composed mainly of sand and clay. The very prone-seismic conditions of the area must also be noted (one must remember the 1755 Lisbon earthquake, one of the most destructive in history).

Preparations for the construction of the new crossing have been under way for a long time, and an immersed tunnel solution has already been studied by Lusoponte, the concessionaire of the Tagus road crossings in Lisbon [14]. However, for various reasons, no significant progress has been made.

The study concluded that the construction of the immersed tunnel is viable but presents significant risks, associated with the high probability of liquefaction of the sands that constitutes the riverbed, in the event of a strong earthquake, and also with the difficulties of the realization of the tunnel connections at its ends, which will significantly increase the overall cost of the project. The cost of the tunnel (at current prices) was estimated at 600 million euros.

A TBM tunnel solution for the crossing, based on TISB and TMF concepts, was, in the meantime, proposed by the author [15–20].

## 5.2 The proposed TBM tunnel solution

In the proposed solution, the tunnel will be a single-tube tunnel, 5.1 km in length with two superimposed road galleries and a service gallery at the base. At the deepest part, the bottom of the tunnel is at elevation  $-59$  m, allowing for a soil overburden identical to the diameter of the tunnel. The tunnel runs through the alluvia along most of its length under the river (**Figure 10**).

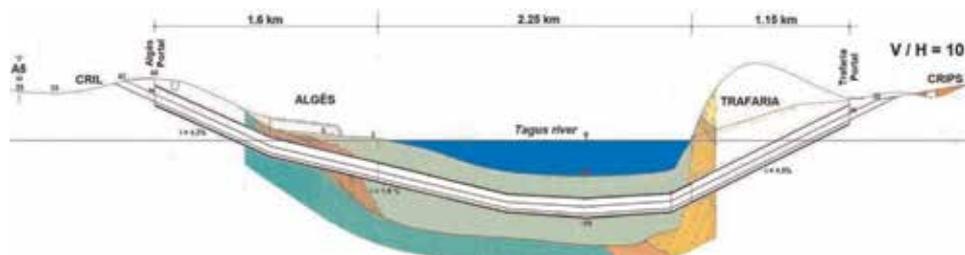
The tunnel has an internal diameter of 14.2 m, with precast segments 0.55 m thick ( $D_i/25$ ) and 0.15 m clearance between the lining and the ground to be injected with grouting; hence, the excavated diameter of the tunnel is 15.6 m.

Despite the large diameter of the tunnel, it is significantly smaller than that of the larger tunnels being built, such as the SR99 in Seattle (USA), with a 17.5 m diameter, and the Tuen Mun-Chek Lap in Hong Kong (CN), with a 17.6 m diameter.

Inside the tunnel two concrete slabs are built, in order to create two superimposed road galleries (each one for a direction of traffic) with two lanes each (3.5 m wide and 4.8 m high); outer emergency lane; inner edge and emergency walkways on both sides, with a total width of 12.6 m; and a service gallery at the bottom, 2.0 m high (**Figure 11**).

To provide the tunnel with adequate structural safety under earthquakes, the section under the riverbed (2.25 km long) will be strengthened with an internal reinforced concrete tube 0.3 m thick, dully confined, in order to improve its strength and ductility, which are essential in the case of liquefaction of the sand (**Figure 11**).

The upper slab is supported laterally on continuous corbels executed in the precast segments or cast jointly with the internal tube (where it exists). The lower

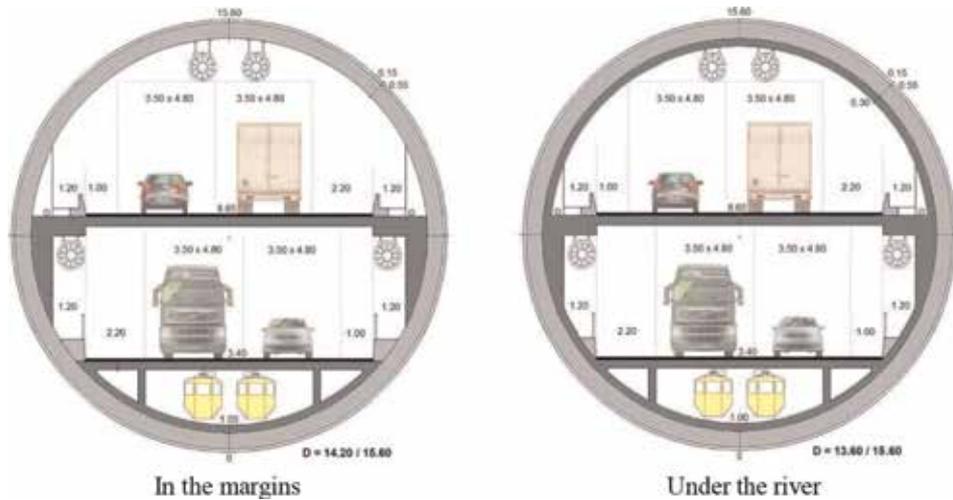


**Figure 10.**  
*Proposed TBM tunnel solution for the Algés-Trafaria crossing. Longitudinal section.*

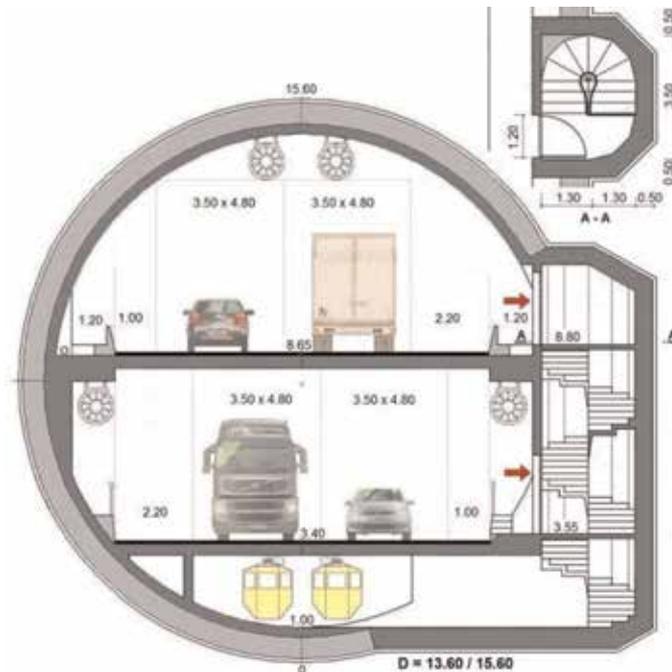
slab is supported on two small longitudinal concrete walls cast in situ and on the TBM tube.

The road galleries have escape doors located on one side of the tunnel, spaced 400 m (less than the 500 m allowed by the EU rules) [5], which give access to the service gallery below, through vertical access galleries with  $2.6 \times 3.5$  m of inner section (Figure 12).

Inside the service gallery, emergency vehicles of the EMEV type circulate to allow for the access of emergency personnel to inside the tunnel and the evacuation of users out of the tunnel in the event of accident or fire.



**Figure 11.** Proposed TBM tunnel solution for the Algés-Trafaria crossing. Current cross-section.



**Figure 12.** Proposed TBM tunnel solution for the Algés-Trafaria crossing. Cross-section at vertical access galleries.

**Environmental impact.** Once the tunnel is bored by a TBM, its construction will not cause any disturbance of the riverbed along the tunnel axis. Traffic on the Tagus River and on the port of Lisbon will not be disturbed either. The connections of the ends of the tunnel to the existing road network, on both sides, present no particular difficulties.

**Safety in operation.** The tunnel is provided with an advanced safety concept, which represents a step forward in the safety of road tunnels.

In the event of an accident or fire in one of the road galleries, users will leave the incident gallery by walking through the respective emergency walkways to the nearest escape door, from where they reach the service gallery, down the vertical access galleries.

Inside the service gallery, dedicated EMEVs that circulate in two parallel lines give access to emergency personnel and evacuate the users out of the tunnel. They are grouped in pairs, being parked at the portals of the tunnel. In such situations there will be no disturbance of the traffic flow in the non-incident traffic gallery.

**Cost.** The cost of the tunnel was estimated by considering a tunnel unit cost per cubic meter of excavation with a value appropriate to the tunnel layout and site characteristics. In view of these conditions, it is appropriate to adopt for this tunnel a unit cost of 420 euros per cubic meter of excavation [19, 20].

With an excavation diameter of 15.6 m, which corresponds to an area of the excavated section of 191 m<sup>2</sup>, the unit cost of the tunnel will be about 80 million euros per kilometer. As the tunnel length is 5.1 km, the cost of the tunnel will be about 410 million euros, significantly lower than the cost of an immersed tunnel, 600 million euros, as mentioned above [19, 20].

### 5.3 Conclusions

The application of the TISB and TMF concepts allows to obtain a very cost-effective solution for the construction of the Algés-Trafaria tunnel crossing the Tagus River in Lisbon, with very low environmental impact, improved structural safety under earthquakes, and low construction costs (significantly lower than that of an immersed tunnel), and provides an advanced safety concept for emergency personnel access and rescue of users in the event of an accident or fire inside the tunnel.

## 6. Application of the TMF and TMG concepts on an optimized TBM tunnel alternative for the Fehmarnbelt Fixed Link

### 6.1 Introduction

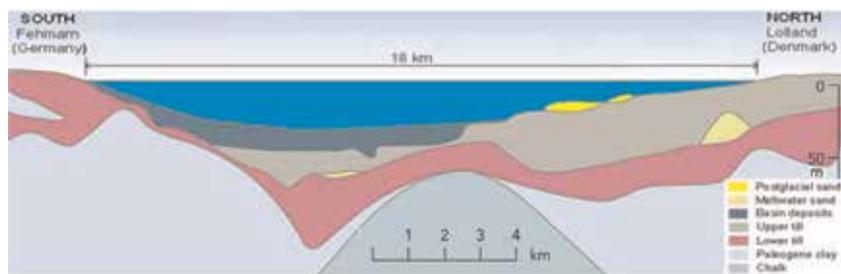
The Fehmarnbelt Fixed Link is a Danish-German project, in the Baltic Sea, 18 km long, to provide a direct link by rail and road between the two countries and Scandinavia (**Figure 13**). It is part of the expansion of the Trans-European Transport Network (TEN-T) of the European Union, being co-financed by EU funds. The project will be owned and financed by Denmark and to be repaid by the users. It is being managed by Femern A/S, a Danish state-owned company.

The project aims to connect the Lolland island (in Denmark) and the Fehmarn island (in Germany), through the Fehmarn Belt. It will be for mixed traffic, with two road galleries provided with two lanes each and two rail galleries for trains at speeds up to 200 km/h, keeping the pressure variation inside the trains within acceptable limits [21].

The geological profile along the alignment of the tunnel is shown in **Figure 14** [21]. Both sides present smooth slopes near the coast areas, the deepest water being 34 m.



**Figure 13.**  
Location of the Fehmarnbelt fixed link.



**Figure 14.**  
Geotechnical conditions of the site.

Under the seabed the soil comprises an upper quaternary layer of post and late glacial deposits (clay and silts) followed by a Paleogene layer of highly plastic clay. The German side is characterized by Paleogene clay and some clay-till, the central basin by sand silts and clays, while the Danish side is dominated by thick deposits of clay-till.

Studies for this project began more than 20 years ago, in the 1990s. It has been studied in several variants, starting with a suspension bridge, followed by a cable-stayed bridge. As both bridge solutions received much opposition, especially from environmental organizations, an immersed tunnel solution was also further studied.

Although the costs of the cable-stayed bridge and the immersed tunnel were broadly similar, in 2011 the Danish authorities took a preliminary decision to adopt an immersed tunnel in the link. A TBM tunnel solution was also at the time studied by the promoter but, being composed of three tubes (two tubes for road traffic and another for rail traffic), despite having less environmental impact than the immersed tunnel, was rejected because the estimated cost was higher [21].

The immersed tunnel solution has then been subjected to public consultation of the Environmental Impact Assessment (EIA) by the Ministry of Transport in

Denmark in 2013. Under the framework of this consultation, the author developed an optimized TBM tunnel alternative based on the TMF and TMG concepts, which proved to be much more cost-effective than the “official” TBM tunnel solution and the immersed tunnel solution [22]. However, this alternative was not accepted, and the immersed tunnel solution obtained its approval.

The EIA of the project was also submitted to public consultation in the state of Schleswig-Holstein, in Germany, in 2014, where, after a rather harsh process, approval was also recently granted. However, the future of the project in its current form is still uncertain, as several environmental organizations threatened to challenge this decision in the German courts.

## 6.2 The immersed tunnel solution

The immersed tunnel solution is a conventional immersed tunnel, consisting of a single prismatic tube approximately 18 km long, 42.2 m wide, and 8.9 m high, consisting of 89 precast concrete segments in general with 217 m in length (Figure 15) [21]. There are also cut-and-cover sections at the ends.

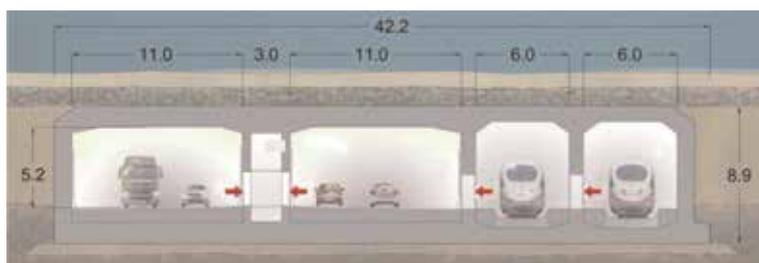
The tunnel is provided with four traffic galleries: two road galleries, 11.0 m wide and 5.2 m high, and two (ballastless) rail galleries, 6.0 m wide and 6.0 m high. It also includes a service gallery, placed between the two road galleries, 2.0 m wide, for the installation of pipes and cables and to be used as temporary refuge although not allowing to be used by vehicles.

The railway galleries are provided with emergency walkways on both sides, 1.3 m wide, while the road galleries have an emergency lane on the outside but have neither internal edge nor emergency walkways.

The precast segments are placed in a trench dredged in the seabed, on a bedding layer of crushed rock. A combination of locking gravel fill and sand fill is then backfilled along the sides of the elements, while a protection layer of stones is placed across the top of the elements. Part of the dredged material is placed over the protection layer.

The execution of the works presents significant risks, since they are developed at the surface of the open sea, in a zone of intense ship traffic, and using precast segments which are significantly larger than those used in prior projects.

**Environmental impact.** As generally recognized, the environmental impact of the immersed tunnel solution is very significant. Among others, the large area of natural seabed of the German Natura 2000 site that will be disturbed by the construction works is worth noting, a width of over 100 m along the entire tunnel length. The huge volume of excess dredged material that will have to be placed in reclamation areas (14.8 million cubic meters) is also noted [22]. Also its significant “footprint” is impressive, with the following quantities of the most representative



**Figure 15.**  
*Immersed tunnel solution. Current cross-section.*

materials used: concrete, 3.0 million cubic meters; rock, 3.1 million cubic meters; and sand, 5.1 million cubic meters [22].

**Safety in operation.** In the case of fire or accident, the rescue of users relies on conventional vehicles that will use the road galleries, to which they access through escape doors, spaced 110 m [21]. However, despite this low distance between escape doors, much lower than the 500 m required by the EU rules [3, 5], as it can be shown, this does not represent a significant added value for the safety of the users of the tunnel [27].

On the contrary, the safety concept of the tunnel presents several significant shortcomings [27], namely, (a) the road galleries have no emergency walkways, so, during escape, those from behind will tend to push the others ahead into the traffic lanes; (b) with the arrival of dozens, perhaps hundreds, of people at a roadway gallery, fleeing an incident gallery, there is the risk of disruption of the traffic flow in this gallery, preventing the arrival of rescue vehicles; (c) rescue of the passengers of a train (full of 600 people) will need at least a dozen buses, which can take several hours to have them at the scene; and (d) the traffic flow in the non-incident galleries will be significantly disturbed by the occurrence of any safety problem in one of the galleries of the tunnel.

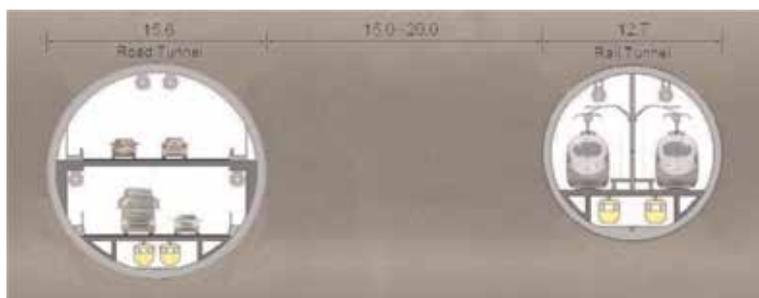
A very serious question is how to escape from the outer railway gallery. Passengers will have to cross the railway line of the internal rail gallery to reach the adjacent road gallery, which will be very dangerous and therefore should not be acceptable (see **Figure 15**) [27].

**Cost.** The cost of the tunnel was estimated by the owner at 5500 million euros [21], which is identical to the tenders in the meantime received for the construction. Given that the project was granted with 600 million euros of EU funds, the financial effort of the promoter will thus be around 5000 million euros, to be repaid within a 36-year period.

### 6.3 The optimized TBM tunnel alternative

Based on TMG and TMF concepts, an optimized TBM tunnel alternative was developed [22–27] by the author. It consists of two separate tunnels, one for road traffic and the other for rail traffic (**Figure 16**), placed beside one another at a distance of about 15–20 m, that go deep into the ground to about elevation –63 m, complemented with cut-and-cover sections at the ends.

The rail tunnel is about 20 km long and has an inner diameter of 11.5 m, with precast segments 0.45 m thick (Di/25) and 0.15 m clearance between the lining and the ground, to be injected; hence, the excavated diameter of the tunnel is 12.7 m, a common size for TBM tunnels. An intermediate slab and a central wall are then constructed, creating two parallel, independent, and isolated rail galleries with



**Figure 16.** Optimized TBM tunnel alternative. Current cross-section.

38 m<sup>2</sup> cross-sectional area, each for a direction of traffic, and a service gallery below, 2.2 m high (**Figure 17**).

The variant B of the TMG concept is used; thus, the emergency walkways of the rail galleries, 1.4 m wide (wider than those of the immersed tunnel solution), are placed on the inner side.

On both emergency walkways, there are escape doors spaced 400 m (less than the required by the EU rules [3]) that have access to vertical access galleries placed in the middle of the tunnel, at the separating wall (**Figure 17**).

Although the placement of the vertical access galleries in the middle of the tunnel causes a slight local decrease in the cross-sectional area of the railway galleries, since the vertical access galleries are outside the portal zones, the comfort conditions within the trains will not be affected.

The road tunnel is about 19 km long and has an inner diameter of 14.2 m, with precast segments 0.55 m thick (Di/25) and 0.15 m clearance between the lining and the ground to be injected; hence, the excavated diameter is 15.6 m (**Figure 18**), the same as of the largest TBM tunnels in operation, although (as referred above) larger TBM tunnels are still being built, such as the SR99 in Seattle (USA), 17.5 m in diameter, and the Tuen Mun-Chek Lap in Hong Kong (CN), 17.6 m in diameter.

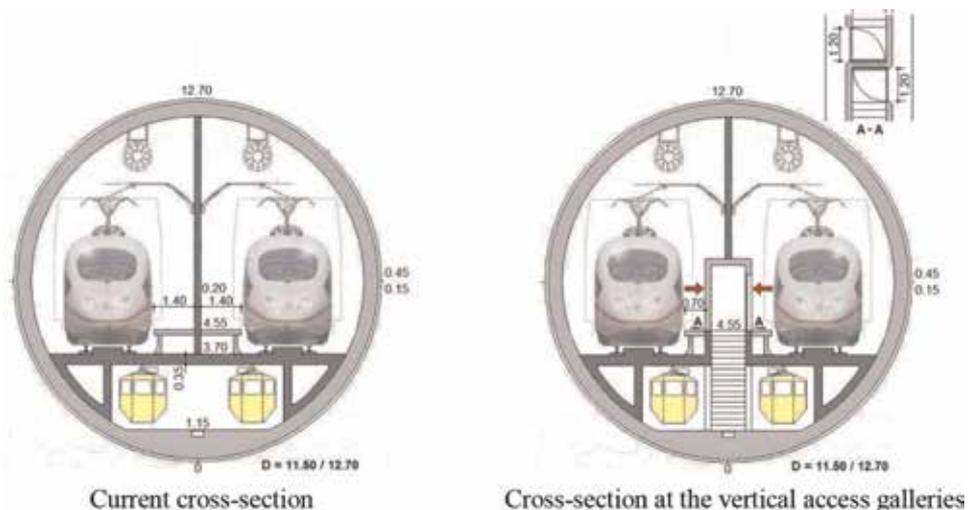
Inside the precast tunnel, two intermediate slabs are constructed, creating two superimposed road galleries, independent and isolated, each one for a direction of traffic, 5.0 m high, and a service gallery below, 2.0 m high.

Both roadway galleries have two lanes of 3.5 m wide each, external emergency lane 2.2 m wide, inner edge 1.0 m wide, and emergency walkways on both sides 1.2 m wide, over a total width of 12.6 m, greater than those of the immersed tunnel (11.0 m).

Laterally to the emergency walkways in one of the sides of the tunnel, there are escape doors spaced 400 m (less than the required by the EU rules [5]) that have access to vertical access galleries, allowing the safe passage of people between the road galleries and the service gallery (**Figure 18**).

**Environmental impact.** Being formed of bored tunnels, the optimized TBM tunnel alternative will not provoke any disturbance of the natural seabed along the tunnel alignment.

The volume of excavated material that will have to be placed in the reclamation areas is about 6.2 million cubic meters, much smaller than the volume of the



**Figure 17.**  
Optimized TBM tunnel alternative. Cross-section of the railway tunnel.

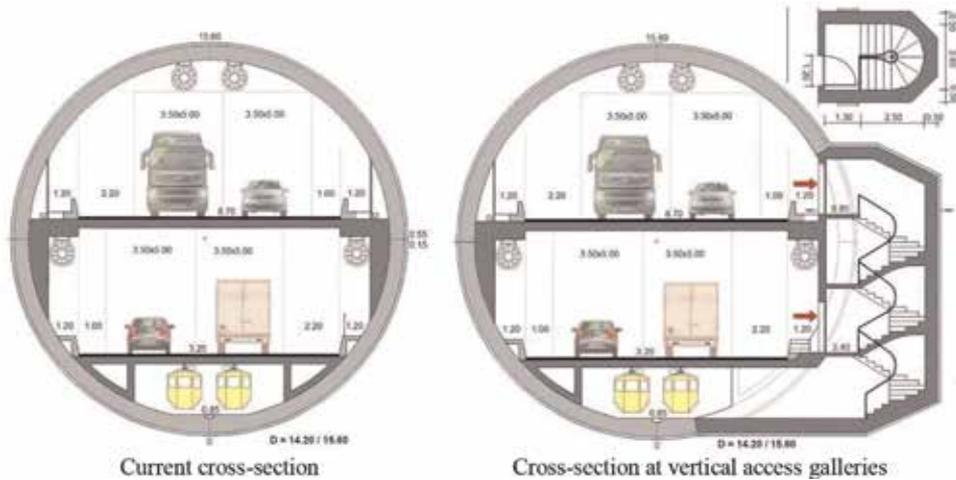
dredged material that will have to be placed in the case of the immersed tunnel solution (14.8 million cubic meters, as mentioned). The spending of natural resources in the main building materials (footprint) is as follows: concrete, 1.9 million cubic meters, and rock and sand, non-significant, which is also much smaller than in the case of the immersed tunnel solution [22–27].

**Safety in operation.** The TMG and TMF concepts provide to the optimized TBM tunnel alternative advanced emergency systems, which are a great step forward in the safety of traffic tunnels [22–27].

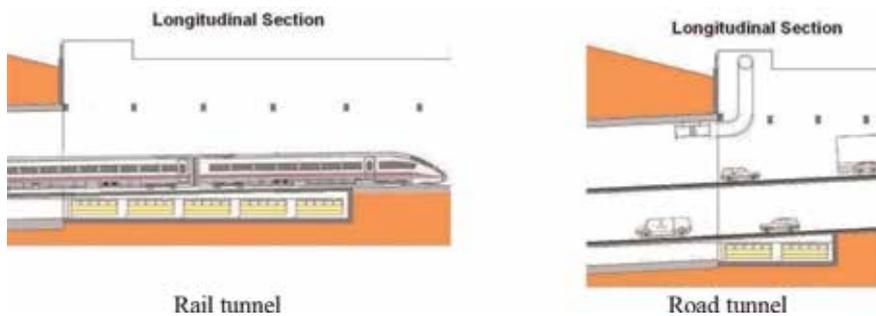
In the event of accident or fire in one of the traffic galleries of the tunnels, users will leave that gallery by walking through the respective emergency walkway to the nearest escape door, from which they achieve the service gallery of the tunnel, down the stairs of the vertical access galleries.

Inside the service gallery, dedicated emergency vehicles of the EMEV type that circulate in two parallel lines will allow for the access of emergency personnel and the evacuation of users out of the tunnel. They are grouped in “trains” (of five in the rail tunnel and two in the road tunnel), being parked at both portals of the tunnels (Figure 19). In such situations there will be no disturbance of the traffic flow in the non-incident gallery.

**Cost.** The cost of the optimized TBM tunnel alternative was estimated on the basis of the estimated cost of the “official” TBM tunnel solution, considering appropriate unit costs for each tube.



**Figure 18.**  
Optimized TBM tunnel alternative. Cross-section of the road tunnel.



**Figure 19.**  
Optimized TBM tunnel alternative. Portals of the tunnels.

The “official” TBM tunnel solution consists of three tubes [21]: two road tunnels with internal diameter of 14.2 m, corresponding to excavated diameters of about 15.6 m, and a rail tunnel with 15.2 m of internal diameter, corresponding to an excavated diameter of about 16.6 m, which leads to excavated volumes of about 3.82 million cubic meters for each road tunnel and 4.32 million cubic meters for the rail tunnel.

Whereas the estimated cost of this solution was 6800 million euros, and considering that the unit cost of the rail tunnels is about 70% the unit costs of the road tunnels [25–27], unit costs of 450 euros per cubic meter for the rail tunnel and 635 euros per cubic meter for the road tunnel are obtained.

Hence, for the optimized TBM tunnel alternative, unit costs of 450 euros per cubic meter for the rail tunnel (the same as the one obtained for the “official” TBM tunnel solution) and 650 euros per cubic meter for the road tunnel (slightly higher than the one obtained for the “official” TBM tunnel solution, since there are the vertical access galleries to build) are assumed.

Whereas in this case, the excavated volumes are about 2.54 million cubic meters for the rail tunnel and 3.63 million cubic meters for the road tunnel, the estimated costs will be 1150 and 2400 million euros for the rail and the road tunnel, respectively. Therefore, the estimated cost of the TBM tunnel alternative is 3550 million euros, which is less than 2/3 the cost of the immersed tunnel solution [25–27].

It should also be noted that the unit costs obtained for the “official” TBM tunnel solution are significantly higher than those normally obtained in tunnels under similar conditions, so it has to be admitted that the estimated cost is exaggerated by at least 15%.

Thus, the cost of the optimized TBM tunnel alternative will probably be around 3100 million euros [27], so the financial effort of the promoter would be about half of that of the immersed tunnel solution.

## **6.4 Conclusions**

Given the above considerations, the following main conclusions are drawn.

With regard to the environmental impact, while in the immersed tunnel proposal it is very significant, in the Optimized TBM tunnel alternative, it is very low; in particular it avoids any disturbance of the natural seabed along the tunnel.

Regarding the safety in operation, while the immersed tunnel solution has several weaknesses in its safety concept, the optimized TBM tunnel alternative presents an advanced safety concept, in which the rescue of users relies on dedicated unmanned electric vehicles operating inside a service gallery, so completely independent of the access conditions inside the traffic galleries.

With regard to costs, the cost of the optimized TBM tunnel alternative is about 3100–3550 million euros, which is less than 2/3 the cost of the immersed tunnel solution, and the financial effort of the promoter would be halved, allowing for an equivalent reduction in the tolls to be paid by the users or in the repayment period.

In summary, the optimized TBM tunnel alternative is undoubtedly much more cost-effective than the immersed tunnel solution.

## **7. Final remarks**

The TISB, TMG, and TMF concepts are innovative developments that represent a step forward in the construction of rail and road tunnels executed with the TBM technique.

In addition to the intrinsic environmental advantages of TBM tunnels, they provide improved seismic behavior, reduction in the construction costs, and improvement of safety during operation.

Regarding the seismic behavior of the TBM tunnels built on soft ground, the TISB concept provides the tunnels with the necessary strength and ductility, avoiding the need for additional soil treatments.

Furthermore, although being formed by single tubes, each tunnel accommodates two completely independent and isolated galleries of traffic (for rail or road) and a service gallery below, which allows a very reliable safety concept, much more reliable than any currently existing.

With regard to costs, simply comparing the excavated volumes of the referred single-tube tunnels with those of the equivalent conventional solutions using two parallel tubes connected by cross-passages shows that reductions of more than 20% will be easily achieved.

With regard to safety in operation, the service gallery at the base of the tunnel provides a very reliable pathway for access of the emergency personnel and the rescue of users in the event of accident or fire inside the tunnel, through dedicated emergency vehicles (EMEVs), therefore independent of the availability of conventional rescue vehicles or the access conditions inside the traffic galleries of the tunnel.

In summary, the TISB, TMG, and TMF concepts provide very cost-effective and safe solutions that can be of great value in the construction of the rail and road tunnels of tomorrow, especially long tunnels.

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# Design of Immersed Tunnel and How We Research Submerged Floating Tunnel

*Wei Lin, Ming Lin, Haiqing Yin and Xiaodong Liu*

## Abstract

This chapter begins with the discussion of the immersed tunnel design, concerning its reason of existence, historical review, general design, transverse and longitudinal design, the interaction, and the critical issues. The discussion is founded on the author's 10 year experience in building the Hong Kong-Zhuhai-Macao Bridge (HZMB) immersed tunnel as a site design engineer. The experience of building immersed tunnel is transferable to build the submerged floating tunnel, which has never been built. In author's opinion, the submerged floating tunnel (SFT) technique will be the next generation of IMT technique. In the second part of this chapter, the author proceeds to discuss the strategy of SFT research and the latest development in CCCC SFT Technical Joint Research Team.

**Keywords:** immersed tunnel, submerged floating tunnel, design, research, civil engineering

## 1. Introduction

Immersed tunnelling is an art of guiding the great natural force, the water, to do engineering works: “guiding” buoyancy for transportation, “guiding” water weights for immersion, and “guiding” hydrostatic pressure for connection. Submerged floating tunnel (SFT) is an even more extreme form of this art, as the full weight of tunnel or most of it is balanced by buoyancy. This chapter discusses the method of immersed tunnel design and SFT research.

## 2. Design of immersed tunnel

### 2.1 Reason of existence

“In order successfully to conceive and to plan a structure or building of any kind it is necessary to investigate and to know well its reasons for existence ...” is the first line of the book *Philosophy of Structures* written by Eduardo Torroja. A city that has water barriers but has no bridge is like a mansion with no elevator, answered Strauss, the chief engineer of the Golden Gate Bridge. However, a bridge could have

its limitations: its span could disturb the ship traffic, and its tower could disturb the air flight if the bridge were built close to an airport.

When a bridge crosses a harbour, the water salinity in the harbour may change due to the slowed water exchange between offshore sea and the fresh inland water, as the bridge piers disturb the water exchange, giving impact to the living condition of sea resident in the harbour region. In Øresund tunnel compensate dredging was performed to eliminate the said effect. In Hong Kong-Zhuhai-Macao Bridge project, the two offshore artificial islands, which connect immersed tunnel and bridges, have a minimum length so that the total water blockage ratio of the entire link is controlled minimum. The water blockage ratio was defined as the projected area of the link that disturbs the water exchange along the axis of the link divided by that of the total area of the water.

The above is the reason of existence for a subaqueous tunnel. Whether to build a bored tunnel or immersed tunnel varies and depends on specific project condition. One commonly seen reason to choose immersed tunnel is more cost-effective because the immersed tunnel is buried shallower than bored tunnel; the latter requires typically a buried depth not less than 1–1.5 times of the bored diameter for construction. In the island and tunnel project of Hong Kong-Zhuhai-Macao Bridge (HZMB Island-Tunnel Project), there were two main reasons for choosing immersed tunnel. Firstly, as both ends of the immersed tunnel connect to bridges via two artificial islands (**Figure 1**), the length of the artificial islands would be twice smaller, leading to smaller water blockage ratio. Comparatively, the bored tunnel would be buried deeper than that of the immersed tunnel due to its longer transition length. Secondly, the geology risk such as encountering boulder for bored tunnel is high, and thus the risk of time delay for the entire 55 km long link is high.

Despite the advancing of technology, the understanding about the marine environment is still limited; the risk of construction of an immersed tunnel in the offshore condition is relatively high. “Stories” of sinking, flooding, and damage exist such as [1, 2]. Therefore, one aim of the design of an immersed tunnel is to find a way to mitigate the construction risk by proposing the appropriate scheme and technical requirement.



**Figure 1.**  
*Overview of HZMB link from Hong Kong side.*

## 2.2 History and state of the art

The earliest attempt was in 1810. The British Engineer Charles Wyatt won the competition by proposing the immersed tunnelling concept, using brick-made cylinders, each around 15.2 m long, and sinking them to a dredged river bed, and then backfilling them. A test was carried out with much care by another British engineer with two specimens, each 7.6 m in length and 2.74 m in diameter. The test results are positive. However, the cost was overrun, the project terminated. It was not until 1893 when three sewer pipelines (diameter of which is only 1.8 m) were made by this construction method. The first traffic immersed tunnel was built in 1910 [1].

The immersed tunnel is, as per the defined term of ITA WG11, a tunnel consisting of several prefabricated tunnel elements, which are floated to the site, installed one by one, and connected under water. **Figure 2** shows the working image of the HZMB Island-Tunnel Project. This tunnel consists of 33 tunnel elements and a closure joint (**Figure 3**). The immersion had been completed on May 5, 2017. Since then it becomes the longest roadway immersed tunnel. This record will soon be broken by the Fehmarn tunnel, which will be around 18 km long and consist of 89 tunnel elements.

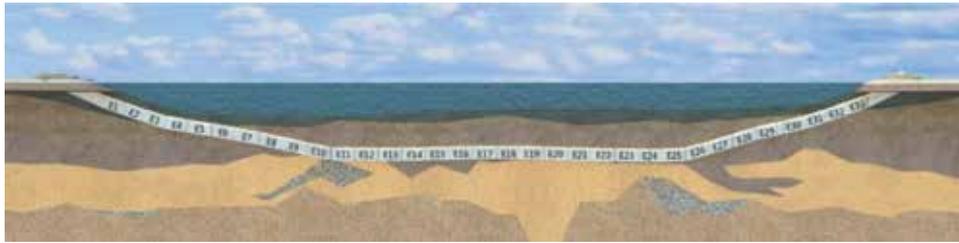
## 2.3 General design

The environment acting on the immersed tunnel depends on the location of the tunnel. Thus, the tunnel alignment needs to be fixed in the first place.

The plane alignment of the tunnel mainly depends on its two ends, the access point of the tunnel portal. For the vertical alignment, that is, the elevation of the tunnel, several considerations need to be taken. The elevation of the tunnel ends shall neither be too high nor too low. If the ends of the tunnel were too high, the immersion depth is inadequate for the hydraulic connection of the first tunnel



**Figure 2.**  
*Construction works of the immersed tunnel in HZMB Island-tunnel project.*



**Figure 3.**  
Vertical alignment of the immersed tunnel consists of 33 tunnel elements in the HZMB Island-tunnel project.

element that connects to the land structure. If the ends of the tunnel were too low, the risk of flooding increases as more massive amount of water could rush into the tunnel due to rain or overtopping. Moreover, the elevation of the middle section of the tunnel depends on the navigational requirement of ships passing over the tunnel. After the elevation of tunnel ends, the section under the navigation channel was fixed. The remained work is to “draw a line” for the vertical alignment of the tunnel. The principle as an experience by the pioneer is to dive down or rise up as quickly as possible. In this way, the tunnel length will always be the shortest, as can be proven in **Figure 4**.

As long as the tunnel alignment is fixed and the environment that will be encountered by the tunnel is thus fixed, the actions such as wind, wave, current, and water depth can be defined as well. In short, structural design can be done.

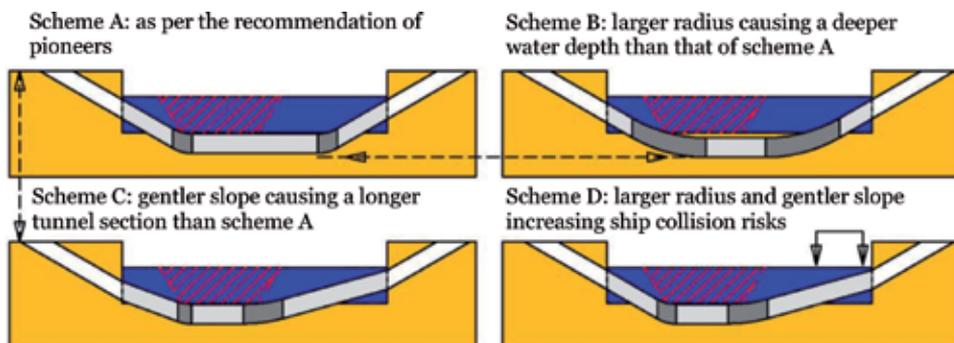
The immersed tunnel consists of one or several tunnel elements. Therefore, the design of the immersed tunnel is, in fact, the design of tunnel elements. The design of each tunnel element usually distinguishes from each other. One reason is that each tunnel element exists in a more or less different environment and the actions on them are different. The other reasons can be seen in **Figure 5**, as an example from HZMB Island-Tunnel Project.

As for the design of each tunnel element, the problem can be further discretized into several subproblems, as will be elaborated in Sections 2.4 and 2.5.

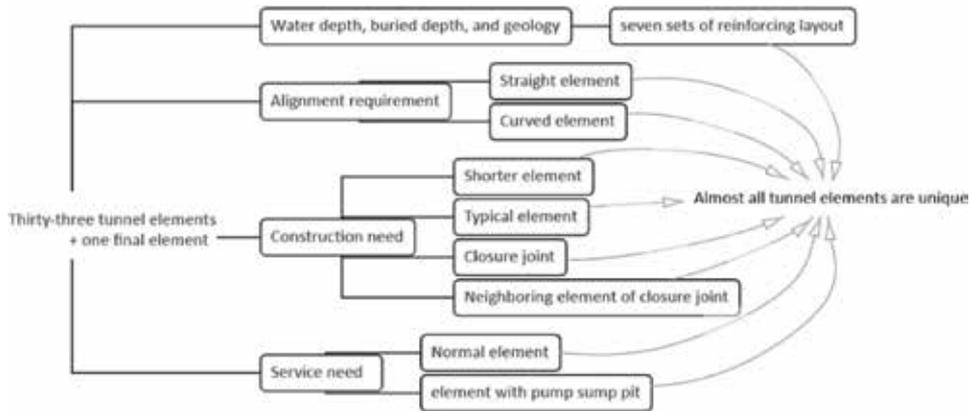
## 2.4 Transverse structure

The transverse design needs to satisfy three aspects: the structural issue, the weight balance, and the interior space.

The structural issue is a familiar subject to structural engineers. Not only the permanent scenarios but also the temporary scenarios of construction shall be



**Figure 4.**  
Illustrative image showing the principle of drawing vertical alignment.



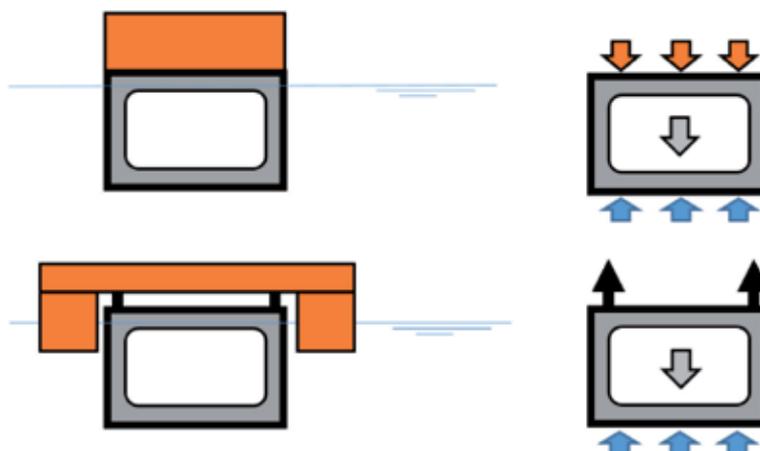
**Figure 5.**  
 Uniqueness of immersed tunnel element design in HZMB Island-Tunnel Project.

considered regarding the boundary condition and load and actions, because the permanent condition of the immersed tunnel may not govern the design as it would do for many other types of structures.

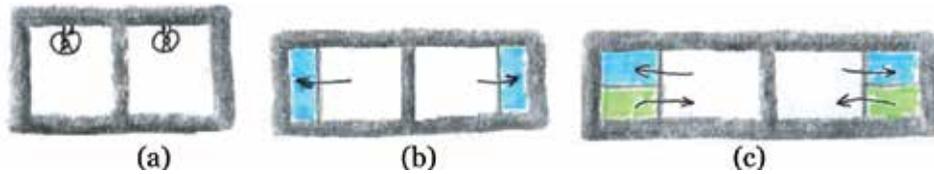
The weight balance means that the tunnel element can float when being transported and can sink for immersion and underwater connection; those are construction need. Further, in the service period, attention shall be paid to ensuring adequate safety factor against uplift, in case of extreme weather conditions. To note, the construction rigs may affect the freeboard of tunnel element when it was afloat (**Figure 6**).

The interior space requirement depends on the traffic clearance (i.e. the minimum space requirement for traffic defined by the relevant regulations/code), the space for interior installations such as ventilator and fireproof panels, and the extra space for accommodating construction tolerances from the prefabrication and immersion of tunnel element.

With the increased awareness of comfort design and life safety, more attention is paid to ventilation and evacuations, in addition to the above said three basic needs. **Figure 7** shows three ventilation solutions, namely, the longitudinal ventilation, semi-transverse ventilation, and transverse ventilation. The longitudinal ventilation



**Figure 6.**  
 Immersion rigs of pontoon sit on tunnel element and reduce the freeboard of the tunnel element while catamaran increases it.



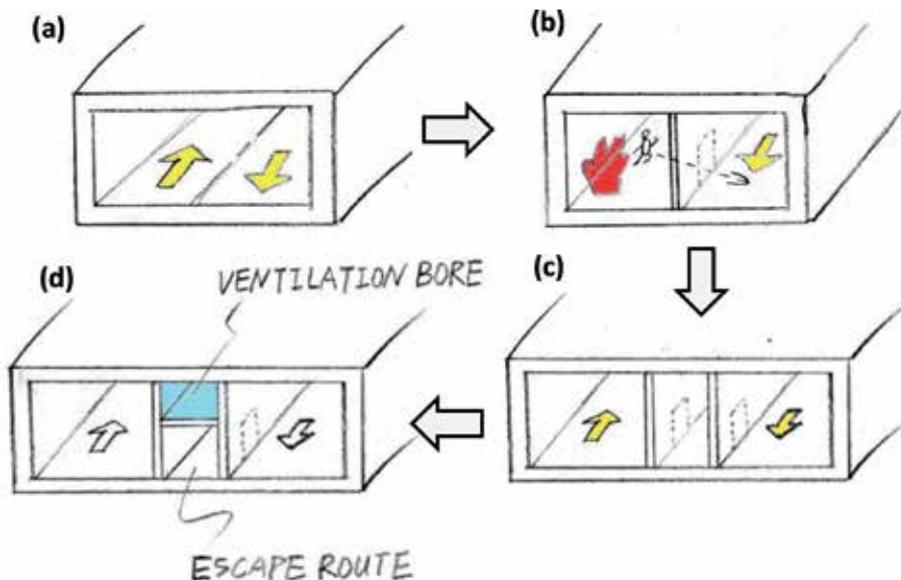
**Figure 7.**  
Ventilation solutions of immersed tunnel: (a) longitudinal, (b) semi-transverse, and (c) transverse.

requires fans that increase the height of the tunnel, leading to a deeper foundation and more dredging works. The transverse ventilation requires special bores and thus increases the width of cross-section, also leading to more dredging volume. The semi-transverse ventilation was somewhere in between. Concerning the setting of the inner walls in the cross-section, **Figure 8** shows its relation with the safety concept. Also, purely from a structural point of view, the more inner walls, the less governing the largest span of the cross-section structure. In the 1990s Japan tunnel favoured a cross-section of two tubes with two galleries, and the double walls gave benefit to both the robustness of watersealing and the safety of the structure.

## 2.5 Longitudinal structure

The longitudinal design needs to consider three aspects as well, namely, the structural system, element length, and joint configurations.

The earlier immersed tunnel had monolithic tunnel element. The cross-section is circular shaped; the structure type is steel shell. To increase the space use from 1937 to 1942, the first reinforced concrete box structure tunnel element was made. Around 10 years later, the segmented-type tunnel element made of reinforced concrete was developed in the Netherlands. In Øresund tunnel, factory method was



**Figure 8.**  
Safety and transverse design: (a) two-way traffic in one bore, collision risk is relatively high; (b) two traffic bores separated by a wall, evacuated people could be hit by vehicle on the other bore; (c) two traffic bores separated by two walls; (d) the centre part is further split into two bores, one for evacuation and the other for smoke extraction.

invented to produce tunnel element of 55,000 tons in a production line [3]. That method was used in the HZMB Island-Tunnel Project (**Figure 9**) for the second time; the production line was capable of incrementally launching the 76,000 t tunnel elements (in which five of them were plane-curved tunnel elements with curvature R5500) without cracking them. In around 1990 in Japan, no more place along the shore can be found to prefabricate tunnel element. Further, the experienced concrete vibration workers were not adequate. In this background, the steel-concrete-steel sandwich structure box-type immersed tunnel element was developed as its concrete requires no vibration; the pouring of concrete can be completed in the floating stage of the tunnel element.

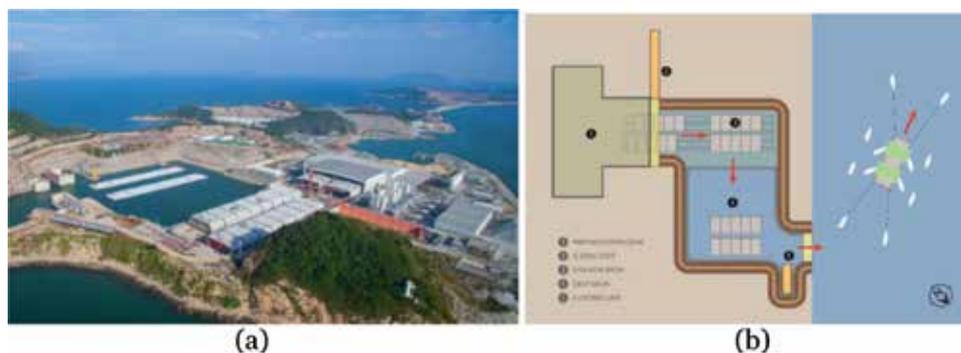
The length of the tunnel element determines the number of tunnel element, given the fixed tunnel length. On the one hand, the longer tunnel element reduces the total number of element and thus reduces the total number of immersion joint, the main works of which are bulkhead and its embedded part, Gina gasket water-stop, and so forth. Further, fewer tunnel elements mean fewer times of the immersion works and thus less risks of construction. On the other hand, the shorter is the tunnel element, the less is the total prefabrication cost as less area of land near the water is needed for prefabrication of tunnel elements, and the less sensitive of the tunnel element structure to the differential settlement issue, hence the less cost of the prestressing system. The above shows that the element length design is a matter of keeping a balance and finding the optimum.

The immersion joints need to ensure watersealing in tunnel's service period taking into account all the unfavourable scenarios such as earthquake, differential settlement, and accident like sunken ships; it also needs to provide a way of connection of tunnel element for construction. **Figure 10** shows immersion joints of a typical tunnel element in prefabrication yard.

## 2.6 Interaction

To optimise the scheme, the works described in Sections 2.4 and 2.5 may be named as “analysis”, and the subsequent work, on the contrary, as described in this section, can be named as “synthesis”, that is, understanding the links between the factors and then looking for the most satisfying design schemes by means of design iterations.

In the transverse structure of the immersed tunnel, the three aspects mentioned in Section 2.4 are interlinked. For example, strengthened structure causes thickened slab or more densely arranged reinforcing bars, either of them would give additional weight to the structure; the weight balance is broken and thus needs to be rebalanced by adjusting the inner space of the tunnel. Taking another example,



**Figure 9.** *Factory for prefabrication of immersed tunnel element. (a) Photo HZMB Island-tunnel project and (b) plane layout.*



**Figure 10.**  
*Photo of immersion joint of tunnel element in HZMB Island-tunnel project.*

enlarged inner space leads to the increased buoyancy, which requires an added weight by thickening the walls to balance the extra buoyancy.

It does the same to the longitudinal structure of the immersed tunnel element. Moreover, the longitudinal structure is interlinked with the transverse structure. Following the above example, the thickened wall allows for larger shear key, which could increase the capacity of shear key; it also means the ability of survival of tunnel element against differential settlement increases. In this way, the tunnel element can be made longer.

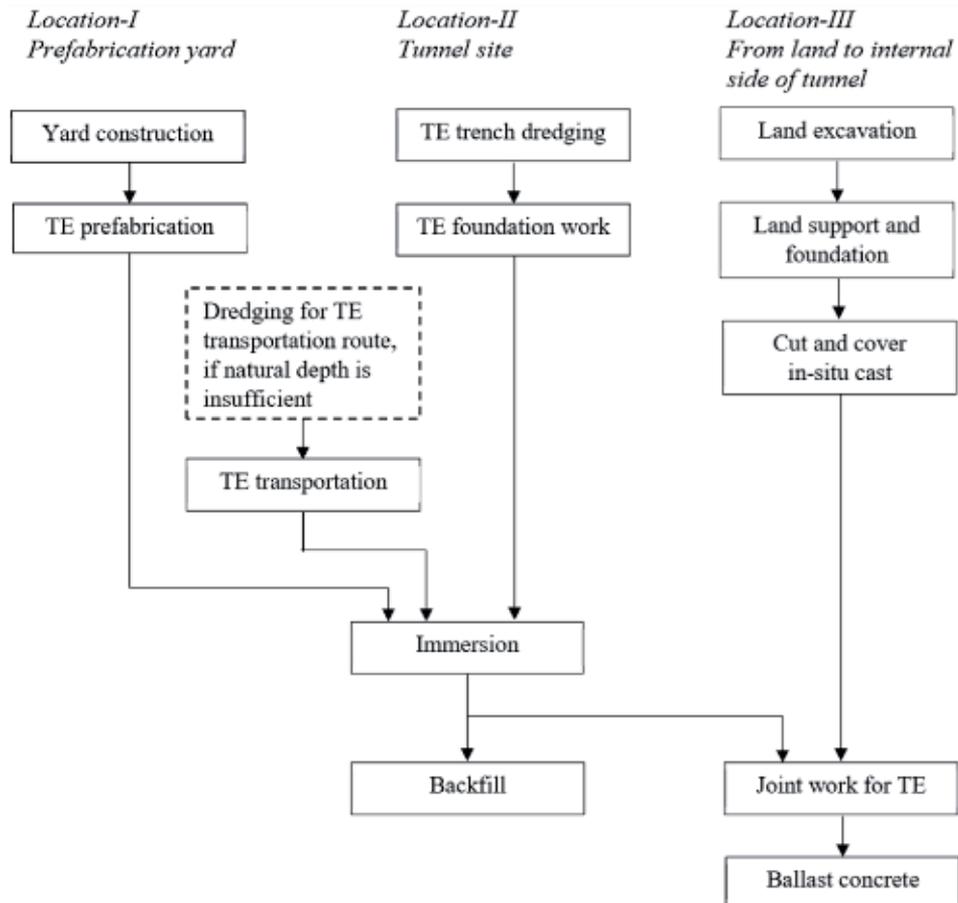
Another link is that the design of immersed tunnel is related to time and space, as shown in **Figure 11**. The prefabrication of tunnel element, the installation, and the inner works of tunnel often take place in three different locations. Moreover, to complete the work, there are sequences to follow. This figure shows that the immersed tunnel design, to some degree, cannot be reproduced; hence, the nature of the design work for an immersed tunnel is indeed to eliminate the gaps of space or discontinuity of time of the immersed tunnelling works.

## 2.7 Two fatal issues

The success or failure of an immersed tunnel project largely relies on the prefabrication yard and the water sealing of tunnel. For the former, in HZMB Island-Tunnel Project, great efforts were made to find a suitable place, six locations were investigated, and the final selected location was on an island for three advantages. First, the geological condition is hard rock, suitable for the incremental launching system of the factory method. Second, the transportation distance of the tunnel element for immersion works is shortened to only 11 km. And third, the prefabrication yard is capable of producing two tunnel elements while store six tunnel elements inside the dock, eliminating the risk of tunnel element damage from the frequent typhoon in summer time of each year.

## 2.8 The latest technological developments and the future

To the author's best knowledge, in Bosphorus Strait, the immersed tunnel had been built around 70 m below the water surface. In Busan immersed tunnel, special facilities were invented to position the tunnel element under the water accurately and to make direction correction of the tunnel element automatically.



**Figure 11.**  
 The workflow of the immersed tunnel shows the discontinuity and gaps of space and time.

In HZMB Island-Tunnel Project, 35 times of installations were carried out in 3 years in offshore condition with no major accident. The novel foundation solution of composite foundation layer and underwater surcharge were implemented, and the settlement of the immersed tunnel had been controlled within 5–8 cm. Nearly 100 million cubic metres of concrete were cast for the main structure of the 5.664 km long immersed tunnel element with no cast crack. Further, the tunnel element was deeply buried below the seabed; maximally 22 m thick sediment will cover on the roof of the tunnel element. And this extremely high overload (as for the tunnel element structural design of immersed tunnel longitudinally) is overcome by the structural innovation of semi-rigid tunnel element structure, setting permanent prestressing; the structure can become more robust taking the advantage of both monolithic and segmented tunnel element (**Figure 12**). The memory bearing can prevent concrete cracking at immersion joint [4]. The deployable element [5] is a highly effective and risk-manageable way to build the closure joint of the immersed tunnel.

The technological development has been pushing the boundary of application in immersed tunnelling regarding length and depth. However, to cross much deeper and broader water, all existing solution of bridge and tunnel would fail; in that case, the SFT shows its good reason of existence. The pioneer engineer of immersed tunnel Walter Grantz left one thought: “all immersed tunnels are briefly SFT’s while they are being lowered into position.”

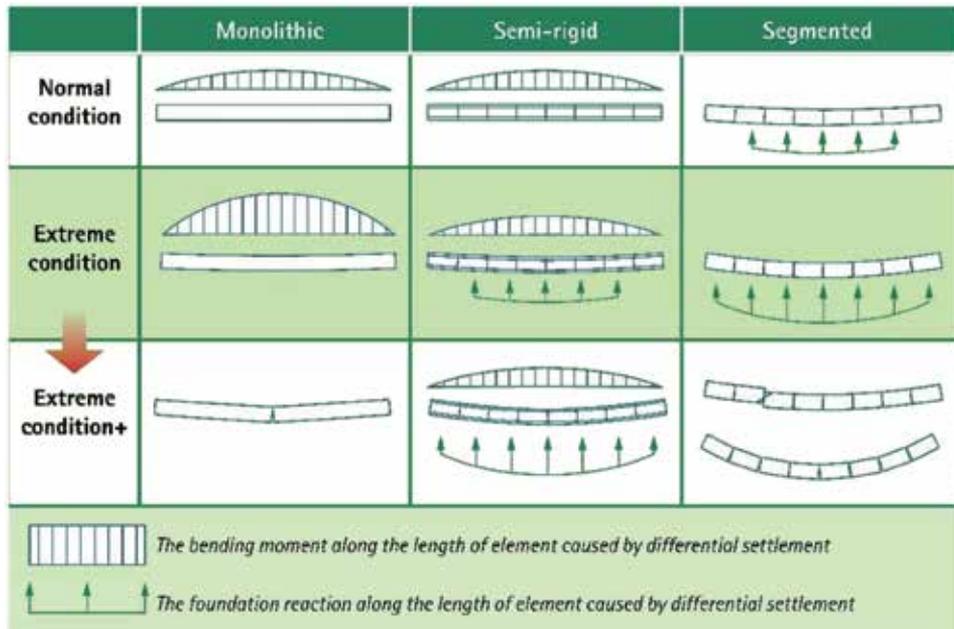


Figure 12.  
The mechanism of the novel semi-rigid tunnel element structure.

### 3. How we research submerged floating tunnel

#### 3.1 Main threads

The SFT is, as per the term defined by ITA WG11, a tunnel through water that is not in direct contact with the bed. Moreover, it may be either positively or negatively buoyant and may be suspended from the surface or supported from or tied down to the bed (**Figure 13**). It has been proposed a century ago but has never been realised due to various reasons, such as fear of invasion, fishery problem, or ship collision. Therefore, to realise SFT attention must be paid to safety. The safety is in direct connection with SFT's structural form and environment. The former can be designed and developed as per our will and, hence, should be the main threads of SFT's research. Further, the more details the SFT's structural form being developed, the more risk issues regarding it will be raised. Therefore, the risk should



Figure 13.  
Image of an SFT with positive buoyant tied down to sea bed.

be regarded as an accompanying thread in addition to the main thread of SFT's research. The study of risk can be carried out both quantitatively and qualitatively.

If the SFT is small, we can hold it in our hand; our research method is to produce hundreds of SFT prototypes, test them, improve them, and perfect them. However, our research resources are so limited compared to the real size of SFT; we can hardly build one SFT prototype, not to mention to agitate and test the real SFT. Therefore, the applicable way to research SFT is to build SFT model instead. The model can be further distinguished by tangible and intangible one, which is, in a researcher's language, the mathematical model and the physical model. The model supports the SFT's thread to gain the true knowledge of SFT.

Another support to the research is the development of the construction method. An SFT design scheme that can satisfy all needs and requirements but cannot be built in realistic work is of little value/use to the engineering knowledge. Sometimes we can hardly resist our temptation to study the details and believe that our work can improve the efficiency of the work; in fact, the parallel study on construction method can help shorten research period and lower the overall cost. Construction experiences from other relevant works such as immersed tunnel (**Figure 14**) and offshore structures may be transferable to SFT.

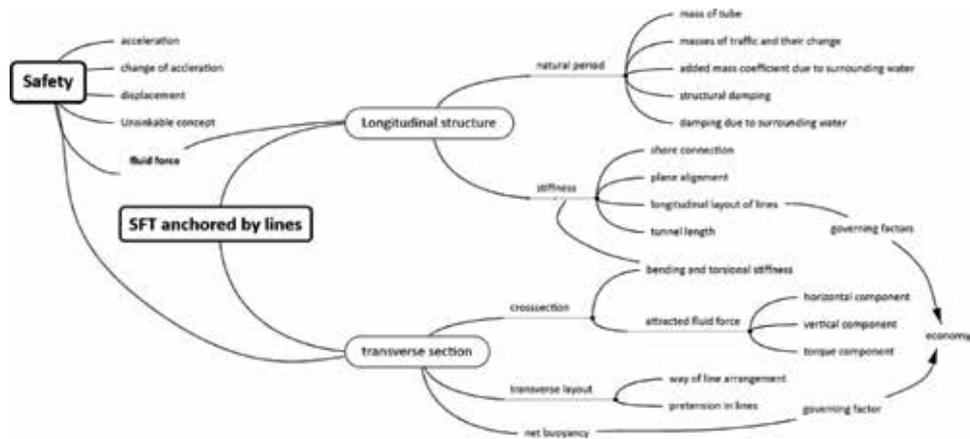
### 3.2 Structural form

In order to understand the links between the SFT structure and its risk, we need to discretize the structural form of SFT into elements, part of which needs to be further discretized into sub-elements. By changing a parameter of the element or sub-element, we can observe how the change affects the structural behaviour or safety of SFT. In this way, we gradually understand SFT's nature. **Figure 15** shows the author's understanding of the relation of structure and safety. This figure needs to be further expanded to cover the full picture of understanding of SFT before making a real one.

The safety belongs to our feeling and judgement, while the element or sub-element of an SFT is a matter; how do the two interact with each other? Mathematics is the gear of interaction. For example, if we set  $\delta$  as deflection and  $t$  as time, then  $\partial^2 \delta / \partial t^2$  stands for the deflection's second derivative to time, that is, acceleration. The deflection not only determines the member force of the structure but also affects our safety feeling if we pass through an SFT. The acceleration will make us have "seasickness", if the value of it is large due to improper design of the structure.



**Figure 14.** Image of immersion works of a typical tunnel element in Hong Kong-Zhuhai-Macao Island-tunnel project, showing immersion rigs, lines, and positioning system.



**Figure 15.**  
Relation of structural element and safety.

For another example, if we set  $\bar{T}$  as the natural vibration period of an SFT and  $T$  as periodic loading acting on it due to natural waves, then  $\bar{T}/T$  represents their ratio. In structural design, we need to avoid resonance by letting this ratio far away from the value of 1. We can either change the mass of the tube or the stiffness of the lines or introduce more damping into the system.

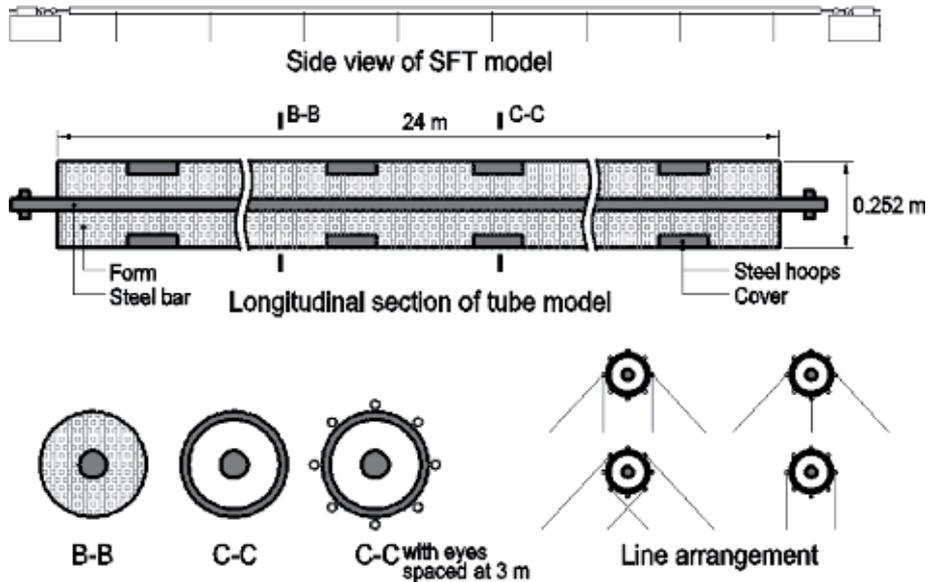
### 3.3 Model

The slenderness and the size of an SFT are like a bridge, and the submergence of it is like a ship or submarine, while the slenderness, the size, and the submergence of SFT are comparable to nothing. Moreover, no real SFT exists. Thus, no mathematical model has ever been validated. Therefore, the physical model is essential.

The ultimate purpose of building a model is not to obtain figures but to obtain the tools that can obtain the figures and to see through the figures, beyond them. A scaled (and simplified) prototype of an SFT is a physical model, and a reproduction of a physical model is a mathematical model. Once the mathematical model is calibrated and validated by the physical model, it can then be used to predict the behaviour of the prototype by scaling the model up in the computer. However, one problem in this procedure is the scale effect, which may distort our understanding and judgement. The SFT structure is submerged in the water. Thus, the scaled effect exists in both structure and fluid. In HZMB Island-Tunnel Project, comparison of the current drag force was made to the scaled physical model and the measured data; great discrepancy exists between the two [6]. In the design of the steel-concrete-steel composite structure of closure joint for the immersed tunnel for the same project [7], special measures were made to strengthen the structure since the size is larger than the previous application in Japan.

To deal with the disturbance of scale effect, one countermeasure is to subdivide the physical model test into three stages, the mechanism test, the parameter test, and the validation test. The mechanism test focuses on the understanding of the structural behaviour to guide the direction of structural form; hence, the influence of scale effect may be neglected. In the parameter test, we ensure that the scale of the model is large enough so that the results of the test can be used either directly or by extrapolation to calibrate the mathematical model. The validation test is the last-stage test for validation of the overall designs to ensure the robustness and comprehensiveness of the SFT structural system. Engineers or engineer researchers may find that the mathematical model cannot match with the physical model and that limitation can be effectively solved by asking help from applied mathematicians or physicians.

To study the overall structural behaviour in water and from current and waves, the author proposed and designed a 1:50 physical model test in 2018; the tube model is 24 m long, with a circular cross-section of 252 mm in diameter, representing a 1.2 km long, two-lane traffic road SFT. **Figure 15** shows the design of the tube model. A steel bar is in the centre of the tube for simulating bending stiffness and covered by the form that simulates the volume. Steel hoops were on the external side of the form simulating weight. Some hoops were welded with eyes for connecting to steel wire lines; the lines were connected with springs to simulate the stiffness of mooring lines. The lines are spaced at 3 m longitudinally. Both ends of the model were fixed. By setting up a reference model and altering structural parameters such as net buoyant, line arrangement (**Figure 16**), and boundary conditions, the change



**Figure 16.**  
*Schematic design drawings.*



**Figure 17.**  
*Photo of SFT overall structural behaviour test sponsored, designed, and led by CCCC; SFT model and sensors prepared by DUT; and basin built by TIWTE.*

of structural behaviour subjecting to the change of structure can be observed. This test is now being prepared (**Figure 17**) as the first case in the world; results are expected to guide the direction SFT structural form design.

#### **4. Conclusions**

The high risk of immersed tunnel construction requires a risk reduction through design. Due to varied location, environment, and construction/operation need, almost each tunnel element design varies from each other. The interior space, structural resistance, and tunnel element weight determine the transverse design of the immersed tunnel, while the structural system, element length, and joint configuration determine the longitudinal design of that. Multifactors of the structure were interlinked and linked to construction, time, and space; hence, a satisfying design requires a spiral-up iteration process including the works of analysis and synthesis. The selection of prefabrication yard for tunnel element, the transportation channel, and the water sealing of tunnel element are detrimental for the project.

SFT can cross broader and deeper waterbody. The main threads of our SFT research are structural form and risk, supported by construction method, mathematical model, and scaled physical model. We will find what is unknown, understand the mechanism, obtain parameters through physical model tests, and understand SFT's behaviour by mathematical measures. When encountering our limit, we need to cooperate with physics and mathematics.

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#### **Conflict of interest**

The author declares no conflict of interest.

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Section 4

# Tunnelling in Digital Era

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# Digital Construction Strategies and BIM in Railway Tunnelling Engineering

*Georgios Kapogiannis and Attwell Mlilo*

## Abstract

Technology has been a strong driver for industrial efficiency in the twenty-first century. Rapid growth in infrastructure projects such as tunnels is synonymous with both disruptive and supportive technologies that automate operations. The sector has rapidly risen to the challenge from buyers demanding a more digitalised experience when looking to (re)design new tunnels. Currently there are projects in the United Kingdom, Greece and Italy investing in tunnels for their transport networks to help commuters to travel quicker. We could argue that construction has evolved because the tunnels developed nowadays are expected to last for several generations but such an argument is count intuitive. Think of having to spend billions of pounds for a tunnel that does not provide an enhanced travel experience and in a few years' time requiring a major investment to remodel in order to operate it. This chapter discusses what, why and how digital construction can add value during the lifecycle of a tunnel.

**Keywords:** digital construction, building information modelling (BIM), tunnel modelling and management, asset management

## 1. Introduction

Technology has been a strong driver for industrial efficiency in the twenty-first century. Rapid growth in infrastructure projects such as tunnels is synonymous with both disruptive and supportive technologies to automate operations. The tunnelling sector has rapidly risen to the challenge from tunnel asset owners demanding more digital design solutions when procuring new tunnels. Currently there are projects in the United Kingdom, Greece and Italy investing in tunnels to improve transport capacity and help commuters to travel quicker. We could argue that construction has evolved because the asset developed in now expected to last for several generations but such argument is intuitive. Imagine having to spend billions of pounds for a tunnel that does not provide an enhanced travel experience and in a few years' time requiring a major investment to remodel in order to operate it. Construction cannot afford to remain stuck in past and must transform to improve delivery efficiency and sustainability.

The World Economic Forum (2016) has developed a transformation framework for the construction industry listing 30 measures of best practice. It highlights three important areas of transformation from its traditional approach. Firstly, it has to be

open for innovation so that opportunities from new technologies, materials and tools are exploited to reduce production costs. Secondly, it should consider adopting mechanised and automated production systems alongside offsite construction techniques to speed up the construction process and enhance timely completion of projects in a collaborative environment. However for projects of high complexity such as tunnelling that involves a number of strategic and operational decisions from the client, designers, contractors, the supply side and regulators, BIM can provide a collaborative platform. Through vertical and horizontal collaboration, processes and resources will be optimised to deliver the client's requirements. Inadvertently this generates a significant amount of data, which needs to be integrated and communicated to the stakeholders so that optioned solutions are agreed on and value is created. Digital construction is an adoption of technology driven initiatives that aim to make use of advances in Information and Communication Technologies (ICT) to enhance integration.

The integration model highlights the need to bring together people, processes, and products of the construction project to deliver value to the client in a more efficient and sustainable way. The third area pivots on the role of project management and the control of costs in the design and planning stages. When procuring projects, contracts are designed to achieve optimum risk sharing across the supply chain with agreed monitoring mechanisms. Lately, the momentum within construction has shifted from the focus on the top down approach of project delivery to more collaborative approach that seeks to satisfy clients' requirements. Results of embracing digital approaches to construction are increasingly yielding positive results and more projects that would otherwise pose high risk of cost overrun are being delivered timely and on budget. Digitalisation of procurement process through the use of approaches such as e-procurement, e-tender and e-sourcing has increased cooperation between client and contractors because of the confidence developed through sharing of accurate data and clarity of information. This has enabled misunderstanding and barriers of culture to be better managed.

In the 1970s, nearly all stakeholders in the construction sector felt challenges of the fast growing technologies and they started to consider improvements to project processes. In the early 1980s the UK government introduced compulsory competitive tender to harness opportunities of growing competition to reduce overall construction cost. This was later relaxed in the early 1990s because of the insecurity it created, and resulted in greater fragmentation in the delivery of projects. Latham's report in 1994 on constructing the team emphasised the need for working collaboratively by collaborating with the supply chain to reduce construction costs and deliver projects more predictably. Construction projects continued to face twin challenges in the wake of the new millennium. They must deliver client value while also have to be resilient to the normality of changing climate and users behaviours. Lean management approaches emerged are a panacea in the early 2000s and were considered for wider application in Virtual Construction which then became what we know as building information modelling (BIM). BIM provides a digital representation of physical and functional characteristics of a facility which can easily be communicated with non-technical stakeholders. The sharing of knowledge gives clarity and shared vision of the project and the resource required in a more reliable manner so that appropriate decisions on its life-cycle can be made.

BIM is a relatively new paradigm [1] in the construction industry trying to integrate three pillars: people, process and technology to deliver assets that meet client's requirements. BIM extends management information system (MIS) and sometimes it is referred to as a specialist business information management system for construction projects. Through BIM key requirements are captured, analysed and shared to achieve higher levels of collaboration. It is in fact an effective tool for

stakeholder engagement as it enables them to take advantage of technology that is linked to a common data environment (CDE), which can be remotely accessed at any time. The added value is that it integrates collaborative technologies and fosters the development of a collaborative culture through the project life cycle. An integrated collaborative environment brings to light projects challenges so that they are managed in a proactive fashion. Since 2016, BIM was made mandatory in the UK for centrally funded projects with the aim of realising a 33% cost reduction and to develop faster delivery schedules that could reduce overall project duration and emission by 50% (Construction 2025 Report).

The use of BIM collaborative approach in the UK railway industry is so far limited to the construction of new lines like Cross Rail and High Speed link 2 (HS2). Its use in a complex operational environment like that of track renewals and monitoring of tunnels and other structures will need adaptation. Vast amount of existing data from various work streams and in various formats need integrating into management intelligence to develop accurate, prioritised maintenance plans to optimise asset availability, essential for efficient running of train services [2].

In the UK, partnerships and collaborative working have long been the preferred method of procuring railway maintenance projects [3] to maximise efficiencies. The UK government construction strategy and its commitment to long-term partnerships to deliver infrastructure projects is revolutionising ways of working and data management techniques. Rapid advances in technology, increases in capacity to handle large volumes of data at lower costs and the development of quicker data analysis tools are increasingly making data management at the core of strategic planning for organisations [1]. Improving asset data management will provide more accurate baseline data and facilitate multi-use of existing data to reduce unnecessary reworks. However, the future use of partnerships to procure railway maintenance work will need adaptation to the new ways of working.

Railway asset maintenance disciplines can standardise their approaches and adapt these modern approaches to suite their unique requirements. Furthermore, in April 2016, the UK government mandated, centrally funded projects procuring public assets to be delivered in fully collaborative 3D, BIM environment [4]. UK rail operators invest millions of pounds in upgrading rolling stock but the state-of-the-art trains often run on much older network infrastructure that consists of tunnel sections the majority of which were constructed over 150 years ago. The railway tunnels were built to last but eventually there comes a time when elements need upgrading and/or replacing after degradation resulting from various factors including vibration, high-speed air flow, corrosion, water ingress and vegetation growth. Poor construction techniques in the past and changing ground conditions also occasionally cause weak points that trigger the need for maintenance works [5]. To ensure safe operation of railway tunnels, they have to be continuously monitored with a tunnel management strategy in place to determine when it is time to take corrective action to mitigate any potential risks. Maintaining existing assets is as important as delivering capacity improvements through the construction of new lines and services for UK railway operators. However, tunnel repair works often cause service disruptions and are a challenge to deliver safely due to space and logistical constraints. As more tunnels are built to meet the rising demand in railway usage, the need for tunnel maintenance will also increase.

## **2. Integration and collaboration**

The multitude of internal stakeholders involved in the briefing, designing, construction and commissioning of a project means that there will be varied interests.

In traditional project delivery methods, fragmentation has been the case that often starves off collaboration to create communication gaps. When the client and the construction team share a common goal on the project, conflicts are reduced and focus is on the project. This enhances cooperation, better stakeholder management and improves chances of successful delivery outcome.

Stakeholders can be an asset to the project when a collaborative environment is developed to enable project data and information to be held in a common data environment (CDE) accessible to the team to support decision-making. Design drawings produced in the formats usable by all, including specialist subcontractors, will reduce requests for customised information from the designers. A collaborative team must maintain an unlimited access to data and information to enhance their knowledge in dealing with problems and thus supporting both decision-making and problem solving. Rowley and Jennifer refer the continuum from data to information and information to knowledge as the principles of the hierarchy of human understanding. This reinforces the importance of data and information management to improve delivery performance of projects.

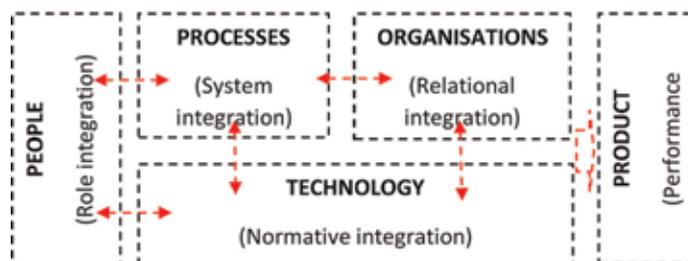
Front loading time to design and plan for the project in the project definition phase will pay off in the long run, the project team is able to anticipate risk and set mitigation plans and contingency. Design and sequencing issues can cost up to 10 times more to rectify if identified during the construction and later phase of the project life. Technology such as virtual reality (VR) is now available to enhance solutions and should be prioritised for risk assessment.

**Figure 1** reinforces the importance of technology as an integrator of people, processes and organisations in the integrated construction environment. This must however, be supported by setting clear communication channels and responsibilities of project team members.

The sharing of data and information, whether formal or informal, can be enhanced by Information Communication Technologies (ICT Technology enables visualisation of production systems and subsystems, so that clashes are detected early in the design and planning stages and resolved. With improved project planning, logistics, both local and across borders, will be coordinated with greater efficiency. Supply chain management also improves which helps in developing trust and a shared culture of quality shared across the supply chain.

Collaboration in Lean and Agile project management show that could work interactively, integrated and intelligent in a unique way to support decision making, problem solving and also pre-identifying project risks. The core requirement is to seek accurate data, the right information that is shared among trustful resources and could add value to the final product (asset).

Although collaboration aims to support information sharing, increased interactions also helps project teams to perform more effectively and efficiently. This further moderates the effects of collaboration on team member learning. Beyond



**Figure 1.**  
*Integrated construction environment.*

the productivity issues, decision making in a collaborative working environment are made more is occurred effectively and thus problems could be solved promptly.

Since the late 1990s, a new trend of research on collaborative learning focusing on new technologies for mediating, observing, and recording interactions during collaboration has emerged known as computer supported collaborative learning (CSCL). It typically uses online networks for facilitating and recording online interactions among two or more individuals who may be geographically and/or temporally dispersed. In construction though there is a need to adapt the technology to improve the design and planning processes in a secure common data environment (CDE). However according to [6] there is a growing trend to design and develop integrated collaborative environment that allows project stakeholders to interact either Mobile, co-located or distant. CoSpaces Projects [7], an IST Funded Project by the EU shows how this concept could be achieved by using different technologies that provide different information richness ([www.cospaces.org](http://www.cospaces.org)). In addition, collaborative tools help facilitate action-oriented teams working together over distant geographic locations, by providing tools that aid communication, collaboration and the means of problem solving.

Technology Integration is the use of technology tools in general content areas in businesses in order to allow stakeholders to apply computer and technology skills to learning and problem-solving. Collaboration requires individuals working together in a coordinated fashion, towards a common goal. Arguably Integrated Collaborative Technologies are those tools that can help stakeholders work collectively towards problem solving without considering geographical distance. These technologies could work either in a synchronous (real time) or asynchronous (not real time) manner, so allowing the stakeholders or the team members to share documents or files from anywhere at any time.

## **2.1 The need for collaborative working in railway tunnelling**

A study [8] in 2008 mentioned that Network Rail (NR) was spending €433 million track maintenance and €1.305 billion more on track renewals, compared to its European peers. They argued that NR can unlock contractor efficiency contributions by a fundamental shift of supply chain relations based on the idea of competition and partnerships. They identified the main areas of improvement to be in planning, better use of possessions, standardisation of asset configuration and focus on quality of the asset condition. With a number of existing tunnel sections geographically distributed across the railway network, coordination of maintenance works is essential to deliver value for money. Crucial to the successful implementation of the strategy is the development of a collaborative culture (CC) and Integrated Data Management Systems (IDMS) throughout the supply chain to allow for better use of possessions and the design of new lines that connect to the existing infrastructure.

According to [9], despite the apparent lack of clear guide the process of collaboration between main contractor and subcontractor, project participants now realise that sharing of knowledge and information is a key element of a successful project delivery and contractual relationship. However [10] provided the strategy to improve collaboration to enhance organisational performance and project delivery through the management of process, people and data and wrote that, despite very strong willingness to collaborate, culture and awareness remain as significant barriers to adoption.

Rail industry leaders recognise the need for change. According to [11] wrote that the introduction of BS11000 in 2010 marked a step change in thinking for the UK rail industry led by Network Rail to encourage the adoption of CC whose benefits

had so far be limited to key programmes. Cross Rail, the biggest rail project in Europe has attributed success so far, in delivering the project with 42 km/26 miles of tunnelling to the strategy of adopting CC, through the supply chain under the NEC contracts on its 40 construction sites according to [12]. Cross Rail further adopted an innovative approach to data management for efficiency in project delivery, based on the principles of PAS 1192-2 as reported in [13].

Railway asset maintenance projects are delivered in a complex operational environment with various asset maintenance disciplines carrying out other works. The maintenance of railway tunnels is further complicated by limited working space, different tunnel designs and materials used. As a result, repair works are not straight forward but having the correct information at the right time helps in decisions about maintenance requirements. A collaborative approach to asset management could facilitate asset data sharing leading to a better understanding of the asset condition. This requires an integrated approach to asset data management that should standardise ways of working and data formats, used by stakeholders to the benefit of the industry. However, this must start with projects having sound data management systems that feed data of high integrity to asset data models.

## **2.2 Data Management in the rail sector (As-is)**

Naturally, organisations have different ways of managing project data to deliver their objectives. According to [14] argued that during the project life cycle, vast amounts of data are generated by different departments in various formats and stored in many different places in an unstructured way. This occasionally leads to losses of data and time spent in unproductive searches.

Decisions on track sections due for renewal are made based on the data/information held in the asset management models like Maximo and the Ellipse for LU [13]. Keeping such models updated with accurate data is fundamental to accurately determine maintenance requirements and ensure adequate funding is available to optimise asset availability, essential for operating a safe and reliable train service. In the report [14] noted that maintenance of the railways is often undertaken with insufficient information and limited resources. According to [13] added that quite often deliverables in large rail infrastructure projects are not clearly defined. This may result in costly scope creeps and reworks in design and other preparatory works.

In addition to his report [14] it was added that railways rarely have enough resources to maintain the track asset at a level that ensures optimum operation of services. They are instead faced with prioritising maintenance actions to optimise safety and reliability. In a recent research about the future of digital railways, [15] wrote that railway operators are struggling to improve asset management, partly because the systems that generate and store data are not connected, even though the capability exists. The concept was affirmed by [1] who urged that in the recent times such levels of organisational intelligence are no longer a fringe concept but at the core of future investment decisions. This affirms the suggestion by [14] that the industry is finding that the solution to working more efficiently lies in using information technology.

The use of technology has enabled faster ways of data collection with gigabytes of data collected, often stored in many different places [14] and creating data silos. ABB's report [15] affirmed and wrote that, too often collected data is accessed by individual departments according to their needs, without a broader view allowing for the organisation to gain a full picture of their asset condition. This makes it difficult to analyse the generated large volumes of data together for effective decision-making about maintenance priorities.

In 2010 [16] and 2017 [2] wrote that project directors and senior managers of leading institutions are advocating for the adoption of common data capture and storage standards across all major projects to have a better understanding of the asset delivered. This can enable multiple-use of data across project disciplines/ departments. Further benefits can be achieved on the use of a single survey grid for geo-spatial data to enable designs to be overlapped and multi-use of data of fixed assets without the need for transformations or reworks.

### **2.3 Industrial responses**

Cross Rail project adopted an innovative approach to data management in a common data environment (CDE) based on the principles of PAS 1192 series [13]. This was to improve delivery performance through better management of data/ information and facilitate structured asset hand over, ensuring that asset managers received reliable information critical to the safe and efficient operation and maintenance of the railway. With over 60 contractors and subcontractors on the project, the adopted method of information management in the CDE was developed in line with BS1192: 2007 and PAS1192-3. A new network consisting of 42 km of tunnels bored by giant tunnel boring machine 40 m below ground level was constructed. Tunnelling was successfully completed in 2015 having negotiated through various underground utilities and existing railway infrastructure.

Research done for NR recommended suggested that the increased use of mechanisation and better methods of data capture and storage. This would increase the accuracy of information generated about the condition of the delivered asset. Such information is later relied upon for effective decisions on maintenance requirements based on [8, 17].

Modern collaborative and data management approaches have mainly been limited to railway green-field sites. In the 2017 report [2] it was suggested that the railways brown field site could also benefit from the creation of the digital rail model from the existing information and maintenance processes. But, noted the challenge of incremental migration from the current, fragmented state to a single integrated model. A structured approach to data management can make it easier to manage job data files to reduce the use of different data, recollection of already existing data, using out of date data and ensure job closeout packages contain data of high integrity.

In addition in Greece—Attiko Metro announce for the Line 4 (new metro line) to use BIM files [18]. High Speed 2 project in the UK requested from their suppliers [19] to be BIM compliance. Though in Scandinavian Countries where BIM is well populated and applied it can evident show the use of BIM in planning stage [20]. Furthermore, it has to be noted that Heikkilä [21] noted that the use of BIM in tunnelling can add value in advancing intelligent information modelling; however, policy in Finland is not available yet. In contrary though Norway has HB138 that determines some basics for the BIM process of tunnels.

### **3. Research in collaborative working**

The use of railways is forecast to grow about 50% by the mid-2030s [22] while the maintenance costs are said to be rising according to [23]. On the other hand, funding to increase capacity and maintain the infrastructure continues to be squeezed. Railway operators and asset managers are thus facing challenges to operate efficiently under tight budgets while transforming the old network to meet the future needs of an integrated transport network. Studies show that there is need for

change in the culture of the industry [8, 15, 22, 24, 25] and a number of performance improvement suggestions have been put forward. The emerging theme is that fragmented renewal projects need to change and adopt collaborative approaches that operate in CDE supported by technology to improve performance in planning, design, construction as-built data management and hand-back of projects delivered in a complex and fragmented railway sector. This will further impact on the asset data quality that is relied upon in making decisions about asset condition and maintenance requirements, and help improve future decisions on maintenance requirements.

Research in collaboration spans a number of incongruent fields such as organisational and social psychology, human factors, computer science, management science, education, and healthcare. In March 2009 the University of Nottingham, a partner in the European Funded project CoSpaces, published the attributes which influence and form part of collaborative work as well as developing an explanatory, descriptive model in order to enable a unified understanding of what it is to collaborate, and how best to communicate this to industry and to support collaborative working based on this understanding. The technique/method followed to check the validity of this research was semi-structured interviews with the CoSpaces user partners, and through drawing on the broad experience of working with a range of industrial organisations [17]. The main factors (individuals, teams, interaction processes, tasks, support, context and overarching factors) and sub-factors (with supporting references) give an overview of their relevance and importance to collaborative working. In addition, in order to assess how meaningful the factors in the model are, a series of card sorting exercises with human factors experts, took place. The study [17] showed that there was a general agreement on the main factors proposed for the model of collaboration. Moreover, groups of human factors experts reviewed the 27 different representational styles for a model of collaborative working, and incorporated the factors that had been considered during the card sort.

In particular, the external factors that influence building collaboration in a business environment and in a project are: trust, time, performance, management, conflict, goals, incentives, constraints and experience. The internal factors influencing the building of collaboration in a business are: teams, individuals, context, support, tasks and interaction processes. In order for external and internal factors to be applied during the project life cycle a number of different activities, behaviours and skills have to be developed.

The social behaviour of employees has a great impact on an organisation's effectiveness within the construction sector. Many aspects of social behaviour are manifested in project managers in interaction with team members. Moreover, working in teams magnifies and intensifies behavioural characteristics as a result of the close encounters that members have with each other, in terms of both formal and informal attitudes, where express responses/decisions are required for problem resolution. Proactive behaviour as a social behaviour impacts on project and organisational effectiveness [26] but the research in this paper show the need to explore and explain how project managers' proactive behaviour could be enhanced in a project.

In the paper [27] proactive behaviour was referred to as, "taking initiative in improving current circumstances; it involves challenging the status quo rather than passively adapting present conditions". In the paper [28] defined proactive behaviour as "self-initiated and future-oriented action that aims to change and improve the situation or oneself". As it is a relatively new field, there is no precise definition of proactive behaviour and current definitions are somewhat unclear and even contentious. Nevertheless, in recent times, a consensus appears to be emerging as to the definition of proactive behaviour, as suggested in [29]. Dictionary definitions

typically highlight two key elements of proactivity. Firstly, they identify an anticipatory element involving acting in advance of a future situation, such as acting in anticipation of future problems, needs, or changes. Secondly, the definitions emphasise taking control and causing change, for example: “controlling a situation by causing something to happen rather than waiting to respond to it after it happens”. In the paper [30] proactive behaviour is defined as “anticipatory action that employees take to impact on themselves and/or their environments”. In particular proactive behaviour has three key features:

- It is anticipatory—it involves acting in advance of a future situation, rather than just reacting.
- It is change-oriented—being proactive means taking control and causing something to happen, rather than just adapting to a situation or waiting for something to happen.
- It is self-initiated—the individual does not need to be asked to act, nor do they require detailed instructions.

The dynamic view of managing projects successfully is through enhancing the skills of the project manager in the manner of controlling and making more accurate decisions. With the increasing number of projects delivered in BIM environment, project managers’ skills must be adapted to suit. What is mainly needed in order to advance the project manager’s skills is the capability to interact with the other participants or members of the organisation or project to foster a collaborative culture. This interaction enhances the communication and the collaboration and develops the building of trust between the project manager and the participants. Estrin [31] stated that, “innovators must trust themselves, trust the people with whom they work, and trust the people with whom they partner, balancing their progress in an environment that demands both self-doubt and self-confidence”.

Communication constitutes conceptualising the processes by which people navigate and assign meaning and is an essential element of collaboration. Communication is also understood as the exchanging of understanding. Montiel-Overall [32] defined collaboration as “a trusting relationship between two or more equal participants involved in sharing thinking, shared planning and shared creation”.

In the research [30] supported the assertion that, in order to enhance trust, communication and collaboration, the construction of the following skills is required: anticipatory skills, change orientation and self-initiation skills. Henceforth, these skills will lead to the development of proactive behaviour. Therefore, a successful project manager/managers need(s) to be self-initiated, future oriented and anticipative. This behavioural situation will be used as the driving force that will initiate change in the operational and organisational system of a company. This approach will give an added value to the current state-of-the-art in project management. The proactivity concept assists project managers to think and act before, during and after a meeting takes place.

Moreover in paper [28] captured and analysed the proactive cognitive model. The model consists of proactive personality, job autonomy, co-worker trust, supportive supervision, self-efficacy, flexible role orientation (organisational commitment) and control appraisal.

The definition of each of the model’s elements is listed below:

- Flexible role orientation indicates the extent to which various problems affecting the longer term goals of projects would be of personal concern to an individual rather than to someone else.

- Co-worker trust refers to trust among the members of a project team.
- Self-efficacy refers to how confident a project manager feels in carrying out a range of proactive, interpersonal and integrative project tasks.
- Control appraisal refers to a belief that a project manager can control and have an impact on project outcomes.
- Change orientation refers to those project managers that have the intention of initiating/proposing changes in a project/task so as to optimise projects/tasks procedures and or performance(s).
- Job autonomy refers to the extent to which the project manager is involved in making decisions within the team.

	Project culture	Data governance	Technology
Key points	<ul style="list-style-type: none"> <li>• Individual skills and behaviour</li> <li>• Individuals in teams with complementary skill sets</li> <li>• Aligned individual/team goals with that of organisation</li> <li>• Interaction of individuals in teams and departments</li> <li>• Trust and experience</li> <li>• Suitable working environment</li> <li>• Coordination</li> <li>• Effective communication i.e. verbal, electronic</li> </ul>	<ul style="list-style-type: none"> <li>• Legislation</li> <li>• Data policy</li> <li>• Grid definition (geo-spatial and CAD)</li> <li>• Data quality assurance</li> <li>• Data storage and sharing</li> <li>• Defined working area</li> <li>• Published data/information area</li> <li>• Data shelf life</li> <li>• Control of updates</li> </ul>	<ul style="list-style-type: none"> <li>• LAB</li> <li>• IT tools</li> <li>• Common data environment—e.g. Bentley AssetWise, Bentley ProjectWise, SAP, Livelink, Asite</li> <li>• Access rights</li> <li>• Editing rights</li> <li>• Communication e.g. Skype, emails, notifications</li> </ul>
Supporting frameworks or standards	<ul style="list-style-type: none"> <li>• ISO44001—Institute of Collaborative Working, CRAFT model</li> <li>• CoSpaces</li> <li>• Integrated Project</li> <li>• Four Ways of Collaboration</li> </ul>	<ul style="list-style-type: none"> <li>• ISO9001 Quality Management</li> <li>• Railway industry standards</li> <li>• Guidance of data management—IGGI</li> </ul>	<ul style="list-style-type: none"> <li>• ISO 19650 and PAS 110092 series</li> </ul>
<p>Integrated project collaborative environment                      Collaborative culture—cooperation (clear definition of scope and deliverables)                      Integrated data management system—(one source of truth for design, construction and handover data)                      Lean construction—(elimination of wasteful activities through better coordination, planning and scheduling)                      Process (BIM)—holistic approach to asset data management and asset maintenance</p>			

**Table 1.**  
*Requirements for a collaborative environment in railway asset maintenance projects.*

- Proactive personality refers to the relatively stable tendency to identify problems in advance.
- Supportive supervision refers to the enhancement of leader effectiveness in a self-management context.

What can be gathered from the above is the need to focus on low project information maturity in order to enhance the progress of a project. The use of BIM and supporting tools then ensures that project information is of the right quality and shared from a CDE so that team members access the same information. The proactive project manager's behaviour aids in developing project information maturity to deliver the project that meets the client's requirements. The added value is that more evidence based higher quality of decisions will be made and the delivery team will have better control of the project through its delivery and operational life cycle.

There is consensus that collaborative approaches are part of the solution to improve project performance. However, as highlighted in the definitions of collaboration [33] and interpretations, the words collaboration and partnership are often used interchangeably making it difficult to have a clear meaning and difference between them. Despite the concept being widely used, there is no clear understanding of what collaboration is [17].

The complexity of collaboration in the rail industry can also be seen from the procurement perspective, where partnerships in various forms seem to be the preferred option in track renewal projects [33, 34] and presumed to be working collaboratively. Collaboration has been seen to improve project performance and its effectiveness can be enhanced through support from good technologies [17] and processes to further add value in the delivery of renewal projects.

The adoption of CC supported by technology in railway asset maintenance projects using IDMS in a collaborative environment can no-longer be ignored in the search for solutions to lower maintenance costs, deliver value for money and provide efficient services, in the face of shrinking budgets. Further, it will facilitate the integration of the UK transport network to provide future customers a first class travel experience of connected means of transport supported by Big Data. **Table 1** shows the requirements for collaborative environment in renewal projects that must be supported by Senior Management and a suitable form of contract/procurement process based on a track renewals case study.

#### **4. Technology in tunnelling construction**

The project data must be kept in a secure environment with set permissions in order for stakeholders and project team members to have access from anywhere at any time. In order to achieve this then a common data environment (CDE) needs to be set up. In this environment all project data need to be correctly labelled based of standard file naming conversions form the ISO 19650 standard. This makes searches using metadata easier and saves time wasted on unproductive searches. Having easy but secure remote access also allows for further (project) data analysis that can help to pre-identify any project activities that provide more information about who is involved in the projects (Organisation Breakdown Structure), what each person is expected to deliver (Work Breakdown Structure) and what the cost of each activity is (Cost Breakdown Structure). All the captured and updated data might be used for further analysis with the aim to support any decisions before, during and after

completion of the project. Such data analysis could support investor(s) capacity to understand based on evidence whether it is worthy to progress their project idea as well as to pre-identify any risks that can be evaluated in consideration of other factors that may affect the project such as in any political, economic, social, environmental, legislation, technological and sustainability issues.

Data used in tunnelling projects is collected using many different tools such as Laser Scanner, Sensors, inspections, etc., and comes in various formats. All these data sets could be available to stakeholders synchronously i.e. either in real time or not. With the increase in analytical tools for infrastructure maintenance, Big Data analytics is an increasingly area in construction sector due to high Volume, Value, Variety, Velocity and Veracity generated through the project whole life cycle. Concerns in copyrights for cloud based data and BIM models are beyond the scope of this book. In addition the power of the Internet of Things (IoT) as a network of physical devices, vehicles (suppliers), and other project materials embedded with electronics, sensors, software, actuators and connectivity enables construction asset/building objects to connect and exchange data too.

The power of visualising information using integrated collaborative environment that uses software such as Autodesk Revit, CADduct and Tekla, Navisworks, Solibri allows stakeholders and team members to understand coherently and stimulate any problem when it occurs. Among the core project challenges, AEC industry faces the lack of sharing of information and cooperation in the development of processes to improve efficiency in the delivery of construction projects.

This is mainly what BIM is renowned for and is the reason why the BIM integrated environment requires further development in the construction industry. Kapogiannis and Sherratt highlight the need and impact of collaboration culture in the architecture, engineering and construction sector.

Considerably the knowledge about BIM and the higher levels of maturity requires further development of processes aiming to improve construction efficiency through automation. Tools that could help to support and enhance knowledge to humans are by teaching machines using Artificial Intelligence and Machine Learning. This idea could go further down to simulate a process and/or an event in a project, for example delivering materials from suppliers or using a robot to paint a wall fairly quickly within cost budget. This can also help stakeholders to generate new business models.

The regular review (Decision) of models and management of associated stage deliverables using information exchange platforms such as COBie to the employer is a key aspect of the BIM process. The employer should ensure that the Exchange Information Requirements (EIR) are defined and agreed in the procurement stage of the project, in a collaborative environment.



D-CIT Lab at The University of Nottingham Ningbo China



Anglia Ruskin University Active Learning Environment

Project meetings undertaken in collaborative environments allow key stakeholders to determine, understand, analyse and review the design models in 3D and other outputs, provide their feedback and validate the stage PLQs. Ideally there should be three key stages to the process:

1. Before review meeting,
2. During review meeting,
3. Post review meeting.

This workflow allows the team to develop a proactive behaviour whilst collaborating and sharing key projects/asset data and information. Check more on BIMPortal of the Scottish Future Trust [34].

For project delivered in high levels of BIM maturity, such review meetings are normally undertaken in a virtual environment using a projector (as illustrated below) or on larger projects using virtual mock-up facilities, such as CAVE (Computer Assisted Virtual Environment) and or immersive lab often available at local universities or further education colleges.

## **5. Benefits and challenge of the use of BIM**

In a survey of 1000 active BIM users in the UK the NBS [35] concluded that BIM was a useful strategy in achieving goals of the construction strategy 2025 as it brings cost effectiveness and reduce time from inception to completion. This conclusion was supported by the fact that 70% of participants believe that BIM reduces overall project cost including initial cost of construction and the whole life cost of built assets. Another 60% agreed that BIM reduces overall time, from inception to completion, for new build and refurbished assets and helps to meet the target of 33% reduction in initial.

- Eliminate costly and timely traditional construction mock-ups
- Different design options and alternatives may be easily modelled and changed in real-time during design review base on end users and/or owner feedbacks
- Create shorter and more efficient design and design review process
- Evaluate effectiveness of design in meeting building program criteria and owner's needs
- Enhance the health, safety and welfare performance of their projects (for instance, BIM can be used to analyse and compare fire-rated egress enclosures, automatic sprinkler system designs, and alternate stair layouts
- Easily communicate the design to the owner, construction team and end users
- Get instant feedbacks on meeting program requirements, owner's needs and building or space aesthetics

But still the biggest challenge is not the use of the aforementioned technologies in a project process but to change the way of thinking by engaging team members. Research shows the significant impact of gamification to support engagement. The potential impact of the use of gamification in construction projects is to change the behaviour of stakeholders and motivate them for better performance through the triggering of their social skills. In our days many enterprises use gamification in order to enhance the collaboration and communication between their employees.

A recent study published by CITB (Construction Institute Training Board) indicates that in the UK, the construction industry will grow by almost 3% and create many job vacancies due to the need of collaboration over the next 5–10 years.

Building information modelling provides the construction industry with an environment and framework for great coordination and integration of people and processes through an open sharing of data and information. A BIM environment comprises of computer-aided design tools that can a better understanding of design and construction sequences through a virtual representation of the built asset so that the construction team and owners and operators can take part in detecting and resolving conflicts in a proactive fashion.

On project delivered in BIM environment, the Client has a strong influence on the extent, effectiveness and efficient use of BIM. Client may also be motivated by short terms goals such as using virtual reality to improve acceptance of the project scope by decision makers. While adoption of BIM for mega projects is likely to generate an advantageous returns on investment, for smaller project a decision on the use of BIM should be made based on the appropriate level of maturity after a careful considerations of cost and benefits. Perhaps the use of BIM on projects should be made on grounds that BIM will be implemented to meet long-term goals in case there are several projects are forthcoming to outweigh the initial investment. It is likely that cost invested in a lifecycle BIM toolkits will be paid back via cost reduction from intelligent operations of the built asset, modernised ways of working that improves service delivery or increasing global competitiveness. BIM has been recommended by designers who already are familiar with it. Clients are likely to maximise benefits of BIM if its use is extended to include asset management and compatible software are in place.

The RIBA plan of work 2013 stages provide for an overlap with BIM tools.

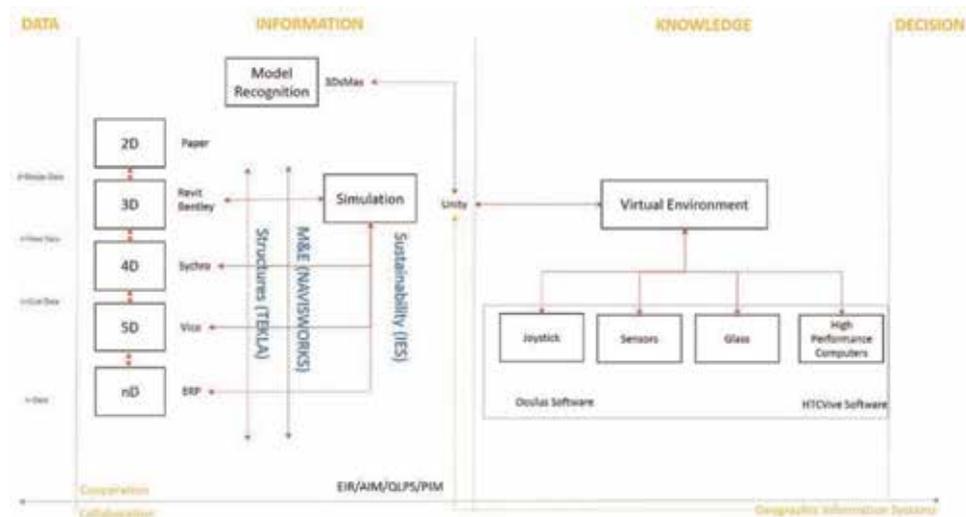
- Stage 1. BIM can be adopted at any stage within the asset development process but to benefit more from BIM, it would be useful to formalise BIM mandate at the strategic definition or inception. When decision to incorporate BIM is made as early as RIBA stage 1 client gets an adequate time to prepare requirements and resources that will facilitate aligning BIM with asset management systems when the project is completed.
- Stage 2. BIM integration strategy will be developed and tried in the concept design stage so that a functioning BIM environment is created. In the concept design stage BIM strategy should be developed to provide an environment for project integration. The strategy will identify the extent to which BIM will be used. It should also assess BIM capabilities in terms of expected product outcomes, required trainings, communication channel, roles and responsibilities and BIM tools. The use of performance-based BIM specifications focusing on product outcome has an added advantage over prescriptive requirements for software. BIM will be used by the lead designer as design tool and to communicate design solutions with project internal stakeholders to facilitate decision making.
- Stages 3–4. A fully integration of BIM is possible during the developed design stage which is characterised by increased in information exchange between the designers and quantity surveyors. A virtual value engineering process will simulate design solutions for cost affordability. Consideration of design options can benefit from visual models shared for analysis and risk reduction. Multidisciplinary 3D geometric models can be shared with others designers and the construction teams through a shared BIM environment so that

buildability issues and impact to the environment can be assessed. Due to the number of exchanges in a coordinated design, high level of design agility can be attained by rapidly adopting changes from peer reviewers and on so doing a design development cycle time can significantly be reduces.

- Stage 5–6. By taking on board all the information modelling then contractor and the supply chain start and manage the construction—building process in an efficient way where tablet and mobile technology could the team could manage the control process, quality assurance, productivity, efficiency, buildability etc.
- Stage 7. ‘As-constructed’ Information updated in response to ongoing client feedback and maintenance or operational developments.

An integrated BIM environment is a virtual workplace that creates a common data environment (CDE) to facilitate an efficient and two-ways exchange of data and models. **Figure 2** is an input process output model illustrating a BIM environment. The most important input for an integrated whole life BIM integration is client’s information requirement which brief the project team needs of the client and how they will align BIM to post project operations. Input data from fully dimensioned 2D and 3D geometric models from designers can be pushed to other specialist designers or the construction teams for feedback or inputs with a reduced risk of data loss.

The first step in implementing BIM in a project basis is for the client to develop an outline BIM strategy at the inception stage in collaboration with the project consultants. The client will prepare a statement, which will form the basis for appointing both the designers and contractors. This statement is referred to as Employers information requirement (EIR) that clearly explain why BIM should be used, its drivers and commitment and capability of the client to collaborate in a BIM environment. Where adoption of BIM is decided upfront, client must make the expectations known before appointing the lead designer. The client will also specify technical information software platforms that align with existing or recommended facilities management software. This has been confirmed though through [26]



**Figure 2.**  
 An integrated collaborative environment © Kapogiannis, 2018.

where researchers shown the added value of BIM in Facilities Management in the hospitals by integrating technologies incorporating 3D modelling and beyond. Using performance based technical specifications empowers designers and contractors to come with most efficient and cost effective solutions to meet the EIR. Prescriptive requirements such as software vendors will force the supply side to incur additional costs of retraining for compliance. The EIR will also stipulate roles and responsibilities including for platforms that will be shared by several parties. It is crucial that management issues are covered in the mandate given to the project manager or BIM manager. The mandate will specify the protocol to be adopted, how information will be secured, system performance, how coordination and will be achieved and issues of ownership. At this stage it will be useful to specify if models will be part of deliverables to be used as part of tender documents.

The NBS protocol defines BIM execution plan as a response of designers and contractors to the employer's information requirements (EIR). It will be prepared as part of the project implementation plan in the pre-contract stage and will be updated after the contract is signed. The pre-contract execution plan communicates the approach to data exchange and confirms the ability the designer or contract to meet expectations of the client.

It is inevitable that a huge volume of information will exchanged through several iterations of value engineering taking place during the design and as the work commences on site. A typical design process will include internal and external peer reviews and approvals from the client. The iterative process will assure the client that the final design is optimum and guarantees good value for money. To streamline communication of information during the design and construction process the project manager will develop and maintain a master information delivery plan (MIDP) which services similar purposes as a communication plan with the focus on information exchange with main collaborators. Although the responsibility of is with the project manager but this is a collaborative document and it should be developed jointly with managers leading the design, construction and procurement. The plan answers information exchange questions such as who is creating a 3D model, when it will be prepared and based on what procedures. The plan must be specific on the deliverables including models, drawings, specifications and whether they will form part of the tender documents to be distributed to bidders. A more detailed execution plan will be agreed by a team of designers and other consultants post-contract. The information delivery plan will be incorporated in the post contract execution plan setting out a strategy for delivering technical design information as well as management reports.

## **6. BIM standards and contracts**

BIM can adopt an information system that promotes a single point of truth to leverage information exchange in a spirit of collaboration and open sharing. However, it is important to emphasise that the collaboration process will involve multidisciplinary teams such as technical designers, cost consultants, facility managers, planners and other reviewers, each of them using a different software platform and are based in different geographic locations. A typical example would be a 3D geometrical models produced by the Architect. The model will be peer-reviewed by other designers before the final approval by the client. It may also be modified by other users when a clash is detected. Information exchange in a BIM environment is likely to lead to series of issues. It is now possible to adopt a BIM integrated system that supports multi applications. Another challenge will be multiple users modifying the model concurrently thereby creating version conflicts. A lockable BIM integrated system is preferred in this case to reduce conflicts and residual data loss.

The exchanged data and models are often large files and if shared using internet based technologies may results into speed and security issues which the adopted BIM system has to revolve proactively.

Considering the fact that procurement strategy and its contract is the key to operate a construction project then BIM might be a new swift that could in the future affect also the way of bid—tender relationship and its management. Crawford and Stephan [36] mentioned the “ultimately also the opportunities BIM offers in revolutionizing the way projects are procured in the first place”. The below diagram (**Figure 3**) picks up on some typical processes associated to BIM that can be applied across different contract procurement methods.

Furthermore there are the following protocols are required to be followed in BIM Level 2 according to Digital Built Britain 2018 [37].

PAS 1192-6 specifies requirements for the collaborative sharing of structured H&S information throughout the project and asset life-cycles. This PAS standard supports the development of structured H&S information for all construction projects progressively from the outset.

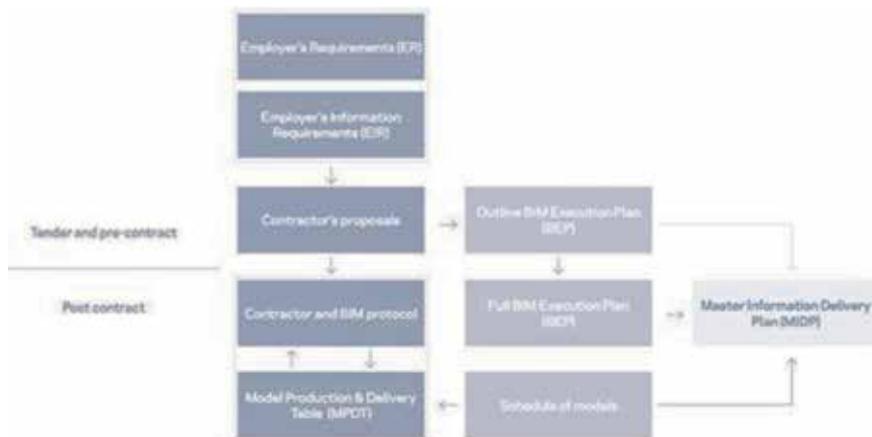
PAS 1192-5 specifies requirements for security-minded management of BIM and digital built environments. It outlines the cyber-security vulnerabilities to hostile attack when using BIM and provides an assessment process to determine the levels of cyber-security for BIM collaboration which should be applied during all phases of the site and building lifecycle.

ISO 19650-1 (former BS 1192:2007 + A2:2016) provides a ‘best-practice’ method for the development, organisation and management of production information for the construction industry, using a disciplined process for collaboration and a specified naming policy.

ISO 19650-2 (former PAS 1192-2:2013): the requirements within PAS 1192-2 build on the existing code of practice for the collaborative production of architectural, engineering and construction information, defined within BS 1192:2007 + A2:2016.

PAS 1192-3 provides guidance to Asset Managers on how to integrate the management of information across the longer term activity of asset management with the shorter term activity of asset construction for a portfolio of assets.

BS 1192 4 outlines the UK usage of COBie, an internationally agreed information exchange schema for exchanging facility information between the employer and the supply chain.



**Figure 3.**  
 BIM execution plan (BEP)—procurement process adopted by ISO19560 1 and 2.

BS 8536-1:2015 gives recommendations for briefing for design and construction, to ensure that designers consider the expected performance of a building in use. The standard applies to all new buildings projects and major refurbishments. Also aims to (a) involve the operator, the operations team and their supply chain from the outset and (b) extend the involvement of the supply chain for the project's delivery through to operations and defined periods of aftercare. The scope of the revised BS 8536-1 has been expanded to include briefing requirements for soft landings, building information modelling (BIM) and post occupancy evaluation (POE).

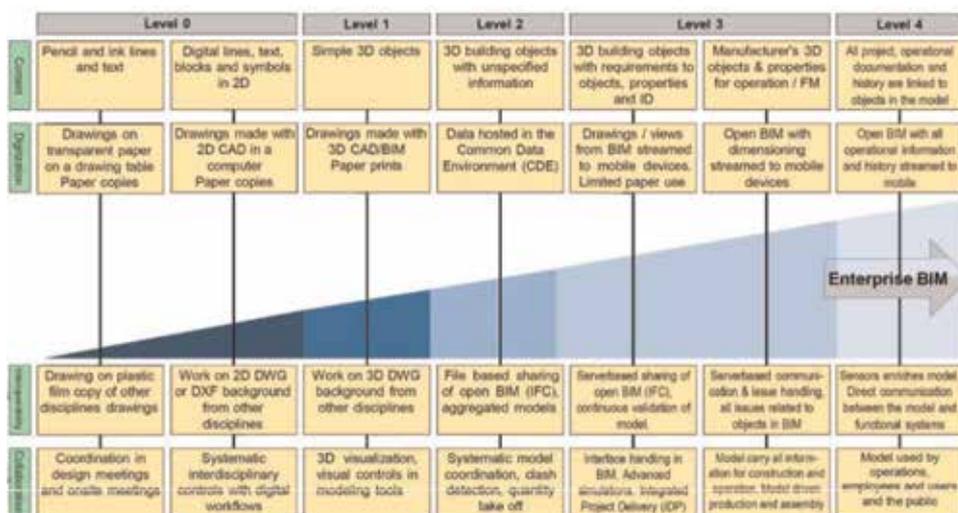
BS 8536-2:2016 is part of the BIM level 2 suite of documents developed to help the construction industry adopt BIM. It gives recommendations for briefing for design and construction in relation to energy, telecommunication, transport, water and other utilities' infrastructure to ensure that design takes into account the expected performance of the asset in use over its planned operational life. It is applicable to the provision of documentation supporting this purpose during design, construction, testing and commissioning, handover, start-up of operations and defined periods of aftercare.

PAS 1192-2R—specification for information management for the capital/delivery phase of construction projects using building information modelling and PAS 1192-3R—specification for information management for the operational phase of assets using building information modelling are in due in 2018.

ISO19650 1:2018 organisation and digitization of information about buildings and civil engineering works, including building information modelling (BIM)—information management using building information modelling—part 1: concepts and principles.

However there are also content, digitization, interoperability and collaboration that are figured in different maturity stages that could enable new business models in BIM and beyond. According to European Union Report [38] the levels are presented in **Figure 4**.

The above has been applied in the United Kingdom on the basis of keeping consistency among content, digitization, interoperability and collaboration in different maturity level during the project management life cycle. Moreover if is



**Figure 4.** BIM maturity level adopted by building information modelling (BIM) standardisation Martin Poljanšek (2017).

required to see from its global implementation perspective according to Autodesk Resources [39]:

*The **European Union** Public Procurement Directive in 2014 encouraged all member states to adopt BIM to increase value on public projects; **The UK BIM** Mandate will be in force in the Spring of 2016 for all centrally funded public projects in England; **France** have appointed a Digital Construction lead for the Ministry of Housing and announced a National Digitisation Plan including promotion of BIM; Germany's Construction Reform Commission has established a BIM Working Group to develop a BIM strategy for Germany and increase BIM adoption on projects; **Austria** has a published National BIM Standard. Though in infrastructure the **Environment Agency** (EA) in the UK is determining its supply chain BIM data requirements throughout project delivery; **Highways** England is running a number of BIM pilot projects to improve design coordination, project team collaboration, stakeholder engagement and project delivery; **Finland's** road authority has stipulated supply chain data submissions will be in LandXML InfraModel 3 (a structured data format for civil engineering) to enhance asset data records and **The Swedish** and **Dutch** transport agencies (Trafikverket and Rijkswaterstaat) have initiated a European Commission funded project 'V-Con' on BIM for roads standardisation and implementation.*

The importance therefore in the use of Digital Construction in Architecture, Engineering and Construction (AEC) forces the education to adopt similar strategies into the curriculum [40].

## 7. Future of tunnelling construction

CoSpaces is an IP project funded by the EC under the IST Programme of the FP6, which has the overall objective to develop organisational models and distributed technologies supporting innovative collaborative workspaces for individuals and project teams within distributed virtual manufacturing enterprises. Thus, the use of CoSpaces and similar technologies for tunnelling construction projects will enable effective partnerships, collaborative working culture, promote innovation, improve productivity, reduce the length of design cycles and take a holistic approach to implementing production phases. Example: Video 1 available from (can be viewed at) <http://cospaces.org/demonstrators.htm>

This will be achieved through enhanced human communication, innovative visualisation, knowledge support and natural interaction and will transform the current working practices to be more competitive in the global market. CoSpaces proposes to validate these collaborative workspaces against three sectors: aerospace, automotive and construction. However, the impact of the technology will go beyond these three sectors due to the generic nature of the technologies. CoSpaces will undertake the ambitious challenge of developing the technical, organisational and human networks to build collaborative workspaces. This will be achieved through a systematic and integrated programme of RTD activities, dissemination, training, demonstration and exploitation activities, led by a consortium of European experts who are committed to this mission.

## 8. Digital construction and businesses

Additional reason for the use of digital technologies is to support an integrate processes and different industries in a way to meet clients' requirements. This integrated environment will allow stakeholders to share data, information and knowledge in a way to be efficient and effective during the project life cycle even

when the product is in use. It is generally accepted that the client would be benefited by the use of advanced design and construction methods by understanding holistically how the final product would like. Henceforth, it will be easier to make any alterations in early stage and/or to assure that the product will be reusable in the future. Studies show that the impact of integrated collaborative technologies on team collaboration is to form a collaborative culture in all stages of construction projects. This collaborative culture allows stakeholders to use these technologies to enhance team collaboration. For example, (virtual) meetings could help to pre-identify clients' requirements, hidden costs and project risks during all stages of the project that are needed, to be proactive. Moreover, stakeholders in this collaborative environment will have the capacity to design a competitive procurement strategy. This strategy will help them to run the project smoothly, eliminating risks and mapping clients' requirements to projects output and outcomes too. Beyond the added value of running virtual meetings and designing a competitive procurement strategy, the collaborative culture allows stakeholders to improve accuracy, sharing and access to project data and information remotely at any time, enhancing well-being and productivity. In addition, the collaborative culture can assist to develop trust among stakeholders and improve the control all project stages. So, considering that a collaborative culture could be generated by stakeholders to improve the design, delivery and hand over of a project through collaboration, then it is also required to identify how project performance could be improved.

## **9. Chapter summary**

As it can be seen the application of BIM and digital construction in tunnelling engineering is vital in order to ensure consistency among content, digitization, interoperability and collaboration in different maturity level during the project management life cycle. Moreover to establish a collaborative culture enabling the next generation of tunnel project and asset managers. In addition, smart tunnels could support the society and make accessible communities where people are able to improve day-to-day life significantly.

## **10. Case study**

### **10.1 TfL's BIM application on a tunnel relining project in an operational environment**

Building information modelling (BIM) is a complex business process that has the potential to enable asset owners to achieve better control over their projects and assets, offering benefits throughout the asset life cycle. Many governments are now demanding that large public facility agencies adopt and implement BIM to improve delivery performance of the construction industry. Some governments have published BIM guidelines with most of these being technical specifications that are useful at the project level, but provide little support for the organisation-level adoption effort [31].

In the UK, the government published the 2011 Construction Strategy that mandated the adoption of BIM to BIM level 2 by April 2016 for centrally funded projects. Although not publicly funded, Transport for London (TfL) adopted BIM to deliver some of its capital investment projects. An example of BIM application at TfL is the award winning Bond Street to Baker Street (BS-BS) tunnel relining

project. This project was delivered in an operational environment, safely, ahead of schedule, under budget and with minimum service disruptions.

Transport for London (TfL) is an integrated transport authority that controls the day-to-day running of the metropolitan public transport network, which includes the London Underground (LU) railway. To provide a structured approach and guidance in the implementation of BIM, TfL has developed a suite of BIM documents for use by project teams delivering infrastructure projects.

The increasing population of London and the rising demand in railway usage demands an efficient transport system with minimal impact on the environment that allows for commuters to travel safely at affordable fares [31].

The LU network consists of 402 km of tunnels, which are a mixture of cut and cover sections and deep tube. Since the underground opened in 1863, London has been and continues to be shaped and influenced by its transport system, which is at the core of the city's economy. The LU lines are heavily used and some operate on ageing infrastructure (Mayor of London, 2015). Strategic network expansion is fundamental to meeting future demand and so is the need to keep existing infrastructure in good working order.

Strategic plans to deliver station capacity, signalling and rolling stock improvements are continuously reviewed, planned and delivered to minimise both disruption to the public and businesses and impact on the environment. It is also critical that the existing infrastructure is adequately maintained concurrently with the construction of capacity improvement projects. In the modern built environment where transport networks often use underground tunnels for transport networks, construction and maintenance of the tunnels is challenging and can be highly disruptive.

BIM processes, supported by advances in digital engineering techniques, are now making it possible to plan, design, build, operate and maintain cost effectively with minimum disruption. For built assets, the challenge is how to make use of the existing record information to support data driven asset management systems. The BS-BS tunnel relining project highlights the challenges of undertaking tunnel maintenance/repairs whilst maintaining the existing service. The use of BIM was fundamental in assuring stakeholders that the tunnel lining repair works could be delivered safely, on time, on budget and without impacting the train service.

Asset management processes could benefit from the rapid advances in technology that are increasing the capacity to handle large volumes of data at lower costs. The development of quicker data analysis tools and Artificial Intelligence (AI) are increasingly making data management the core of strategic planning for organisations [10] and influencing change towards data driven asset management. BIM adoption will help improve the quality of data in the asset information models and allow for predictive maintenance and ultimately lower maintenance costs.

LU is one of the oldest and most complex rapid transit rail networks in the world, with up to 5 million passenger journeys a day for the local population and business commutes. A strategic approach to asset management as guided by the ISO 55000 series is fundamental to asset performance. Periodic inspections of assets help detect deterioration of the asset condition and are used to inform decisions about remedial action required. With the increasing understanding of BIM, its application for asset management could contribute to reduction in maintenance costs and help organisations like TfL continue to provide affordable public transport.

Tunnels used for rapid transit systems are generally constructed using the cost effective cut and cover and rock tunnelling methods. Cut and cover tunnels in the built environment are more disruptive than the deep bore tunnels and have major

logistical challenges during construction and maintenance. Deep tunnels are more vulnerable to water incursions that may weaken the new tunnel structure. Limited space in the tunnels also makes it difficult to undertake maintenance tasks and major repair works often require the tunnel sections to be closed for long periods of time. For rapid transit systems, this can result in major service disruptions that can attract negative publicity and political pressure.

The award winning tunnel relining project of the 215 m section between Bond Street Station and Baker Street Station is an example where BIM delivered real value for TfL. Information used in this Case Study is based on data collected from company records relating to the specific project.

Constructed in the early 1970s using Expanded Pre-cast Concrete (EPC), defects on the single bore southbound tunnel section between Bond Street and Baker Street were first noticed in 2000. Tunnel monitoring and extensive investigation identified that acid in the ground water and the desiccation of the surrounding clay regions had weakened the tunnel section, causing EPC segments to crack and the tunnel to lose some structural integrity. This increased the risk of water ingress and partial collapse of the tunnel which may have resulted in the partial closure of the line for a long time.

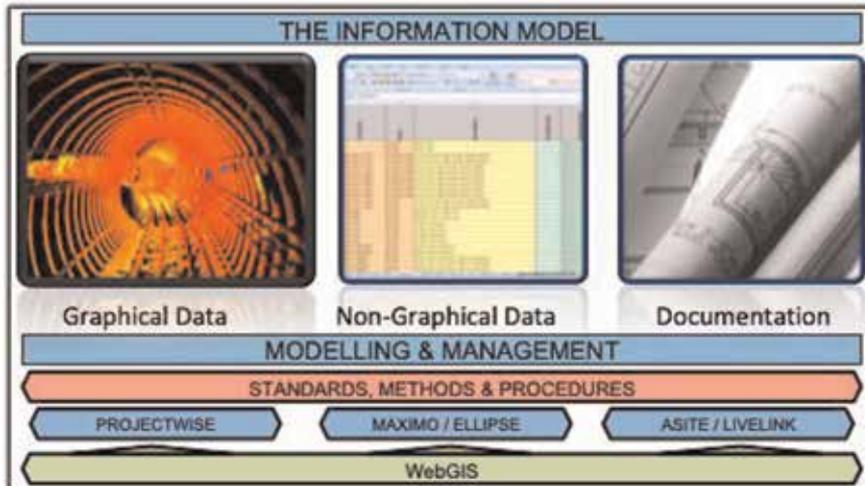


Closing the line would have had a huge impact on the transport network, disrupting TfL and businesses in the local area. Local repairs and comprehensive tunnel support works were carried out to make the tunnel safe while a long-term solution was considered. The decision was taken to reline the tunnel while the line remained operational and BIM was to be used to improve the delivery assurance of this challenging project. The key objectives for the project were to deliver the project safely, without affecting running the full service of 30 trains per hour on the line, to the agreed time line and cost.

## 10.2 BIM application

BIM processes, based on standards available at the time i.e. BS1192-2007 and PAS1192-2: 2013, were used to manage the information needed to design and repair the tunnel section.

Collaboration between teams involved in the project was instrumental to the successful delivery of the project as it allowed for standard methods and procedures for the production, storage, sharing and use of project data to be agreed early. A common data environment for graphical data, non-graphical data and documents was established as shown in **Figure 1**, to provide a single source of project data.

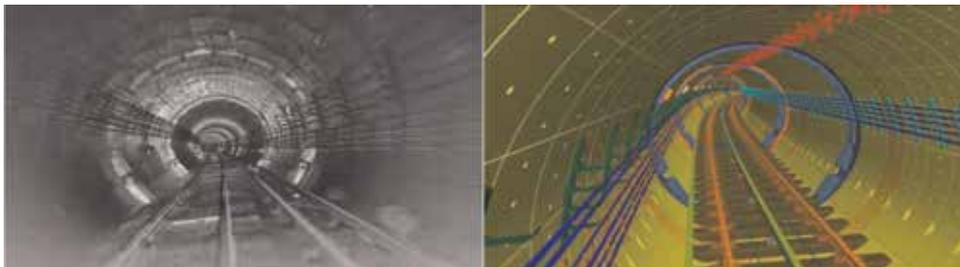


BS-BS project information requirements were defined early from the asset inspection records and helped the project have a clear and detailed scope. The project was delivered internally and it was the first time of doing this kind of work in an operational environment. Outsourcing the work would have been a challenge as it would have been difficult to define the scope for contractors and find the one with relevant experience. To minimise the risk a key objective was set to prove that the project could be delivered before site works began. This required design solutions for tunnel repair and plant together with the delivery plan to be developed and tested virtually as proof of concept.

The existing asset records for the project were in an analogue format and were not able to help create a 3D design model. So, laser scan surveys were conducted and the resulting point cloud data was used to create a 3D model of the existing tunnel. This was stored in Bentley ProjectWise, which provided a single source of graphical project data (**Figure 5**).

Using the survey model, 3D, 4D and 5D models were created. The 3D design model was used to design modifications of the train used for transporting materials and equipment to site and to develop bespoke equipment used to install prefabricated tunnel segment rings. The modified train was also used to remove waste material from site, as there was no room for on-site storage (**Figure 6**).

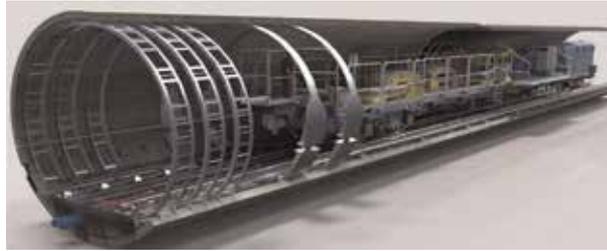
Most of the work was carried out at night when trains are not running, to minimise service disruption. The shifts had a typical on-site working window of 2.5 h



**Figure 5.**  
BS-BS laser scan survey and 3D design model.



**Figure 6.**  
*Model of BS-BS train. Source: <https://www.youtube.com/watch?v=eN2MBIfhxBI>*



**Figure 7.**  
*BS-BS 3D model of train in the tunnel.*

and the modified train with working platforms and carriages for carrying waste materials solved a major logistical problem and helped maximise output on-site.

A virtual reality model was created and used to demonstrate constructability of the project and was instrumental in convincing stakeholders that the project could be delivered against the set objectives. It was further used for virtual training of operators before they went to site, to improve safety (**Figure 7**).

### 10.3 Project outcomes

The design was completed and tested virtually before construction work started. This allowed the project team to plan and schedule tasks using 4D model and reduce delivery risks. Planning using the 4D model contributed to the successful delivery of the BS-BS project and meeting the key objectives of being delivered safely and without affecting the travelling public.

On completion in May 2015, the BS-BS project was delivered at £2 million below the initial budget of £34 million and 4 months ahead of schedule. The renewed tunnel now requires minimal maintenance and lessons learnt from the project will be passed on to future projects.

Key to the successful delivery of the BS-BS project was the implementation of BIM. BIM enabled better coordination of the project that enhanced the health and safety planning of such a complex project throughout the project life cycle. Lessons learnt from the project will be used to improve the delivery of similar projects using some of the tools developed for the project.

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# BIM and Advanced Computer-Based Tools for the Design and Construction of Underground Structures and Tunnels

*Panayotis Kontothanasis, Vicky Krommyda  
and Nikolaos Roussos*

## Abstract

Technology and digitalization are continuously producing changes in sectors and fields of human activities. Infrastructure industry needs this support in various and extensive ways, since it affects involved parties and society overall. Even though many individual branches have been transformed, design and construction show some kind of reluctance on encouraging and implementing comprehensive digitalization. A major reason is the significantly high complexity of infrastructure projects and the extended chains of work procedures and activities that are produced. All those are applying through the whole time scale of buildings' existence. Considering that safety and durability remain always the ultimate goal, every new method and concept shall be exhaustively tested, in order to prove its value and efficiency. The current chapter aims to define and prove technology contribution all along the infrastructure sector, concentrating in tunnels and underground structures. Since evolution is proceeding in accelerated rates, future perspectives are also analyzed to provide broader visions and set indicative standpoints for potential and incentives.

**Keywords:** building information modeling, tunnel construction, design tools, automatism, clash detection, decision-making, digitalization, disciplines, IFC, interoperability, tunnel monitoring, operation and maintenance, sustainability, semantics, simulation, virtual construction, artificial intelligence

## 1. Introduction

Tunneling projects and underground structures compose infrastructure projects including a variety of aspects concerning disciplines, scientific domains, faculties, required skills, implemented rules, and requirements. All those, as well as the included further features, are executed and applied through the entire projects' chain, that is, from the initial perception and planning, to design and development, realization, and building of structures, ending up to operation and maintenance for the whole life cycle. In order to accomplish the above, a vast variety of tools

and software and hardware items are used, aiming to combine and simultaneously fulfill all the scientific knowledge, standardized criteria, and respective codes.

The overall outcome consists of a complicated combination which is actually the core of many engineering projects. The differentiator factor is the fact that tunnel and underground structures are realized and function on a widely diverging scale (from km to detailed cm scale). More specifically, projects are realized and extended through multiple domains: survey and alignment, excavation and retaining measures, tunnel model and engineering, and detailed parts over the life cycle, such as boring machines, mechanical-electrical equipment, utilities, and many other components according to the tunnel's type and usage. Another distinctive side is the strongly interdisciplinary nature of tunnel infrastructure, resulting to a variety of specialties, stakeholders, suppliers, etc.

In combination with the uncertainty of ground properties and behavior, there is a clear necessity to detect and utilize all available and advanced design tools, with the aim to combine semantic, geometrical, and constructional aspects up to the final projects' accomplishment.

## 2. BIM and advanced design tools: definition from conception to development and future growth

Building Information Modeling (BIM) presents an infrastructure project in the form of three-dimensional representations of elements, which can be further associated with information about other characteristics and properties [1]. The created intelligent 3D model enables document management, coordination, and simulation during the entire life cycle of a project (plan, design, construction, maintenance, and operation). The evolution of technology-digitalization and the continuous progress in software and hardware equipment provided multiple capabilities to BIM. As a result, BIM has been converted from a design tool to a separate concept affecting all areas of engineering, and nowadays, it has been altered to define a whole industry applying to fields apparently irrelevant from engineering yet using the same technology and tools, focusing on similar goals, and sharing common inspiration.

It constitutes a main principle that *BIM as a term is not defining a specific and single software or process*. BIM is the *fundamental concept* which has absolutely dominated in infrastructure. *All possible branches of engineering and infrastructure could be effectively realized through BIM processes*. The three main processes *modeling, analyzing, and monitoring* are executed and integrated through BIM. Even at cases, where conventional methods are used, BIM provides methods and tools in helping to incorporate and use the available data and output in terms of structures' completion and integrity.

Initially, BIM started from 3D representations and gradually has ended up to communicating design intent in 7D terms. All dimensional aspects are defined and accordingly updated through the whole life cycle of a project (**Figure 1**):

*3D aspect*: Geometry, semantics, physical visualization, clash analysis

*4D aspect*: Time scheduling, project phasing simulations, activity progress, virtual construction

*5D aspect*: Cost-budget tracking, cost analysis scheduling, estimations for materials, equipment, man power

*6D aspect*: Sustainability, energy consumption analyses, infrastructure performance

*7D aspect*: Facility management, operation, maintenance, scheduling, project phasing simulations, activity progress, life cycle

Since BIM has dominated, several standards, codes, and terms have been established in order to provide rules and guidelines and to facilitate design and



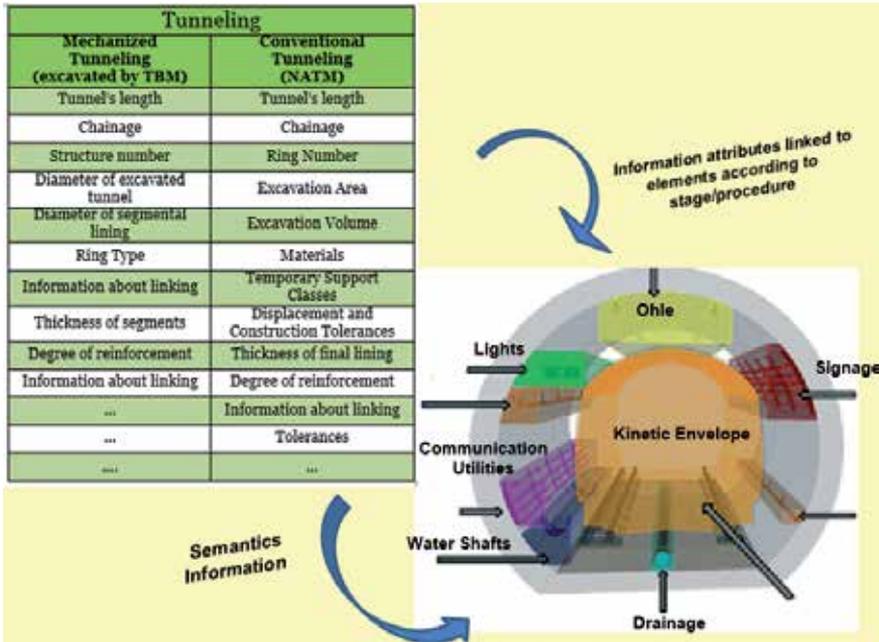


Figure 3. BIM level of information [3].

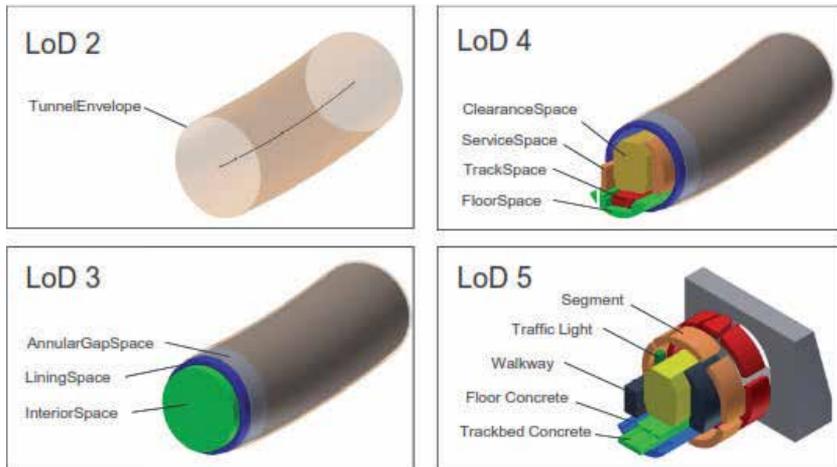


Figure 4. BIM level of development, shield tunnel case [1].

### 3. Why BIM?

#### 3.1 Tunneling complexity in terms of conventional tools

Through the years, many tools and products have been used for the realization of underground infrastructure projects. The efficiency and produced relations of quality, time, value, and integrity vary at each case accordingly. The continuous and significant development of software and hardware and upgraded technology are the regulators of growth. Obstacles are overpassed, and solutions are detected and applied for repetitive and common issues. However, available conventional

products do not support the multi-scale and multidiscipline aspects required to properly handle large infrastructure projects or smaller specialized ones. Moreover, the nature of underground and tunneling engineering is clearly distinguished by the degree of scientific data interpretation, which is spread along the time and size extents of the projects (Figure 5).

The successful execution requires the use, exchange, and “translation” of data in different formats, in order to be used as input, parameters, factors, and constraints. In the majority of cases, small differentiations, omissions, and lack of parameters have a decisive influence at design and construction, varying from favorable to conservative, according to the case. A quite common issue for engineers is dealing with data in a form difficult to be handled, such as reports, measurements, and observation results. There is a particular variety of input from boreholes, scanned documents, earthwork reports, mapping layers, photographs, geo-referenced files, etc. Additionally and according to each case, we have to include water designs and surveys, drainage network connections, road networks, underground and superficial deposits, linear features, and so on. Consequently, the overall performance could include disputable error margins, unclear parts, liabilities, underestimation of risks, or creation of fake ones.

In few words, the conventional and traditional tools, even at their most updated versions, provide us a theoretical simulation of tunneling projects, directing us to several debatable assumptions. The final results and output from reports to drawings and calculations are actually representations on the ideal basis that the taken assumptions are fully satisfied. This is due to the fact that engineers have to deal with a heavy use of 2D information and large volume of static documentation and descriptive data. Consequently, there is deviation from the real behavior of the structure, especially in the part of interaction between the real structure and the physical world. The necessity of a liaison concept is fulfilling and indivisibly connecting design, calculations, construction, monitoring, and so on has been forcibly revealed. BIM tools and procedures act determinedly providing not just particular solutions yet reforming and expanding procedures, strategies, and possibilities.

### **3.2 BIM's role between common and new challenges**

Isolation is the critical key to maintain data integrity and security, while linking is the productive key. BIM stands from its own definition to be the effectual key connecting those two concepts. This is the defining factor for the effective use of BIM on a project [4]. Translating and interoperability of information, while keeping a continuous access to the native form of data, and transformation from one software to another are basic parts of BIM's core.

In modern era, *value engineering* has been established as a major demand. Projects need to prove value and performance and achieve specific targets and rates from conception to operation and maintenance. Projects are no longer considered as single and separate entities, but they are incorporated into the wider economic and social environment, interacting with other structures—and not only during the construction phase. New needs of resources' savings are revealed, and new terms such as waste management, energy performance, etc. are introduced. Engineers are dealing with the process outcome and transformative business before even starting the actual work. More than ever before, we are asked not just to construct but also to deliver the services that a project is intended to provide. Associated risks and hazards are also transferred to engineering. The output and the overall footprint must be clearly *defined, measurable, and documented*. The realized design logic shall optimize a combination of tools and solutions, in order to address those outcomes.

To continue to further aspects, urbanization, failing infrastructure, and increased risk of natural disasters underscore the need for a stable, fit-for-purpose built environment [5]. Considering the contribution of infrastructure to the global economy, the produced energy footprint, in combination with required natural and human resources and respective produced waste for the realization of the projects, we are at a point where we have to include aspects previously considered as elaborate and exaggerated yet now totally necessary in order to be competitive and effective. Growing of population and incessant accumulation of people in urban areas are incrementally feeding the necessity of building new structures. Numerous work sites are running at the same time, requiring detailed time and cost schedules to be planned and actually followed without deviations. It would not be an overstatement to say that our world is a living evolving construction site continuously requiring updated tools and ways to exist and run.

Tunnels, in addition to the above, have the particularity to influence and interact both in-ground and underground conditions and environment. This interaction is dynamic, especially during the construction phase, and it consists of the critical concept, demanding a constant reflection at every single part of the life cycle. Thus, additional parameters and difficulties are created, definitely directing us to proceed further to nonconventional methods and procedures.

The fundamental strength of BIM entails on being a process that runs over the entire asset life cycle, providing a digital and actual representation of physical and functional elements, continuously contributing to decision-making. BIM proposes *a general methodology for creating and building multi-scale product models which combine semantic, geometrical, and engineering aspects* in a steady, coherent, and reliable manner.

Since tunneling is actually a link between ground and underground, BIM allows to be closer to an ideal final design, created and fit to frame existing site conditions and socioeconomic and environmental requirements and specifications. Besides tunneling, the design and building of truly complex, interconnected systems carry huge *risks*, unlike other industries and projects. A risk, not always noticed, is the one of delivering assets and systems designed and calculated on time schedules and prices at a time scale of years (or even a decade) ahead of the final product itself [5]. An additional risk refers to implemented techniques and materials' estimation and maintenance.

Another reason, which reveals the necessity of BIM implemented in advanced design tools, maybe even more than the building of new projects does, is the *monitoring, repair, and renovation of existing ones*. In those cases, there is an accountable amount of work, energy, resources, etc. consumed in registering the existing conditions and detecting deviations from the original design—if of course a full documentation is available. We are facing the consequences of data waste, that is, not using the data or recreating data through the project's life cycle. For engineers and parties, already experienced in BIM concept, the waste of time and energy is absolutely obvious, since they can easily identify the parts they could skip and already resolved if the design was generated by BIM.

In order to meet the already challenging demands and the newly created ones, the enhancing of automation capabilities of infrastructure software is a more secure way. This fact is strengthened by the level of automation and technology appearing in every aspect of the modern world; therefore, *we could not serve an upgraded project by traditional and conventional tools*.

There is also a *global direction* to incorporate tunnels in larger infrastructure, creating intricate complexes of tunnels, bridges, highways, roads, etc. In addition, many projects are designed with long-term visions aiming to constitute international and cultural points of reference. In such cases, the use of enhanced

automation is unquestionable, since it not only speed up the entire workload, but also removes a great amount of it—many times tedious and repetitive—enabling to focus energy to solve more complex problems with creative solutions. Constraints, rules, and challenges are no longer dictated only from engineering criteria, since we are addressed to a global and international market.

*The collaboration of modeling, calculation, monitoring tools, the integration of all software parts to a common environment, and the sharing of information in digital data are some of BIM's cornerstones.* BIM's technology provides the ability to quickly and cost-effectively capture information about the physical world and make it digital. As this technology is developing and being implemented in projects, we get closer to having a true digital mirror of our physical world [5]. This massive uptake of active collaborative data production demands not only to ensure that we do not neglect any part yet also to guarantee information storage, integrity, and security in our projects. In the world of cloud, mobile, and social connectivity, it is more than obvious that our tools could not remain idle ending up to be obsolete and practically useless (Figure 5).

### 3.3 BIM's role between common and new challenges

Focusing on future directions, many universities worldwide have already included training classes of advanced 3D design and 3D computational modeling. Educational institutions are the core of knowledge, research, and novelty, so it is quite evident that they should actually convert to pioneers of BIM development. Encouraging of investments, fostering faculties' collaboration, and orientated training should already been considered as top priorities. Future engineers must be prepared and skilled to deal with real and demanding problems. The entire direction of their education must be reformed and make clear that engineering faculty requires a wide diversity of knowledge and continuous edification.

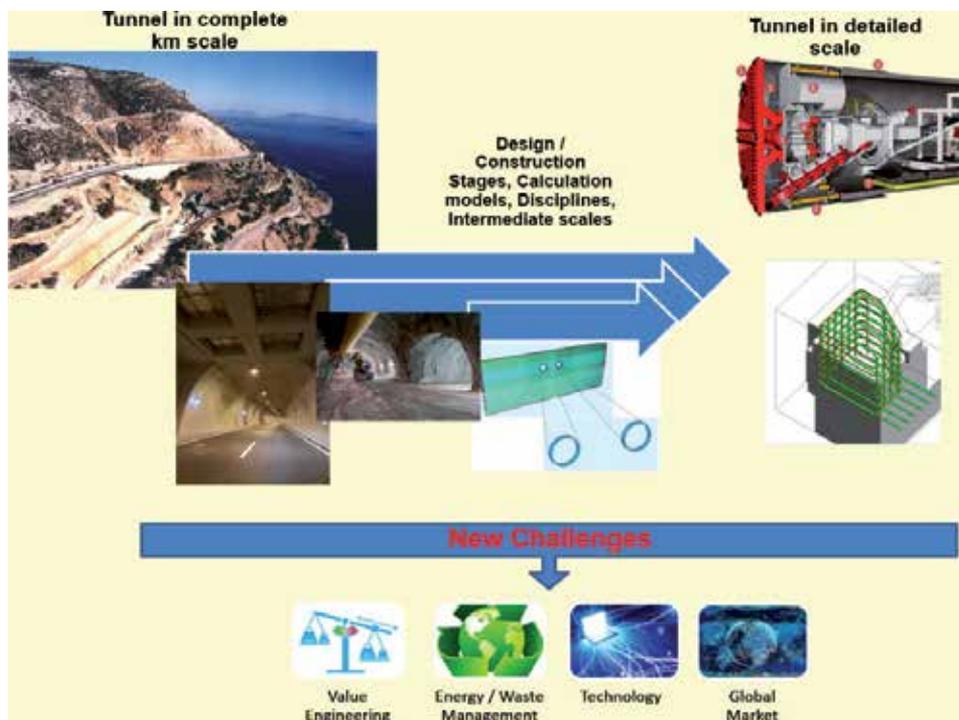


Figure 5. Tunneling design/construction demands—new challenges [6].

*The following are indicative references of software-specialized companies and tools, widely used in infrastructure projects for design, model, analysis, monitoring, etc.:*

Autodesk [7]: Revit, Civil 3D, Navisworks, InfraWorks, BIM 360, ReCap, 3ds Max, Fusion 360; Nemetschek: AllPlan, Bluebeam, SCIA; SOFiSTIK; BIMobject; Plaxis [7]; Bentley Systems; Leica Geosystems; AEC3; Trimble.

BIM stands as a unique concept fulfilling in a verified and upgraded level the rules of engineering in combination with the necessities of the present focusing to the future. The world operates and moves forward requiring the consideration and affiliation of enormous information amount, borders have been eliminated, and projects are performed internationally affecting not only a narrow society and area yet interacting in a global scale. This interaction is fully exposed to socioeconomic criticism, especially on the preliminary and building stages, since we have to deal with investment concerns and unproven performance of an unfinished structure. In our digital world, every engineering construction, regardless of location, size, or cost, is accessible even from mobile devices of affected citizens, at cases long before construction and even regulatory approval [8]. Considering that tunneling projects hugely affect people's daily life, having a direct impact in a time scale of many years, we are not allowed to reject any innovative tool. And above all, infrastructure has to stand solid, safe, and functional through the years, enabling people's prosperity and production of economic value. This necessitates that all involved parties and first of all engineers change the way they work and overcome worries affecting these goals and achievements.

#### **4. Benefits through procedures and workflows and uses and tools from point zero throughout structure's life cycle**

It consists of an undeniable fact that BIM's favorable and beneficial impact is not at all isolated in specific parts; however, it extends to every aspect of tunnel and underground projects from the initial inspiration up to the end of construction lifetime. The benefits apply to all involved procedures (modeling, analysis, etc.) and parties, directly and indirectly and similarly to BIM's philosophy of linking, integration, and cooperation; those benefits do not work separately, yet they are interconnected and interacting (**Figure 6**).

##### **4.1 Interoperability achievement**

To start with the physical and *geometrical aspect*, BIM creates an absolute and real three-dimensional representation of building components. The meaning of representation may not be so inclusive, since we are talking about a realistic visualization not only for the main structure yet for any other desired auxiliary or interacting part of structures and surroundings. BIM tools are not allowed by default to design with geometrical inconsistencies and errors. In tunnel and underground infrastructure, this is even more valuable, since arched, skewed, and complicated geometries are common. Saving of time and energy is large, not to consider the errors and discrepancies that are avoided. These advantages are even more enhanced considering the combination of all involved disciplines in each project. For example, mechanical and electrical parts are designed in an actual detailed level, in order to facilitate the subsequent construction stages. There is a variety of tools and software, executing clash detections, combining models of different formats into a single project model.

One of the main key words proving the change that BIM has brought to infrastructure is the *interoperability*, which throughout the years and during BIM's usage has been evolved to a whole concept for the engineering faculty. With the aid of

network servers, cloud services, etc., the design is executed in an integrated model, which combines all desired disciplines, design-construction stages, and the input from all involved parties. This model could be comprised from an unlimited number of other individual models, organized on a specific structure. Many teams and faculties are able to work on the same model simultaneously. All parties have access to a *common data environment* (CDE). This model sharing accelerates the internal coordination and boosts productivity and project development. The CDE is shared to all parties and stakeholders; therefore, at any time information is accessible to its updated status. We are actually dealing with a totally new and highly upgraded perception about structures' accomplishment.

## 4.2 Evolution of calculation tools

One of the key breakthroughs that BIM has brought in engineering is the full assimilation of structure's calculations with semantics, geometry, sequence, and any other aspect of the project. Analyses and computations are no longer treated as isolated tasks, yet they are continuously interacting with all parts, generating a realistic representation of structural performance. The geometrical model is directly used from the respective *computational calculation software for the analysis execution*. Upgraded computational tools include geometrical aspects and structural considerations enabling the interpretation and replication of constructive elements and their mutual interaction. Each alteration and adjustment during design-construction from minor ones to complete conceptual modifications are directly reflected to analyses, which are no longer error-prone to manual drafting updates. A direct impact of this linking is obvious in workload, and the required time is rapidly reduced in all stages, from preliminary to detailed and as-built design, including intermediate rework and modifications. Each involved party shares this benefit, and moreover there is not anymore a reason to hesitate for testing alternatives and different techniques. Integration of actual structure and engineering behavior promotes the scientific field, since engineers are more flexible and confident to conduct forensic analyses, enable multiple code reviews, and test many different failure criteria.

Calculation tools have been developed in all directions, offering a vast variety of options and libraries aiming to cover all cases, theories, criteria, etc.

*Element types:* structural components are completely defined based on geometry, stress-strain conditions and function, critical state, etc. (trusses, beams, interfaces, flat/curved shells, damping points, embedded components, plane/complete strain, anchors, tendons, springs, and so on). Engineers are able to fully define the overall function of the analytical element, in order to ensure the proper simulation of engineering response.

*Materials and computational criteria:* isotropic, homogenous, orthotropic materials. Self-healing concrete behavior and integration in construction. Temperature dependence and energy associated with material's shape. Evolution of Young's modulus in model codes, laboratory curved, or customized subroutine. Crack prediction in linear/nonlinear analysis. Maturity dependence of shear behavior, tension softening, and compression. Creep/shrinkage in transient mode. Material aging, plasticity, hardening, and hysteretic models for steel reinforcement. Viscoelasticity with temperature-dependent Young's modulus. Overall, physical/material properties, engineering criteria, and behavior are explicitly defined and fully incorporated in all computations.

*Analysis types:* upgraded software is equipped with powerful solvers in order to optimize solution procedures for all types of linear/nonlinear/dynamic complex models with accurate results and fast computations. By this way, engineers have also the option to simultaneously perform more analysis types finding the best

structural option and achieving a better understanding of design intent, yielding less errors and omissions. Besides the typical ones, more complex and time-consuming types are added: construction stage analysis, seepage steady or transient, drained/undrained analysis, saturated flow, consolidation, pressure-dependent degree of saturation, porosity, soil swelling, P-delta analysis for second-order effect, dynamic analysis, liquefaction, strength reduction ( $\phi$ -c), ground stratification from borehole data, use of relaxation factors to model body's 3D behavior during excavation, and frequency response.

A serious impediment that engineers are dealing with in calculations is the generation of an *accurate and representative mesh*. New tools use different input, hybrid mesh, and Boolean operations, generating 3D surfaces and intelligent node-to-node connections; simulating even small, very distorted fissure elements; and eliminating local imprecisions. These effects are extremely essential to obtain a consistent model, since errors, thin faces, and local inconsistencies lead to failures during the simulation, as model's continuity is not guaranteed. Using BIM, mesh follows structure's irregularities, and objects with complex geometry do not require excessive simplification. Complicated geometries, like intersections, junctions, caverns, elevated structures, etc., are not anymore resolved with questionable assumptions. Moreover, 3D meshing procedures of higher order displacement interpolation, 3D inclusive interfaces, and triangulate surfaces for faults and horizons from geological data are feasible.

Another typical however intricate task of calculations is the definition of *boundary conditions*. Similarly with other aspects, different types, values, and theories can be performed to investigate and conclude to the more realistic ones. The definitive advantage is that computational model could be directly compared with field measurements and gives us an assessment of model's reliability.

All those could be further developed using *event simulation* and time history analysis to simulate different stages/events during the life of the structure and model more realistically the stress state at any time, leading to identifying of potential deficiencies, which may cause damage or reduce performance. Phased analyses with load history and sequencing combined with dynamic, thermal, etc. loading-unloading, and material behavior determine worst-case reaction and stresses.

Since multiple cases and scenarios could be analyzed, the form of derived *results and output* acquires major importance. New tools provide useful options for automatically produced and updated diagrams, plots, etc., easily compared and providing possibilities, since it becomes quite easy to visualize the influence of design scenarios/changes across different iterations and isolate specific parts. Software could even provide interpretation of analysis and *solution procedures* through automatic solver selection. Based on the results, preliminary design of other disciplines could be generated, for example, 3D rebar models.

All of the above enable different and realistic decision-making, since engineers are able to identify critical stages, recognize problematic regions, and detect vulnerabilities. Initial hypotheses are tested, evaluated, and progressively adjusted to reach an actual consistence with experimental tests and realized construction. Physical-design-calculation models are interactively reflected from one to the other. Options like shape and material optimization, especially in reinforcement, are evaluated based on reliable and actual data. *The final tunnel infrastructure tends to be more close to the optimum balance between engineering behavior, safety, economy, and functionality.*

#### 4.3 4D and 5D influence joined with digitalization: IFC development

BIM's profits to the whole extents are realized through the incorporation of *4D and 5D perspective*. The nongeometrical and material attributes are interrelated through all processes. Besides, the created construction phases provide a real

visualization of the building sequence. The result is a solid and actual chain of the project's entities. Models of different format and discipline are linked, enabling 3D clash detection and 4D construction planning simulation, which allow a better understating of the project, enabling decision-making and efficient resolution of issues. Clash detections are set on a routine schedule, and the execution offers the ability to check and compare the actual design-site conditions at the entire time scale. Data segregation works simultaneously with data integration. It becomes also quite clear that the generated 3D, 4D, and 5D processes provide accurate and real data regarding materials, procurement, quantity takeoffs, and overall cost. In combination with the ability of BIM to elaborate construction drawings (e.g., shop reinforcement), quantities and costs can be extracted at any time, facilitating resources' management and site planning. Procurement is regarded as a part of a broader life cycle, rather than as a stand-alone process, and actually commences from the inception stage finishing when the project is delivered for management. All those are proved to be huge assets for companies and contractors, especially in tender stages. Time and cost records, which previously were considered as approximate estimations are now reflecting reality throughout the work procedure.

*Prior to construction:* BIM could act in a precautionary manner, reducing discrepancies and rework costs and preventing constructability issues. Especially, this last feature consists of a valuable asset for engineers, who quite often deal with serious technical issues on site, several of those requiring accountable time, effort, cost, and rehabilitation actions, not to mention cases where problems are irreversible affecting safety, quality, and overall performance of the infrastructure. Due to the project's consistent better understanding, unacceptable and unforeseen circumstances can be detected, analyzed, and resolved. Especially in tunnels, we must not neglect the size factor, which acts incrementally and affects every other part. This is why rectifying problems prior to construction progression is quite beneficial, since at an earlier stage, while corrective measures are still practical, the cost of all kinds is considerably less.

As soon as excavation works and support installation start, engineers are able to make an ongoing follow-up from the already performed simulation of the real construction phases. All federated models, documents, and calculations are continuously updated through all processes, in order to maintain consistent data and conclude to transparent, accurate, and reliable workflows. As a result, site responds quicker to design changes; solution adoption is more feasible, fast, and efficient; and project delivery is apparently improved. Especially in tunneling (e.g., NATM), where engineers have to decide and keep up based on the dynamics of building progress, they are more flexible and confident to test and implement different techniques and solutions. Efficiency is checked and confirmed by simulating the actual geometry with the actual encountered conditions. Unnecessary design and construction challenges are avoided by developing an optimal route and organizing construction sequencing focused on minimizing insecurity and ideally any deficit of infrastructure realization [9]. Furthermore, BIM is the key element to meet aggressive timelines and handle massive coordination. In our days we have to deal with largely extended projects, where a great number of firms, authorities, subcontractors, consultants, and people of various faculties have to work on the same line. This extraordinary level of coordination is practically unfeasible and non-affordable to be achieved with traditional and conventional 2D methods and procedures, even the most evolved ones.

Keeping up with BIM's profit enumeration, we should include the features of *digitalization and automatism* in infrastructure. Those features act with all the already mentioned aspects and benefits, enhancing in an absolute way all capabilities and offering a decisive boost to progress and evolution. Repetitive tasks are

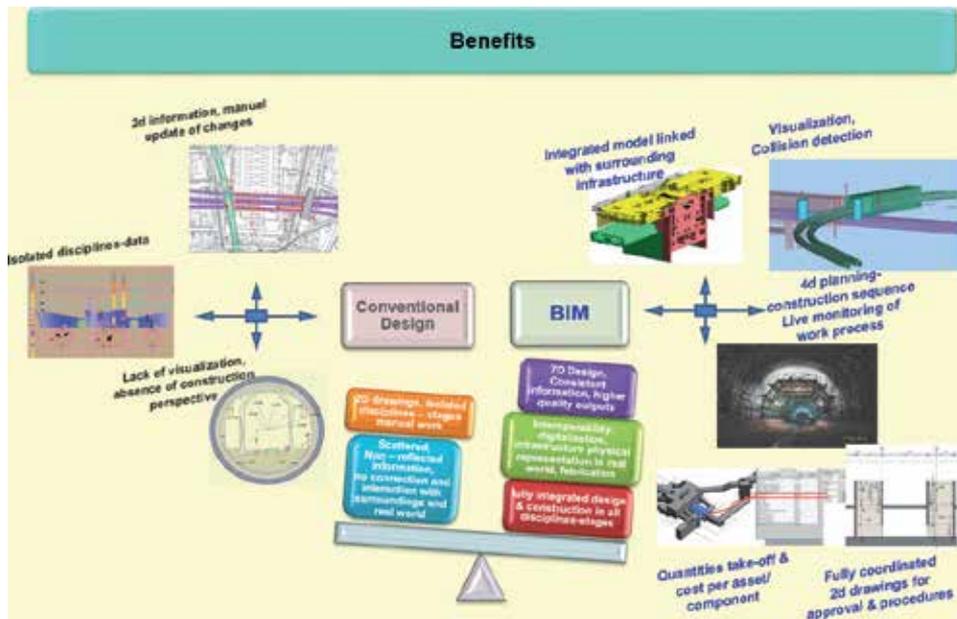


Figure 6. Conventional VS BIM design [6].

simplified and at cases even eliminated. There are plenty of ways and alternatives to build scripts and conduct routines for complex recurring tasks, quickly and efficiently. Visual programming tools help engineers to analyze and design data, standardize tedious workload, and aid in processing. Easy tasks are now executed automatically and the most complex ones in a faster and more accurate way. The exchange of input and results between design and calculation software formats—not necessarily compatible—occurs on a regular basis. Time and energy consumed on a specific software are saved from other parts of the workflows, since all kinds of information produced are circulated and used as an input. BIM by default enables the reuse of information generated during modeling and calculations, avoiding data duplication and inconsistencies which typically occur when different parts process the same input.

The most common “translator” used widely in infrastructure is the Industry Foundation Classes (IFC) [10]. IFC is an open common data format/structure transferring and decoding information. It works as an open data model schema for the definition of components’ geometry, physical, and engineering properties, providing a rigid and authoritative semantic definition of the elements and the produced associated relationships, dependencies, and properties. IFC is documented as an international standard, and due to its extended usage and proved efficiency, it often consists of a major requirement in projects’ contracts and standards. In this way, all involved faculties have a common language, which becomes even more valuable in our era, when projects are typically accomplished from people and accompanies of different countries. In this way the design is not reliant on a particular software. Moreover, information could be used and tested from one project to another, in order to compare and verify results. Modeling information exchange is targeted, working on the principle to only share what is relevant and applicable to specific activities and disciplines, using IFC as a parent data schema. In combination with other applications, components could be analyzed and monitored with the goal to improve performance in the entire range from engineering to operation and cost. IFC acts as catalyst, tightly interlinking the

processes and forming an iterative loop of communicated information in the flow of investigate-plan-design-calculate-construct-monitor-operate-maintain.

#### **4.4 Monitoring: risk assessment and hazard control**

A main task of a project's design which is also continuously present during construction stages and operation is progress and performance associated with *risk and hazard control*. All project decisions come with both short- and long-term implications and risks. The key to success is to understand the impacts and act in a precautionary way taking full advantage of *advanced monitoring*, using high technology equipment combined with scientific experience and correct engineering assessment. In *monitoring*, BIM can work with many digitalized tools and equipment, in order to provide measurements and results of the current condition. Moreover, with the use of the already elaborated models and calculations, we are able to compare results between the designed and the current state and consequently verify our estimations, in order to proceed to probable modifications and even prevent emergency situations. Gathering and evaluation of data are quite critical during temporary states and works. However, this has to be implemented without disturbing and dragging back construction progress. In our days, *drones and 3D laser scanning* resolve the issue of space and access, while they also provide trustworthy results which can be easily handled and exported for further process. Optical fiber sensors can be installed on any place without compromising structure's functions.

Monitoring tasks are not based anymore on single measurements, theoretical assumptions, and hypothetical cases. Upgraded tools have widely extended the fields and potential of monitoring. New devices offer the ability to measure both static and dynamic events and detect and filter fake measurements and temporary obstacles, while ensuring good temperature compensation, which is a must, for facing several environmental conditions. Strain gauge-based sensors have an advantage of high long-term stability, being operational during the whole lifetime, without needing recalibration. Crack detection and crack shapes are realized on a reformed basis, and in combination with calculation software and hardening concrete models, we have the valuable asset to predict cracking. The range of collected data is spread on multiple fields and in all construction phases, such as measured and expected displacements, loading of shotcrete lining, surface settlements and spatial distribution, ground deformation in the area of structures, allowable distortion or curvature in the expected influence area of the underground construction (buildings, railway tracks, gas-water pipes, wastewater sewers), corrosion and fatigue of concrete, reinforcement, and any material. Laser scanning delivers accurate and reliable complex cavities, openings, and as-built plans, allowing performance of exact volume calculations and quantity surveying tasks, monitoring of construction, and detection of narrow areas in advance. The total outcome enables engineers to identify structures' "normal behavior," detect deviations on time, and assess and predict all types of displacement development and ground conditions. A live and continuous comparison of designed, predicted, and measured data is feasible, and any party has easy and transparent access. In special and accidental cases, as earthquakes, smart sensors study the resonance behavior, in order to better predict structural performance. Sensor technology, combined with seismic and time history analysis running on as many ground motions as needed, provides response histories and maximum global seismic demands solely based on sensing results without making any finite element model. The affiliation of records with BIM models can also conclude to suggested design alterations and/or corrective measures.

Using those records, a series of cases can be detected, and workflow efforts can be managed to mitigate risks. Benefits of reliable risk and hazard control apply

to construction workforce, as well as to project operators and maintainers and of course to users and general public. Dangerous activities in use and operation are recorded and handled. Critical equipment during work execution is protected properly, and the planning for emergency and alarm situations is realistic. Specific feasible plans and schedules for managing construction and functional hazards are conducted and timely implemented. We are able to define substances and components hazardous to safety and health. The goal for monitoring project structural health is to form a database for tracking the behavior of structure and avoid any potential deterioration in safety and performance (bearing capacity, stiffness, serviceability, durability). This whole sector is quite critical, not only because it is required by legislation, yet it applies on the essence of engineering that sets as a first and central priority the assurance of safety and integrity.

#### 4.5 Maintenance and operation aspects: productivity growth

*Maintenance and operation* are features systematically neglected in infrastructure sector during the design and construction stages. We end up consuming great amount of money and energy on those through the life cycle of projects, and at cases those costs even exceed the cost of design and manufacture! BIM provides a series of procedures to manage those issues. The accurate costs, demands, and activities can be planned and calculated in advance, interacting precisely with projects' development, qualifying optimization design against future demands. As-built models and centralized data systems remain at the disposal of projects' operators, subject to revisions. Renovation of existing structures is released from inconsistencies, becoming a feasible and functional solution. BIM can be leveraged in the entire construction network management. By using the information in external data sources, the optimal distribution of capital, time, and resources is plausible to meet defined objectives [4].

It consists of an undeniable fact that the application of BIM philosophy integrated with advanced tools affects *productivity* in a positive way. Each one of the mentioned aspects and benefits has a direct or ancillary impact in job performance and productivity. The more easy way to modify and revise the design, while ensuring that alterations are communicated and shown at all respective deliverables and disciplines, becomes clear to every BIM user from the very start of application. This is enhanced considering the new ways of dealing with repetitive and tedious tasks. In general, the ability to communicate design intent and ongoing work progress, associated with the continuous access on actionable records of project's current and foreseen status, promotes the boost of job performance. Time schedules are visualized, and suggestions for improvement are easily communicated in order to optimize sequence of activities. The meaning and value of collaboration and teamwork are apparent more than ever before and in a broader extent. Transparency is finally present in all procedures, enforcing hazard identification and engineering judgment and responsibility.

A common practice in engineering is using references and already realized projects as a source for already resolved cases and evidence of fixed issues. The truth is that engineering faculty has not been quite committed and diligent on saving and organizing the bulk of information generated up to the structures' accomplishment. Regarding maintenance and operation time, lack of data is even more evident, and even at cases, where records are available, they appear to be inconsistent with previous phases of design-construction. This issue is partially justified from the fact that before BIM implementation, retaining and updating the entire information of a structure in a secure and useful manner required time and resources, ending to unaffordable costs. Thus, a vicious circle has been created, since we are actually

constructing the same type of projects with a vision to be more advanced, without using past experience and acquired knowledge. In this major wound of successful infrastructure, BIM provides solution not just by offering a database yet by giving the option of *parametrization*, since no project could be identical to an existing one. BIM enables the easy, accurate, and functional creation of databases and libraries to include all models, input, output, and deliverables all through the life cycle. This is an innovative way to accelerate the design, without jeopardizing safety and quality, since the performance of existing structures is recorded. Moreover, databases are used as tools of further examination and checking and not as an automatism or magic solution used without judgment and evaluation. Each new project has the opportunity to be raised on upper rates of quality and performance. Civil projects appear to be standard; however, differences occur each time, even more when considering that we have to deal continuously with new demands, materials, etc. Advanced software formats enable the creation of parametric elements in all attributes, geometrical, nongeometrical, physical, computational, material, classification, and so on. Models can be built with certain constants and limitations, while enabling parametrization in other parts that could act in a dynamic and variable way. In practice, it is easier to use the right points and nodes to start the design, than creating something new from scratch [11]. In tunnels this applies from the generation of geometry, to load cases and combinations, excavation categories, implemented support measures, reinforcement, niches, utilities, and practically the entire range. We end up creating *pilot projects* by means of “intelligent” constraints and interdependencies between model elements. In combination with BIM’s ability to speed up the completion of repetitive design, projects’ accomplishment settles in new standards.

The constantly increased demands of modern world have raised the levels of project delivery. The large number of ongoing constructions on a worldwide basis and the high standards they are expected to achieve impose the improvement of all available applied methods. The factor of time tends to be one of the most important priorities and an indicator of success. Therefore, *prefabrication* has turned into a main asset. Computer-aided manufacturing is becoming a common practice. Using software simulating tools, engineers are able to create machines’ setup and procedures and analyze the whole chain of fabrication. BIM models can be converted and used for the manufacturing process, for example, milling and laser cutting. A quite representative case is the reinforcement of tunnels, where the design data can be fed directly to machine tools and link design with manufacture without needing any intermediaries.

To sum up, BIM works as a binding agent and ensures a constant and smooth alignment between those who design and construct a structure and eventually those who manage and use it. Enumeration and evidence of the acquired value could be further developed and specialized at cases. However, working and interacting even once in BIM environment can provide the best proof in a concise and practical way. Importance and revolutionary changes of BIM are self-justified and overriding. Value of science, knowledge, and experience find the best means and paths to be expressed, quantified, and implemented in infrastructure. Technology, automation, and digitalization act as conductors of this evolution, incentivizing all forces to reach successfully the final achievements.

## **5. Case studies: metro projects and underground structures**

An increasing request for the use of underground space has been fostering the tunneling industry during the years. In combination with renovation and repair necessities, there is an immediate demand for the progress and use of advanced

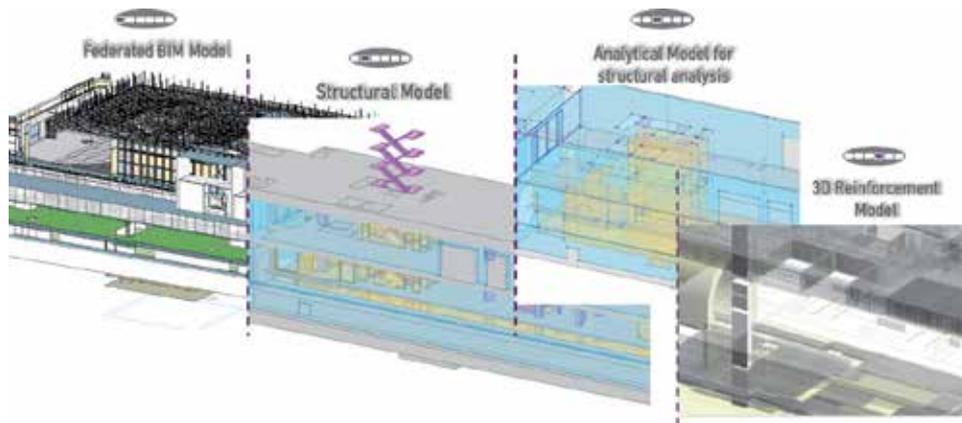
numerical simulation tools. Those urgencies are even more fed by an increasingly intensive interaction with ground structures, which necessitates not only a common and functional operation; however, it also reveals hazards and risks from the construction stage up to the whole life cycle.

Civil engineering is particularly risk-averse. Conservative nature has been deeply established in the entire industry, in order to balance uncertain factors and unpredicted conditions. BIM involvement in advanced tools of *structural and monitoring analysis* has already caused great difference in tunneling and implemented methods. Great underground projects all over the world have been successfully delivered in those terms [12]. Design and construction have been fully developed in 5D rules. This level has been conquered and consists of a prerequisite. The distinguished difference, which has been achieved in tunneling, is modeling linear structures joined with complicated sections (enlargements, shafts, junctions, etc.) in a background of ambiguous behavior regarding strength and deformation. *Representative cases of built projects and potential of 5D up to 7D development are mentioned in the following.* Some of them are in initial stages; however, the field is favorable and promising for quick progress [13].

Starting from *ground investigation*, an initial model is generated using the advanced monitoring tools—preferably 3D laser scanning—and locations for exploratory holes are identified. Real-time data gathered from the field are communicated to geotechnical laboratories, and after tests and process, the results are consistently introduced back to the *geotechnical model*. Tools are used to visualize this information, interpret data, and conduct reports. Besides verified and interpreted data, the digital model must also represent and use the state of uncertain knowledge. This is a crucial ability, which BIM offers comparing to traditional methods. Produced results assist to further refine the geotechnical model, and material properties are added to physical zones including also the time-dependent behavior within the model. In continuation, all geometry-engineering attributes are inserted as input into analytical software. Special purpose-models related to specific requirements can also be assembled to a coordination model. As the design is built and altered, the analytical results are automatically updated. Although complete digitalization is not yet feasible, data can be easily exported and transferred to the analytical software minimizing the need to retype. Laborious interpolation and extrapolation of the determined ground evidence and connection of individual boreholes to form strata boundaries are executed in a more advanced and secure way.

The produced comprehensive *3D finite element simulation* model reflects the relevant specifications and auxiliary construction measures, both temporarily and permanently. BIM cases incorporate soil improvement methods, ground freezing, saturated soil, grouting, retaining measures, special formworks, temporary props, etc. All techniques could be either designed initially from engineers or advanced tools could be used to simulate suggested alterations based on the results. 5D importance is more amplified due to the fact that support measures extend through several rounds. Models are built as in real construction, including portal areas, cross passages, launching structures, emergency exits, etc. Differences of excavation from the designed to the actual stage is clear. For the relevant tunnel types, temporary and final lining, deformable shield, and segments are simulated accordingly. Complex numerical analyses are overpassed via a unified, IFC-based product model, directly linked to the numerical simulation software, contributing to decipher and integrating the initially unrelated data. Instead of converting data and jeopardizing misinterpretation or loss of information, data models coexist and provide coherence and continuousness (**Figure 7**).

A special challenge in metro cases is the fact that critical decisions are made on a quite early stage, having a significant and ultimate impact on the direction of the

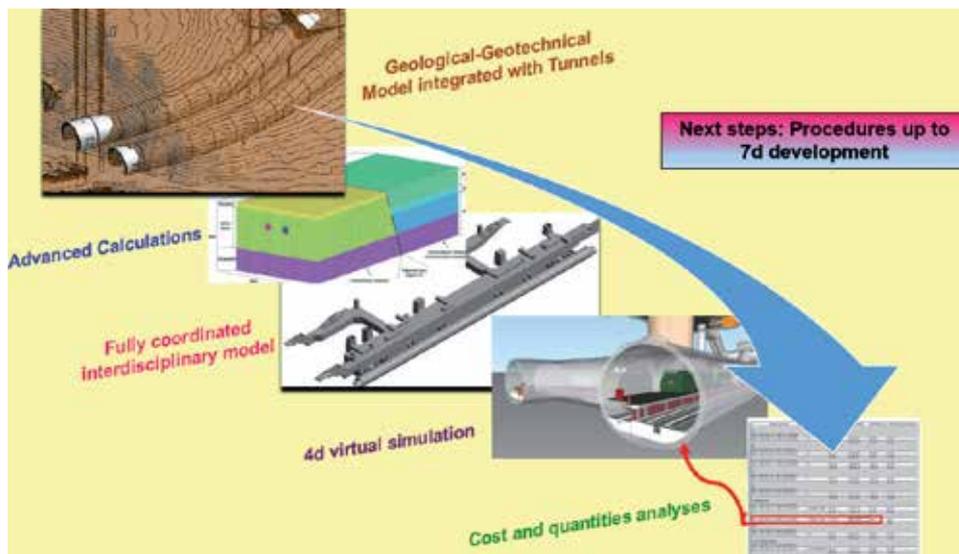


**Figure 7.**  
*Interdisciplinary model linking in metro cases and underground structures [6].*

final design. This is why the execution of *intelligent analyses* demands accurate existing condition data integrated to thorough semantic modeling. Tunnels must resist to the least favorable combination of parameters, so the produced interfaces must be capable of taking into account lower and upper limits of input ground parameters (E modulus, cohesion, lateral pressure ratio, friction angle, etc.). Calculated and forecast deformation and convergences are derived for the main as well as for ground and surrounding structures. On an early stage, this settlement effect can lead to alignment variants. Collision detection is performed, to detect clashes with existing and planned structures or fault zones in the ground. *A series of engineering calculations can be performed in a reliable way through the advanced tools, without simplifications used in the past.* Groundwater treatment and forced alterations from excavation works can be measured and reflected in results. The same applies for flood, traffic, sonic effect, noise protection, ventilation, smoke extraction, and evacuation simulations. Modeling the interaction of all those challenges offers to engineers and stakeholders various design scenarios, influencing the project in a definite way: extension of tunnel, shortening a trough structure, option of cut and cover, mechanized tunnel, etc.

To further establish the above, the *experience of complete metro and underground structures* consists of the best evidence. Modeling, on direction of 7D terms, is even more critical, since the project interacts from the beginning with urban and socio-economic environment and rapid adaptability has been a consistent demand. For example, it is quite common to deal with difficulties in finding convenient places for shafts and stations due to existing infrastructure, expropriation, and intervention in social life but also due to other parameters, such as crooked and intensive settlements during excavation works. Unique challenges could be also encountered, such as archaeological findings (**Figure 8**).

The reduction of time that the use of BIM has brought in workflow processes is even larger due to the size scale. Ground models cover an extended area through the alignment, which means that besides the initial process of data, accountable amount of rework is required for any alteration. Reality capture methods, advanced monitoring, and automatism in construction make all procedures of a project site fast and less painful. As the work progress continues, changes are imported and updating all models, calculations, and consequently results. 5D models are capable of representing the caused variability of scheduled dates and resulting cost consequences. This is very important, since a factor typically derails time and cost schedules are the repetitive variation



**Figure 8.** Metro cases and underground structure models in minimum 5D design [6].

orders, requested for a series of reasons such as clashes on disciplines and constructability (**Figure 8**).

*Operation and maintenance* in metro projects dominate the life cycle, since a typical period is about 80–100 years. Over the entire use, the nominal costs reach the magnitude of the initial investment. This is why a digital twin of the integrated model must be used to update all systems, components, and landscape. A detailed strategy plan for this purpose can be developed in advance, offering exact knowledge to operators, instructions for optimum facility management, simulations of structures' behavior, and renewal options. Occupational health and safety plans in metro projects are not anymore disregarded; on the contrary we have virtual reproductions of hazard analyses, escape and rescue routes, rescue facilities, visualization of accident risks, and access restrictions.

All mentioned and elaborated benefits of BIM concept apply and provide huge advantages in tunneling design-construction, monitoring, maintenance, and operation. As we are proceeding to a complete incorporation in tunneling, further ways and tools are tested, and we are able to have documented performance, remove impediments, and achieve effective risk assessment. We are not anymore forced to artificially set low benchmark by the inclusion of projects that fail to deliver value. The field of improvement in tunneling remains still wide and requires changes, investment, and encouragement of innovation. The attained gains are the best motivation to remove hesitancy, stop procrastination, and finally move tunneling in the new digitalized and interconnected era.

## 6. Future, vision, and targets: the new era is here waiting for us to respond

A revolutionary change that BIM concept has brought in infrastructure is that we are no longer imagining future as something distant. Evolution is at our disposal, and it is in our will whether and how we take advantage of it [14]. We are actually experiencing a reverse of the whole concept. In the near future, our needs will

dictate tools and procedures. Technology methods used in sectors irrelevant with engineering can work as an inspiration and provide solutions to infrastructure (Figure 9).

Since nowadays we are using point clouds, gradually BIM models will be entirely created and built from *reality capture data*. Aerial and object photographs, points from laser scanning, etc. will be converted to 3D models, and from this start all other parts shall be accomplished. Even for underground projects, drones and satellite images are valuable, since structures are always a part of a broader urban or suburban environment.

*Virtual construction* is closer than ever to be established as a common practice. 3D models will interact with construction schedule, planning, and phasing and are constantly updated with data received from the field, providing a cloud-based project. Regular photogrammetric surveys will track the building progress also detecting physical changes in underground-ground conditions enabling live monitoring of the whole complex, including earthworks and surroundings. Aerial surveying methods can provide safety, especially on hard to access or inaccessible areas, where conventional methods could be dangerous or impractical. Especially in tunneling, where the accuracy and adequacy of geological/geotechnical data are a top level priority, design and construction processes could be reformed at a great level. Construction management and decisions shall be based on real-world environment capture. Drones could also provide point clouds creating 360 degree photos, fly-through animation videos, and many other virtual reality experiences. Consequently, internal and approval procedures will be based on a totally different basis to convey the information. Submissions and documentation will no longer include hard copy deliverables. All means and devices, even tablets and mobile devices will gather, update, and convey information. Jobsite performance will be managed on a daily basis, and site conditions will take into account weather conditions to adjust the schedule and provide specific proactive measures. By scanning existing conditions, we will analyze requirements for future excavations, backfilling, etc., using also the produced accurate surface models.

*Job automation* is already here, and it will continue to dominate in infrastructure. It will transform older and existing industries and create new ones. Workforce will be adapted, so engineers will be interacting with technology more than ever in a technologically upgraded and sophisticated environment. Gradually, rework and redesign procedures will be part of the past, and the gained time could be used more creatively, to solve real problems. In fabrication, robots on site will be working side by side with construction workers. Workforce will be gradually moved from handling machines and equipment to handle and supervise software and input flow. All those will allow the implementation of unique techniques, since from the labor viewpoint cost and effort will be the same. *Augmented reality and virtual reality* (AR and VR) are an integral part of future evolution, and they are also entering infrastructure industry. We will get at a point, where AR and VR will work together, providing innovative experience and integrating the actual with the virtual aspect, since besides walking through the structure, we could experience pressure, temperature, materials, and so on.

Another technology asset, already in use, is *3D printing*. In the near future, 3D-printed deliverables will be a basic demand during workflows, since they could give with clarity the real perspective, overcoming limitation of visual angles. Combined with virtual reality tools, we could reproduce the actual structure. Besides that, 3D-printed components are already used in construction. With the use of robot machines, we will have the possibility to 3D print elements

of various materials, including concrete. Building of more complex geometries; reduced waste of material, time, and cost; and safety on site are only some of the benefits.

Regarding *hardware and software*, infrastructure industry is absorbing and adopting in an increased level the evolution and the generated enhanced possibilities. In addition, the dominance of *cloud services* will affect the whole sector and applied procedures. We will not spend time translating and processing data between different software, future cloud will be software free, enabling to capture, create, and edit information regardless the initial format generator. Another feasible potential is to produce multiple iterations in conceptual tools to reflect different design options or to implement modifications without having to remodel over and over again.

All the above will be integrated and conclude to a *radical reform* of concepts, strategies, and procedures. We could mention some *indicative examples*; however, possibilities and limits seem to be undefinable. For instance, imagine the typical case of a modification in our project. Regardless the extent and importance, it will not be just reflected in the model, yet it will act like a trigger, activating a series of necessary arrangements. All relevant parts will be notified, and with the aid of a proper project strategy, instructions and needful activities will be automatically initiated and communicated. We will deal with automatically generated markups in drawings and automatically updated calculations and even receive photos illustrating issues and already incorporating necessary solutions and actions. Suppliers and construction site will be informed for probable changes in their schedule, for additional required materials, etc.

The traditional 5D coordination will be moved in the initial stage of the design. Engineers will feed, for example, the software with load requirements and basic properties, and the algorithm will autonomously proceed to design and model structural, reinforcement, MEP discipline, and so on. Construction phases will be then created, considering all particular conditions and interaction with the real world. Material lists and quantity takeoff will be conducted in accordance with phases and time, transferred to possible suppliers, allowing price comparison and cost-effective budgets. Machines will use the models to start manufacture and building. Robotic cranes and equipment will receive the construction sequence, the same for prefabrication machines and factory suppliers. 3D laser scanners will monitor works, and the records will be constantly redirected to all parts by cloud services. A deviation from risk assessment will mobilize the needful processes. Imagine the case in a tunnel's site where a possible hazard automatically enables the alarm and evacuation actions. Certainly, possibilities extend to operation time, providing information for required repair actions and improving the plans of accidental cases. Sensor data from a tunnel, for example, will detect the initial forming of cracks and other malfunctions, which will be reported on time and efficiently resolved before the occurrence of failure. Through the whole life cycle, the idea is to *resolve the issue before it ever exists*.

As much revolutionary and innovative, this evolution on infrastructure appears to be the real tectonic shift will be made via *artificial intelligence* (AI) in the industry. Through AI, we will not design projects by machines, yet machine learning could enhance the expertise of engineers, by providing from the start the optimum design and construction solution. In general, structures are of specific types, built in certain environment conditions, and defined and restricted by engineering criteria, standards, and safety performance. What if we feed those rules to hardware and software, in order to obtain an optimal building and structural footprint? We will not analyze to get force and stress results, we will ask from the machine to provide us the structural system for the desired output. It might seem as a science

fiction scenario; however, AI is applied in many other sectors and soon will be partially used in infrastructure. Instead of conceiving the alignment of a tunnel and questioning for the best and feasible solution, AI will answer “which is the finest infrastructure complex and how the tunnel will be a part of it.” From this point, the options seem limitless. AI will give the alignment of the tunnel affiliating ground properties, connection with road network, traffic volumes, transportation requirements, hazard control, specific codes-standards, and future development. For metro cases, additional parameters would be the connection with other lines and transportation means, land acquisition for construction, required accesses, crowd simulation, surface conditions, etc. AI could provide solutions for all aspects: applied excavation categories based on geotechnical parameters, retaining measures, rationalized tunnel geometry per type, TBM segments, required reinforcement, even machine equipment properties (capacity-pressure), arrangement on site, and generally a whole organized construction sequence. During construction, AI could adjust the design on the encountered conditions. Overall, calculated and complete engineering solutions and decision-making could be realized through artificial intelligence. We moved from drawings to models and in the next step from models to systems, where a computer will provide outcomes based on specific attributes, which engineers can review, revise, and set in function.

We conclude, that in the era of connection, instead of questioning whether the project is designed right, we will ask whether this is *the right project from the first place*. Does the tunnel need to be widened? Will it address the expectations as those have been set? We are moving to the era of *generative design*, using automatism and computation to define, explore, and choose alternatives. New expectations require projects to deliver value, and future challenges are treated like they already exist. Infrastructure should acquire the inherent capability to respond and adjust in conditions and ways beyond the ones intended when they were conceived. Environmental issues, incidence, and effects of natural and human disasters have become a reality, and previously those aspects were even ignored in the elaboration of projects.

It consists of an undeniable truth that engineers have also to operate like problem solvers and innovators. All efforts should focus on accelerating the pace of change and evolution. At last there is no need to invent more innovations if we do not test and practice the existing ones. Knowledge and technology are present everywhere, waiting for our actions and response.



Figure 9.  
Future vision and targets.

## **7. Conclusion**

Advanced technology and innovations have really brought radical changes in the way we design and build. Acquired benefits are more than evident, especially in our connection era of overwhelming demands and necessities. We are not justified anymore to treat the parts of design, calculation, construction, monitoring, and operation as individual. BIM concept has created a solid circle to circumscribe all parts. We are moving forward in a velocity requiring continuous alert and flexibility. Instead of being idle, showing unjustified doubt, we should deploy a focused strategy with orientated actions. Engineering faculty shall invest more to research direction and promote faster application of upgraded tools. Engineers shall be active in these procedures, being able to identify and prioritize the emerging technologies and accelerate the integration among diverse sectors. The currently noticed different levels of progress adoption shall be eliminated through investments in skilled workforce, and universities are the point to start fostering the next generations of digital natives.

As much as machines dominate in our lives, the human factor remains the governing leader, pulling the strings. Scientific research and progress have always been the driving forces of engineering, and in the era of digital and accessible information, we could be more confident on setting ambitious and challenging visions and thriving.

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*Edited by Michael Sakellariou*

This volume presents a selection of chapters covering a wide range of tunneling engineering topics. The scope was to present reviews of established methods and new approaches in construction practice and in digital technology tools like building information modeling. The book is divided in four sections dealing with geological aspects of tunneling, analysis and design, new challenges in tunnel construction, and tunneling in the digital era. Topics from site investigation and rock mass failure mechanisms, analysis and design approaches, and innovations in tunnel construction through digital tools are covered in 10 chapters. The references provided will be useful for further reading.

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