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ITA/AITES Report 2006 on

Settlements induced by tunneling in Soft Ground

Presented by the WG "Research"
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Abstract

This document is primarily designed to inform participants directly involved in construction (owners, engineers, design offices, contractors, etc.). It is also to inform private and public decision makers, or even local residents, and to clarify the current misconceptions on the so-called "zero settlement promise" by giving a well-documented presentation on the admissible settlement concept.

This document serves as a first stage. It shall be revised in due course to provide methods for estimating settlement and provide damage criteria derived from experience. We may assume that with the support of owners, who are directly concerned with the consequences of their works, there will be considerable feedback from the many work sites under way at the time of writing these recommendations.

Introduction by Yann Leblais, Animateur ITA Working Group (Research)

For a period of several years The International Tunneling Association Working Group (Research) has considered the impact of tunneling beneath urban areas and wide-ranging discussions on the subject have taken place during the meetings. Whilst a general consensus view on the main issues and principles has been achieved, it is natural that there remains a variety of emphasis in the approaches and techniques adopted by Member Nations.

Further, because of recent progress in the ability of tunneling machines to cope with difficult ground conditions, the ground movements produced have been greatly reduced. Whilst the largely empirical predictive methods remain much the same, their application is constantly evolving as recent case history data becomes available. The Working Group is therefore considering the creation of a case history database of ground movements caused by tunneling from projects throughout the world. This may greatly assist contractors, designers and owners because the reduced impacts of tunneling will be quantified and projects considered more favorably due to the reduced impacts particularly when tunneling beneath cities. It is felt that a robust database demonstrating the improved ability of the tunneling industry to control ground movements would give owners added confidence in proceeding with the exploitation of underground space beneath our cities.

A Working Group of the French Tunneling Association (Leblais et al., 1996) has published a substantial and authoritative review paper 'Settlements Induced by Tunneling'. This paper forms the basis for this report together with some additions and revisions to reflect, as far as is possible, the comments received from representatives of the ITA Member Nations and discussions within the Research Working Group.

Acknowledgements

The general reporters are grateful for the assistance of the Research Working Group Animateur, Yann Leblais, Vice-Animateur, Yoshihiro Takano and all Working Group colleagues from Member Nations, as well others from many nations who have contributed to this report.

The data from the Channel Tunnel Rail Link are presented by kind permission of Union Railways (North) Ltd.

1. Purpose of the recommendation

Density of land usage is an important element of the construction of new public or private infrastructure. Similar attention must be paid in this respect to increasing demand from the communities for more areas free of road traffic. Both aspects contribute to more underground

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projects being undertaken, as well as an increased use of underground space. The construction of new underground facilities however will inevitably interfere with existing surface buildings and underground structures, given that no blueprint exists for actual underground land usage.

It is trivial to state that underground infrastructure must be constructed within the subsoil, and that the main uncertainties designers and contractors have to face in undertaking such projects relate to the ground conditions that will eventually be encountered during construction. Local residents and businesses may be affected by such works, be it during construction or in the longer term.

The response of existing structures to tunneling induced ground movements depends on their geometry, construction type and overall structural condition. This emphasizes one major unknown in evaluating the actual impact of underground works on existing overlying buildings, as there is usually little knowledge among property owners of the history of deformations experienced by the structure previously, and even less when it comes to building foundations.

The purpose of this document is to provide some clarification on the soil/structure interactions phenomena involved in the construction of underground structures (other than open cuts), as well as a review of the approaches developed to evaluate, measure, prevent and treat such effects, with due account of associated contractual issues.

The document is meant to provide recommendations on the way to approach settlements induced by tunneling in soft grounds.

On the other hand, it is not intended to be used as a tool for obtaining calculation recipes on foreseeable settlements. There are two main reasons for this:

– Evaluating settlements is principally based on engineering judgment and experience and remains an input from specialists.
– Research is still underway in this area within the international scientific community.

It should also be reminded that every project should be assessed on a case-by-case basis, using expert opinion, as well as available literature.

2. Tunnelling-induced ground movements

The relationship between surface settlements and tunnel depth is neither simple nor linear. In reality, ground movements depend on a number of factors including (1) geological, hydro-geological and geotechnical conditions, (2) tunnel geometry and depth, (3) excavation methods and (4) the quality of workmanship and management. It is however clear that a shallow tunnel will tend to have a greater effect on surface structures than a deep one.

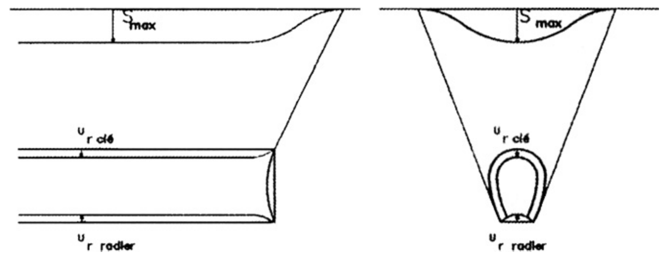

Fig. 1. Displacements of the excavation profiles: basic cross-sections.

The construction of a tunnel inevitably affects existing ground stresses and hydro-geological conditions. This modification of the natural stress conditions is typically accompanied by a rapid inward displacement of the face and convergence of the tunnel walls (Fig. 1). In soft cohesive soils, additional long term deformations may be observed as a result of pore pressure changes induced by the tunneling works.

The magnitude, orientation and the location of ground movements around the opening depends on the geotechnical conditions encountered, existing geostatic stresses and surface loads, hydro-geological conditions, as well as the techniques used for tunnel excavation and ground support. Where the strength of the ground mass is exceeded, significant displacements can be generated, both in terms of magnitude and acceleration. This may lead to the formation of shear planes within the ground mass, with detrimental effects in terms of required tunnel support (gravity loads) as well as limitation of ground movements.

Typically, the construction of an unsupported tunnel opening in soft ground would generate large ground displacements which, in turn could lead to the formation of a failure zone behind the face (Fig. 2a). In weaker grounds, the failure zone may propagate towards the ground ahead of the tunnel face (Fig. 2b).

A good appreciation of the risk for failure to occur at the tunnel face is essential, both from the standpoint of providing a safe working environment and evaluating the probability for large settlements to occur, given that ground movement at the face accounts to one major source of tunneling induced surface settlements.

2.1. Face stability

Analyzing tunnel face stability provides an indication of the most probable failure mechanisms, as well as of

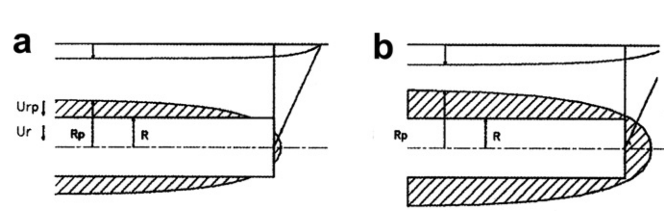

Fig. 2. (a) Yielded zone rear of the face. (b) Yielded zone ahead of the face.

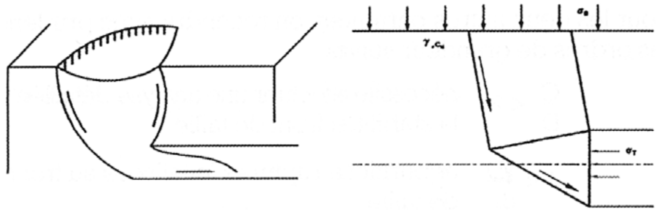

Fig. 3. Face collapse: basic diagram in cohesive ground soils.

parameters to be taken into consideration in the evaluation of ground movements induced by tunneling. Based on the nature of the grounds encountered, two types of failure mechanisms may be observed.

In the case of cohesive soils (Fig. 3) face failure involves a large volume of ground ahead of the working front. This mechanism leads to the formation of a sinkhole at the ground surface with a width larger than one tunnel diameter.

In the case of cohesionless soils, failure tends to propagate along a chimney like mechanism above the tunnel face (Fig. 4).

Both mechanisms have been evidenced in centrifuge tests carried out in clays (Fig. 3) and dry sand (Fig. 4).

Such conclusions are consistent with the results provided by theoretical studies (Chambon and Corté, 1989, 1990; Dormieux and Leca, 1993; Leca and Dormieux, 1990, 1992; Leca and Panet, 1988) as well as field observations (Clough and Leca, 1993). They are however based on the consideration of idealized conditions and should, of course, be adjusted to account for the actual conditions found on each individual worksite: non-homogeneous grounds and water inflows. In particular, in water-bearing sands, ground stability will be considerably influenced by hydraulic gradients induced by seepage towards the face.

It is also worth mentioning that the mechanisms shown in Figs. 3 and 4 refer to failure conditions and reflect the general trend for ground deformations at the face rather than the actual pattern of tunneling induced displacements.

2.2. Propagation of movements towards the surface

Ground movements initiated at the tunnel opening will tend to propagate towards the ground surface. The extent and time scale of this phenomenon will typically be dependent upon the geotechnical and geometrical conditions, as well as construction methods used on the site.

Two propagation modes have been identified, based on the conclusions of in situ measurements and observations. These modes can be used to evaluate, in a transverse plane, the degree of propagation of displacements initiated at the opening. They will be referred to, in the following, as primary mode and secondary mode (Pantet, 1991).

The primary mode (Fig. 5) occurs as ground stresses are released at the face. It is characterized by the formation of a zone of loosened ground above the excavation. The height of this zone is typically 1–1.5 times the tunnel diameter and about one diameter wide. Two compression zones develop laterally along the vertical direction. For deeper tunnels ($C/D > 2.5$), the observed tunneling impact at the ground surface is generally limited (Cording and Hansmire, 1975; Leblais and Bochon, 1991; Pantet, 1991).

The secondary mode (Fig. 6) may occur subsequently, when the tunnel is located close to the surface ($C/D < 2.5$) and insufficient confining support exists. These conditions result in the formation of a 'rigid' ground block, bounded by two single or multiple shear planes extending from the tunnel to the surface. Displacements at the ground surface above the opening are of the same order of magnitude as those generated at the opening.

These ground response mechanisms typically lead to vertical and horizontal displacements that tend to develop

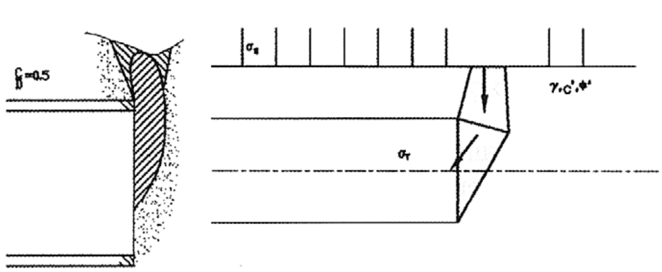

Fig. 4. Face collapse: basic diagram in dry granular soils.

Fig. 5. Primary mode: basic transverse cross-section.

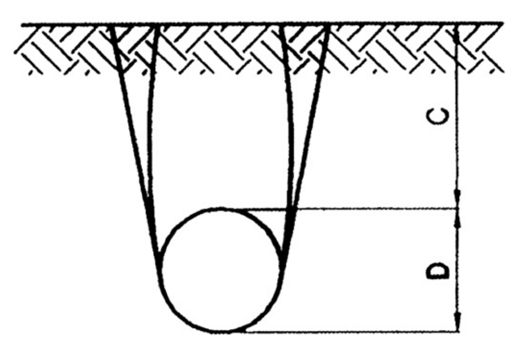

Fig. 6 Secondary mode: basic transverse cross-section.

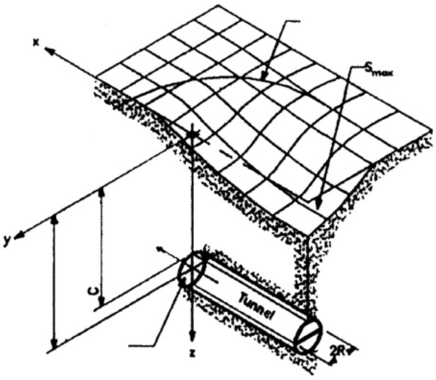

Fig. 7. Three-dimensional settlement trough.

at the ground surface as excavation proceeds; this results in what is referred to as the settlement trough (Fig. 7).

For practical purposes, the observed three-dimensional trough is conventionally characterized by means of a transverse trough and a longitudinal trough along the tunnel center-plane.

2.3. Main parameters involved in the stability of the opening during construction

Regardless of the nature of the ground, the magnitude and distribution of tunneling induced surface settlements depend on the ground layering (e.g. alternated heterogeneous layers), deformability (in the short and long terms), induced ($K_0 \neq 1$) and structural anisotropy (strength and deformability). Of course, the ground response to tunneling will also be influenced by existing hydro-geological conditions on the site. For example, stability time will be dependent upon the ground permeability.

It is clear that a good understanding of the site's geotechnical conditions is essential for assessing these fundamental parameters. This emphasizes the absolute need for a high quality ground investigation to be completed [refer on these aspects to the AFTES recommendation "The selection of parameters and tests for the design and construction of underground structures" (AFTES, 1994)].

Theoretical and experimental works on tunnel face stability have allowed the identification of a limited number of key parameters that (together with seepage conditions) can be used to characterize the stability of the opening. These parameters are described in Fig. 8.

2.3.1. Purely cohesive soil (clay)

For tunnels in clayey grounds, the overload factor *N*, defined (Broms and Bennemark, 1967) as:

$$N = \frac{\gamma H}{S_u}$$

where *H* is the depth to tunnel axis, γ is the soil unit weight, and s_u is the undrained shear strength of the ground prior to excavation has been identified as the fundamental ratio for characterizing the instability of the face.

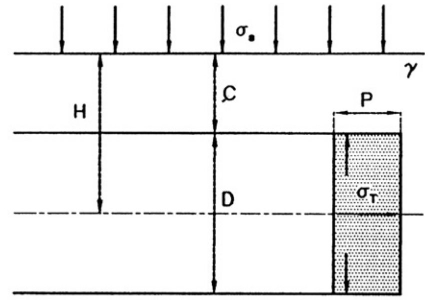

Fig. 8. Stability parameters.

Another two parameters also need to be considered: $\frac{C}{D}$ and $\frac{\gamma D}{S_u}$, where *C* is the depth of cover and D is the tunnel diameter.

The first ratio controls the effect of depth on the stability condition, while the second accounts for the possibility of localized failures to occur at the face.

In the more general case, where a surcharge is applied at the ground surface and a support pressure is used at the face, the overload factor, *N*, can be expressed as follows:

$$N = \frac{\gamma H + \sigma_S - \sigma_T}{S_u}$$

σ_S: surcharge acting on the ground surface
σ_T: support pressure applied at the face

Field observations (Peck, 1969) show that *N* values ranging from 5 to 7 typically result in tunneling difficulties and may cause tunnel face instability. Subject to more refined considerations, as indicated by experimental (centrifuge testing) and theoretical findings, it can typically be established that:

- when $N \leqslant 3$
  the overall stability of the tunnel face is usually ensured;
- when $3 < N \leqslant 6$
  special consideration must be taken of the evaluation of the settlement risk, with large amounts of ground losses being expected to occur at the face when $N \geqslant 5$;
- when $6 < N$
  on average, the face is unstable.

As for the other two parameters, the following general criteria can be considered with care:

$\frac{C}{D} < 2$
a detailed analysis of the face stability is required
$4 < \frac{\gamma D}{S_u}$
localized failure can occur at the face.

Moreover, special care must be exercised if the tunnel support is installed at some distance *P* behind the face, with face stability being dependent on the magnitude of the *P*/*D* ratio (Schofield, 1980).

The above parameters, which control the stability of the ground mass at the working face, may influence

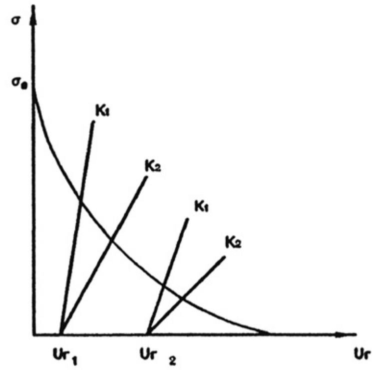

Fig. 9. Influence of support conditions (stiffness, installation time frame) on convergence.

surface settlements when the ground is subjected to stresses close to its shear strength. Some correlations have been established between the overload factor N and the magnitude of surface settlements (Clough and Schmidt, 1981).

2.3.2. Cohesionless soils (sand)

The face of a tunnel in cohesionless ground cannot in theory be stable. However, these ground conditions usually exhibit a slight cohesion that will influence the stability conditions, at least temporarily (e.g. capillary tension).

The factors of instability in such grounds are also more difficult to assess given that works on these structures are more recent. It must also be kept in mind that the propagation of ground motion towards the surface is influenced by other parameters such as the ground deformability and anisotropy (Lee and Rowe, 1989).

Theoretical and experimental studies relating to dry sands indicate that the tunnel depth (C/D ratio) is of lesser influence than in cohesive ground, whereas the tunnel diameter has a determining effect, with stability conditions being primarily controlled by the ratio, $\frac{\gamma D}{\sigma_T}$ and the soil's friction angle, φ'.

2.3.3. Cohesive frictional grounds

A more comprehensive analysis of tunnel face stability in a frictional, cohesive ground mass (i.e. with a strength characterized by a cohesion c' and a friction angle, φ') leads to four controlling parameters:

$\frac{\gamma H}{\sigma_c}$, $\frac{\gamma D}{\sigma_c}$, $\frac{\sigma_T}{\sigma_c}$ and φ'

where $\sigma_c = \frac{2c' \cos\varphi'}{1-\sin\varphi'}$

2.3.4. Rock

For shallow tunnels in rock, the ground strength is rarely reached as a result of stress changes induced by excavation. The present recommendation does not specifically cover the specific case of hard rock tunneling for which stability is primarily controlled by structural parameters (stratification, joint orientation and continuity, etc).

2.4. Convergence of the excavation

In addition to face stability, ground movement is also influenced by the convergence of the tunnel lining.

It should be kept in mind that one essential factor in reducing wall convergence is the early installation of a stiff support system behind the tunnel face, or even ahead of the face. This is clearly illustrated on a convergence-confinement diagram (Fig. 9), where it can be shown that a stiffer support system ($K_1 > K_2$) installed closer to the face ($Ur_1 < Ur_2$) will contribute to limiting the convergence, with more load being carried by the tunnel liner.

3. Causes for construction induced settlements

Before discussing the different approaches for estimating ground movements induced by underground excavation, it is desirable to review on the basis of the current state-of-the-art, the different causes of tunneling induced settlements. Prevention and remedial techniques will be addressed later in the document (Section 6).

Generally speaking, movements along the tunnel center-line are initiated at some distance ahead of the face and keep increasing until a complete support system is in place. Therefore one must differentiate between the settlements associated with the methods of excavation used at the face, and the settlements that occur behind the face.

Given the fundamental progress brought in this respect by the shield technology and associated developments, one must differentiate between continuous shield-driven construction and sequential tunneling techniques. The term "sequential" in the latter is preferred to "conventional" which is often associated to methods poorly suited to the control of settlements (ribs and wood) and do not reflect the richness of recent technical advances.

Settlements associated with groundwater and worksite conditions will be generally dealt with at the end of the chapter. It must also be mentioned that the following sections relate to the generic case of an isolated tunnel structure. For the purpose of simplicity, it has been considered preferable to focus on the basic principles, rather than addressing such specific conditions as that of side-by-side tunnel excavation (simultaneous or staged), so that no additional factor would influence an already complex situation. It must however be recognized that the latter may result in aggravated conditions as regards the impact of tunneling induced settlements.

3.1. Case of the sequential method

For works of this type, four major settlement sources can be identified:

– settlements associated with the stability at the face;
– settlements associated with the characteristics and conditions of installation of a temporary support system;
– settlements associated with the cross-sectional staging (sequencing) of the excavation works;
– settlements associated with the final lining installation and response.

3.1.1. Influence of tunnel face stability

Controlling the face stability is essential. Review of the latest developments on tunnel face stability clearly indicates a direct relationship between the control of face stability and the settlements induced ahead of the tunnel face.

3.1.2. Influence of the temporary support

The selection of an appropriate temporary support system is a key outcome of the project feasibility studies. This involves a compromise to be made between theoretical requirements and those imposed by construction methods considerations, and leads to assessing two fundamental parameters:

– the nominal stiffness of the support system which must account for its mechanical characteristics and installation methods.
– the time required for installing the support system which depends on the installation distance to the face.

These two parameters are used to evaluate the overall ability of the support system to resist ground convergence (Fig. 9) and, subsequently, limit construction induced settlements at the surface. Once the theoretical support requirements are determined, it is necessary to ensure that they can be achieved given the actual worksite conditions.

3.1.3. Influence of construction staging

Construction staging may strongly influence ground deformations around the opening:

– at the face, in proportion to the face area;
– at some distance from the face; this is dependent upon the ability to rapidly secure the tunnel liner, the staging of face excavation and length of unsupported tunnel walls behind the face;
– ground movement at some distance behind the face is further influenced by the distance of final liner installation to the face, as this structure is usually significantly stiffer than the initial liner and subject to less deformations; its early installation may also contribute to a more uniform longitudinal distribution of liner loads thereby limiting ground deformations.

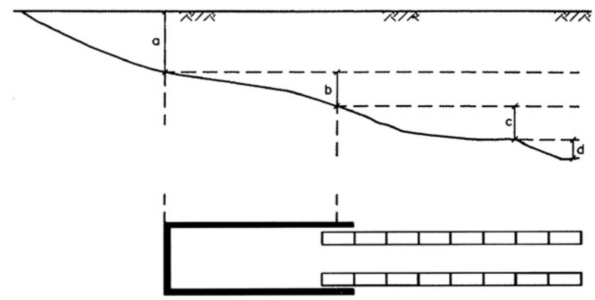

Fig. 10. Evolution of settlements along a shield.

3.1.4. Influence of the lining

The influence of liner deformations on ground movements must be taken into account, particularly in the case of large tunnel spans with limited cover.

3.2. Case of shield-driven tunnels

Settlements induced by shield tunneling can be broken down into four contributions (Fig. 10):

– settlements ahead and above the face;
– settlements along the shield;
– settlements at the shield tail skin;
– settlements due to liner deformations.

3.2.1. Settlements ahead and above the face

Settlements at the face are due to ground displacements ahead of (face loss) and above the shield towards the opening. Displacements depend on the level of confining support at the tunnel face (within the spoils chamber), the ground conditions and hydraulic conditions.

3.2.2. Settlements along the shield

Measurements taken above shield driven tunnels indicate that ground movements are rarely stabilized at the tail skin, and that the response time of the surrounding ground tends to decrease as the cover increases. The few existing observations of such phenomena tend to show that tunnel displacements propagate towards the ground surface at a constant speed for a given ground (Pantet, 1991).

Settlements along the shield may principally be caused by the following:

– overcutting induced by peripheral cutters intended to produce a slightly larger diameter to that of the shield, and thus reduce skin friction and facilitate guidance especially in tight radius curves;
– shield guidance difficulties, particularly in relation to its tendency to plough (dive), which usually requires the shield to be driven with an upward angle so that pitching can be prevented. Similarly, the tendency for the shield to yaw results in an excavated transverse section

which is wider than the shield section, thus contributing to widening the gap between the excavated and theoretical tunnel diameter;
– tapering (if any) of the shield;
– roughness of the cutting wheel that may, by friction and ground shear, induce crown settlements and ground movements ahead of the shield.

3.2.3. Settlements at the shield tail

At the shield tail, a gap develops between the ground and the outer face of the liner segments due to:

– the gap generated along the shield;
– the thickness of the tail-skin that varies according to the type (single/double) of shield and tunnel diameter;
– the clearance between the inner face of the tail-skin and outer face of the liner segment, to house the tail seal.

The magnitude of surface settlements depends on whether the tail gap is properly grouted.

It should be noted that these considerations typically refer to the case of segment installation within the tail-skin, and do not account for techniques such as the expanded liner segment method. This latter method may be of limited use for settlement control due to the level of stress release they produce within the ground.

3.2.4. Settlements due to lining deformation

Precast concrete segments installed within the tail-skin must be of sufficient strength to sustain the thrust of the shield jacks. As a result, the radial deformation of the liner rings is likely to be acceptable provided the tail gap is properly grouted.

3.3 Effect of groundwater

Numerous examples can be found of difficulties and accidents in underground works that were caused by groundwater. It must be emphasized that groundwater control is a prerequisite for the successful completion of underground works.

Settlements induced by groundwater typically fall under two categories.

The first category refers to the occurrence of settlements almost concurrently with construction.

Lowering of the groundwater table, prior to excavation (through drainage) or as a consequence of tunneling, may cause immediate settlements to occur in layers or lenses of compressible soils, as well as in weathered rocky materials. The impact of such lowering of the groundwater table varies in proportion to its magnitude and radius of influence:

– when localized, induced deformations are often prone to generate large differential settlements that can be damaging to the surrounding buildings;
– when widely spread, their consequences are generally less severe (Auber station, line A of the Réseau Express Regional (RER) – Paris express railway network, St Lazare railway station in Paris, Est-Ouest Liaison Express (EOLE) – Paris East-West underground link).

The occurrence of groundwater at the tunnel face may induce settlements as a result of:

– the hydraulic gradient weakening the mechanical conditions at the face and on the tunnel walls thereby increasing ground deformations;
– worsening effects on preexisting mechanical instabilities (washed out karsts, etc);
– worsening of the mechanical properties of the ground in the invert, particularly when the sequential method is used, with the risk for punching of the foundation ground by the temporary support due to loss of confinement.

The second category refers to delayed settlements that are typically observed in soft compressible grounds. As a result of the tunneling works, the ground can be locally subjected to stress increase and subsequently excess pore pressures. Similar mechanisms can develop at a larger scale with fully pressurized shield tunneling. Moreover, as a result of seepage towards the tunnel walls that inevitably occurs during and/or after construction, either along the more pervious materials present around the opening or through the tunnel liner, consolidation will take place within the entire ground mass. The magnitude of consolidation settlements will be larger in areas experiencing higher reductions in pore pressures.

3.4. Effect of worksite conditions

This includes the settlements induced by the general worksite conditions, especially vibrations induced by boring whether with the sequential or shielded method and muck removal operations. Settlements of this type have been observed in soft ground conditions, or in good ground with poor surface backfill material.

4. Evaluation of ground movements

4.1. Computation methods for the evaluation of ground movements around the opening

To date, the theoretical determination of the displacement field around a tunnel opening remains difficult, particularly when it comes to achieving a mathematical representation of the complex phenomena observed during tunneling, due to the large number of parameters to be taken into account and to the three-dimensional pattern of the ground motion around the opening.

The resolution of this mechanics problem requires the determination of the constitutive laws representing the fundamental behavior of the materials involved (soil, lining material and, when appropriate, grouting products). The influence of the soil's constitutive model on the determination of the ground movements around the opening has been demonstrated by numerous theoretical studies.

In France, the analysis of the convergence of the tunnel walls is completed using the Convergence-Confinement method (Panet, 1995). This method provides a two-dimensional representation of the three-dimensional deformation pattern around the opening by introducing a fictitious tunnel support pressure, the magnitude of which is adjusted in proportion to the stress release coefficient, λ. The magnitude of this coefficient is varied to account for the behavior of the ground at the tunnel face, the distance of installation of the support system behind the face, the construction method and quality of workmanship. The most recent developments also allow for the influence of the support stiffness to be taken into account.

The equilibrium reached within the ground mass, after it has been disturbed by the excavation works, can be analyzed using two conventional techniques (with the ground being modeled as a continuous medium subjected to external loads):

- analytical methods;
- the finite element method (FEM).

Analytical methods are based on simplifying assumptions in terms of geometry, ground layering (single homogeneous layer), selection of constitutive models and definition of boundary and initial conditions. Scientific literature provides numerous analytical formulations (Clough and Schmidt, 1981; Dormieux et al., 199; Rowe and Lee, 1992; Sagaseta, 1987; Yi et al., 1993). In most cases, the authors focused on defining the new stress field generated by the excavation; fewer works have been devoted to the evaluation of the distribution of ground movements around the opening and time effects, due to the complexity of such analyses.

On the other hand, numerical techniques such as the FEM take account of heterogeneous ground layers with more sophisticated constitutive models, as well as initial and boundary conditions similar to the actual field conditions, and time dependent effects. They are particularly effective for the study of tunnels excavated in grounds that can be modeled as continuous media, with due account of non-linear behaviors, as well as complex staging and geometrical conditions. However, three-dimensional analyses remain complex and the recourse to a simplified two-dimensional approach may be necessary, thus reducing the modeling potential of this technique.

4.2. Evaluation methods for surface settlements

With the exception of scale models that are essentially used for research works, two main methods are available for the evaluation of surface settlements.

4.2.1. Empirical and semi-empirical methods

These simplified methods consist in estimating surface settlements based on a limited number of parameters, which allow taking account of:

- the excavation size and depth;
- the ground conditions;
- the volume of ground loss or convergence induced by tunneling.

The simplest method consists in making a pseudo-elastic analysis, which allows to express the maximum surface settlement $s_{\max}$ as:

$$s_{\max} = K.\lambda.\frac{\gamma R^2}{E}$$

where K is the dependent on the ground stresses, ground conditions and tunnel geometry; λ the stress release coefficient; R the excavated radius; γ the average unit weight of the ground; and E the Young's modulus of the ground.

This approach is usually found to be oversimplifying for the following reasons:

- it cannot strictly speaking be applied to a shallow underground structure (given that the stress field around the opening can only be considered uniform when $H \geqslant 3D$);
- it does not explicitly take account of the tunnel depth;
- it establishes a proportional relationship between the magnitude of maximum surface settlement and the amount of stress release, which is often far from being backed by experience (Section 4.3.3).

However, this approach has some merit in that in allows identification of the fundamental parameters involved in the determination of surface settlements:

- tunnel cross section (R^2);
- ground deformation (E);
- construction method and workmanship (λ);
- experience factor (k).

In practice, empirical methods are most commonly used; these are more or less combined with analytical methods or finite element computations, and calibrated with data from case histories.

These methods are usually simple and allow parametric studies to be performed on the influence of the structure on surface settlements to be carried out. They are therefore particularly useful at the preliminary design stage and may be sufficient to fulfill all design requirements when site conditions are well known and design parameters calibrated accordingly.

This pragmatic approach was introduced by Schmidt (1969) and Peck (1969) and further developed in the United Kingdom, primarily on the basis of numerous studies related to tunneling in homogeneous ground in London Clay (Attewell et al., 1986; Kimura and Mair, 1981; Mair et al., 1981; O'Reilly, 1988; O'Reilly and New, 1982; New and Bowers, 1994).

4.2.2. Numerical methods

These methods aim at computing the ground displacements at every point within the ground around the opening. They take account of the characteristics of both construction and ground conditions (geometry, initial stresses, ground behavior, excavation stages, etc.). The most widely used approach is by means of two-dimensional FEM analyses in a plane perpendicular to the tunnel axis, which is consistent with the analytical approach and the Convergence-Confinement concept.

It should be noted that this approach can also be used to obtain an evaluation of the loads carried by the tunnel liner, and provides a powerful means for tunnel design, but its implementation remains relatively complex. As a result, although these techniques allow for a comprehensive determination of all design parameters, they are primarily used as simplified preliminary models, with the most refined models (accounting for all geotechnical, geometric and construction specificities) being restricted to a selection of key design cross-sections.

For shallow structures, these methods may lead to an erroneous representation of the impact of tunneling at the ground surface, particularly if failure mechanisms are involved. In particular, in cohesionless grounds, two-dimensional FEM models tend to distribute tunneling induced deformations over a wider area than that derived from field observations. This may result in overestimating the lateral spread of deformations and width of settlement trough, and subsequently underestimating the magnitude of maximum surface settlements. Research currently under way (soil behavior, initial stress conditions, full three-dimensional models, size of finite elements) should allow further modeling improvements in the future.

It must be kept in mind that large discrepancies exist between the apparent accuracy of the results derived from such powerful models and the poor level of appreciation of design assumptions, particularly as regards ground stiffness and construction staging. Hence, it is absolutely necessary to test the model's sensitivity to a variety of design assumptions so that (potentially serious) erroneous representations and subsequent disputes can be prevented.

As an example, the highest care must be exercised when introducing secondary constitutive soil parameters, such as dilatancy. It is further believed that, until a commonly accepted method of determination of such parameters is developed, and in view of the sensitivity of computational models to these parameters, adverse effects should be expected from their introduction. These effects can be expected to be enhanced by the excessive importance such parameters may be given as a result of the apparent accuracy provided by the elaborate numerical tools they are used with.

It must be noted that numerical methods allow, when necessary, to fully model the interaction between the ground, the construction works and the existing overlying buildings. The use of these theoretical models in the back-analysis of real case data can prove very useful in determining geomechanical parameters, calibrating empirical methods and interpreting data obtained from in situ monitoring.

4.3. Basic methodology for estimating surface settlements

The proposed approach consists of three main chronological stages:

(1) evaluation of the volume of ground loss generated at the opening (V_e);
(2) evaluation of the proportion of ground loss reaching the ground surface (V_s);
(3) determination of the settlement trough shape:
   - determination of the trough width (2B),
   - evaluation of the trough depth, i.e. the maximum surface settlement (s_{max}).

4.3.1. Evaluation of volume losses around the face

With the Convergence-Confinement method, the determination of the volume of ground loss at the opening (V_e) can be achieved by evaluating the convergence of the tunnel walls. Several analytical approaches are available for the case of a circular tunnel driven in a homogeneous isotropic material. These approaches also provide a reasonable evaluation of the volume of ground loss around non-circular tunnels using the equivalent radius concept.

With this approach, the key parameter is the stress release coefficient, λ, which accounts for the volume of ground loss developed at and next to the face.

In the case of the sequential method, this coefficient is varied as a function of the excavation and subsequent support installation stages.

In the case of a shield-driven tunnel, although a single overall value of the stress release coefficient may be sufficient for determining the lining thickness, a series of stress release coefficients should be applied to take account of the different sources of ground loss in the evaluation of surface settlements (Section 3.2). This is a delicate process, which requires sufficient feed-back from experience in order to calibrate the break down of ground losses on the basis of their incidence on observed settlements. Based on current knowledge, the following distribution of settlement contributions can be proposed:

– 10–20% caused by the face intake;
– 40–50% produced along the shield;

– 30–40% observed at the tail-skin.

In view of current advances in construction techniques and methods, and based on observations made on recent work-sites in difficult geometric and geotechnical conditions (extension to Vaise of Line D of the Lyon metro; Line 2 of the Cairo metro), it can also be established that:

– the magnitude of observed settlements clearly tends to decrease (10–20 mm);
– the above percentages of contribution to surface settlements tend to vary, with settlements at the tail skin now accounting for a smaller portion of the overall settlement due to advances in grouting technologies (Section 6.5.3). Recent experience from earth pressure balance machine (EPBM) driven tunnels also shows that the proportion of ground lost at the face may be greatly reduced. In some cases heave ahead of the tunnel face may be experienced.

4.3.2. Propagation of displacements towards the surface

The second stage of evaluation consists in determining the volume of settlement trough (V_s) induced at the surface or a given depth.

The simplest assumption consists in considering the ground as incompressible. In this case, the volume of settlement trough equals the volume of ground loss at the opening. This assumption is actually highly dependent on the nature and cover of ground above the tunnel. It is typically valid for shallow tunnels in cohesive grounds.

Whilst there exists few cases of increase in settlement volume, several factors can contribute to lower volumes of settlement being observed at the surface than those produced at tunnel level. These may include:

– large depth of cover resulting in up to 80% deformation dampening;
– the presence of a stiffer layer over the tunnel (bridging effect);
– the presence of a layer of dilating material in the tunnel cover (dense sand).

It is clear that each case history is specific and, as a result, it is difficult to provide a general relationship between the volume of settlement trough and the volume of ground loss produced at the opening. One can refer for more details to the abundant literature on this matter. Fig. 11 provides an example of such observation, derived from measurements made on a few French case histories of tunnels excavated with closed-face shields.

The time required for tunneling induced settlements to reach the ground surface and stabilize is extremely dependent upon project conditions. It is advisable for an appreciation of such phenomena to refer to the existing literature on case history data.

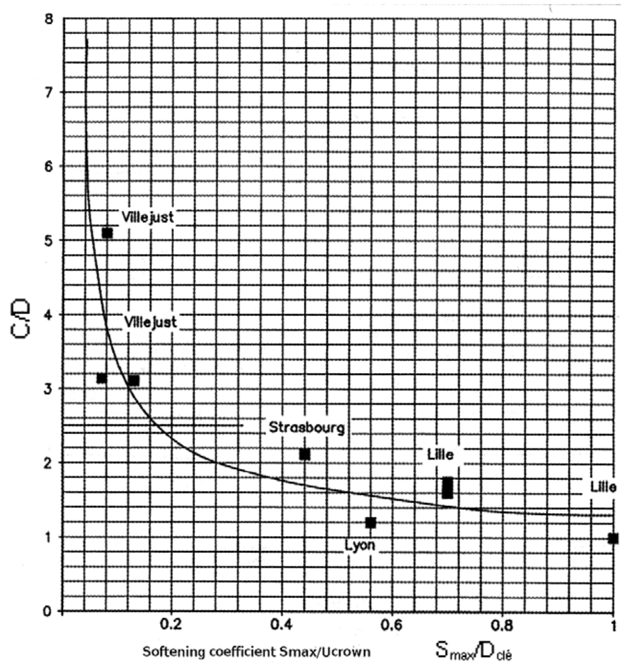

Fig. 11. Dampening coefficient vs. *C*/*D*.

4.3.3. Transverse settlements and displacements

The shape of the subsidence trough above mining excavations was examined by Martos (1958) and he proposed that it could be well represented by a Gaussian or Normal distribution curve (Fig. 12). Later, Schmidt (1969) and Peck (1969) showed that the surface settlement trough above tunnels took a similar form.

O'Reilly and New (1982) developed the Gaussian model by making the assumptions that the ground loss could be represented by a radial flow of material toward the tunnel and that the trough could be related to the ground conditions through an empirical "trough width parameter" (*K*). The model was guided by an analysis of case history data. These assumptions allowed them to develop

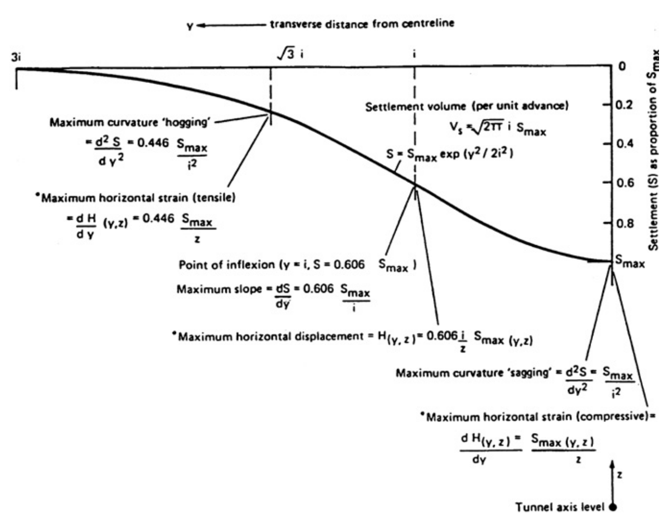

Fig. 12. Idealized transverse settlement trough.

equations for vertical and horizontal ground movements that were also presented in terms of ground strain, slope and curvature (both at, and below, the ground surface). The equations have since become widely used particularly to assess the potential impact of tunneling works during the design process.

The base equations are given as

$$S_{(y,z)} = S_{(\max,z)} \exp - y^2/2(Kz)^2$$
$$V_s = (2B)^{1/2}\, KzS_{(\max,z)}$$
$$\text{and } H_{(y,z)} = S_{(y,z)}\, y/z$$

where:

- $S_{(y,z)}$ and $H_{(y,z)}$ are the vertical and horizontal components of displacement respectively at the transverse distance y, and the vertical distance z from the tunnel axis;
- $S_{(\max,z)}$ is the maximum surface settlement (at $y = 0$) and vertical distance z from the tunnel axis;
- K is an empirical constant related to the ground conditions (e.g. 0.5 for stiff clay and sandy clay to 0.25 for less stiff sands and gravels);
- V_s is the settlement volume per unit advance.

Note that the product Kz defines the width of the trough and corresponds to the value of y at the point of inflexion of the curve; for most practical purposes the total trough width can be taken as $6Kz$.

4.3.4. Relationship between crown displacement and surface settlement

The use of the above described procedure, as well as computations of displacement fields around the excavation or use of an empirical approach, may lead to a direct relationship between the displacement in the tunnel crown (U_{rcrown}) and the middle surface settlement ($S_{\max}$).

Several researchers have proposed formulas to calculate $S_{\max}/U_{\mathrm{rcrown}}$ according to H/R and a parameter varying with the ground condition (Sagaseta, 1987). Each formula has been designed for specific cases. In particular, the choice of the parameter associated with the ground deserves attention because it can incorporate many other factors.

It should be remembered that another method of evaluating surface settlement can be carried out from a typical pseudo-elastic computation (Section 4.2.1).

4.3.5. Back calculation

It may prove useful to start from what is admissible at the surface (cf. Section 5) and go back to the volume loss that can be tolerated above the tunnel alignment. By back calculating, we can envisage different settlement troughs meeting the requirements of surface buildings. In this case, a method similar to that applied for a feedback analysis shall be adopted.

4.3.6. The settlement trough in three dimensions

The equations given above (in Section 4.3.3) describe the form of the ground movements in two dimensions normal to the tunnel axis. In practice the settlement trough also proceeds in advance of the tunnel face. It is a natural consequence of the assumption of a Gaussian transverse profile that this trough should take the form of a cumulative probability distribution and this has been demonstrated by Attewell and Woodman (1982).

Tunneling works commonly comprise a variety of intersecting excavations where tunnels change in diameter and where cross connecting adits and other openings occur. New and O'Reilly (1991) incorporated the radial flow and trough width parameter assumptions into the cumulative probability distribution model to provide a three dimensional model and demonstrated its application to a relatively complex excavation.

New and Bowers (1994) further developed the cumulative probability distribution model by refining assumptions regarding the location of ground loss and giving a full array of equations for the prediction of ground movements in three dimensions. The method is straightforward to apply as the only inputs required are the geometry of the tunnel/site, the predicted percentage ground loss volume (V_s) and the empirical trough width parameter (K) described above. The equations give the vertical and horizontal ground movements, and associated strains and ground curvatures. In particular, this approach gives significantly improved predictions in the vicinity of the tunnels. This model was validated by extensive field measurements taken during the construction of the Heathrow Express trial tunnel at London Airport and elsewhere. Also suggested is a method for the prediction of movements caused by shaft sinking.

Settlement predictions are usually carried out using empirically based procedures without specific regard to the method of construction. However, the proposed construction method will influence the value taken to represent the volume of the settlement trough and thereby the predicted ground movements. Where ground movements are considered important, every effort must be made to control the ground as early and effectively as possible at each stage of the excavation and support process.

The convenience of the Gaussian/cumulative probability distribution curves leads to a series of straightforward mathematical transformations and an apparent precision that may not always be apparent in field data. In practice unexpected ground conditions or poor tunneling technique can lead to significantly larger than predicted ground movements. The considerable strength of this approach lies in its ease of use and in its general validation by field measurements from many sources over many years.

It is of little practical consequence to the ground movements whether the ground loss occurs at the tunnel face or at the periphery of the shield or lining. The construction method will not usually influence the final shape of the ground movement profile but construction sequence can alter the maximum angular distortions experienced in a direction parallel to the tunnel axis.

4.3.7. Volume loss – current EPBM performance

The choice of the volume loss parameter V_s is of considerable importance and the value chosen will be related to experience of the tunneling technique and ground conditions at the particular project for which predictions are required. In this respect, good case history data is vital.

In recent years TBM performance has improved considerably and in particular the reduced volume losses now possible using earth pressure balance machines (EPBM) has significantly reduced ground movements. A very extensive database of ground movement information has been obtained during tunneling works in the UK for the London tunnels of the channel tunnel rail link (CTRL).

Fig. 13 shows the volume losses for approximately 34 km of 8.15 m outer diameter tunnels bored through a variety of soils (Bowers and Moss, Personal communications). The results from the eight EPBM are provided as an example of current achievements in controlling ground movements.

4.3.7.1. C220 Stratford to St Pancras. The Kawasaki EPBM were driven over almost the whole contract length in closed (i.e. pressurized face) mode.

Between chainage 7+000 and chainage 4+500 the tunnels were predominantly driven in dewatered sands. In this area typical volume losses recorded were 0.2–0.4%. It should be noted that this section was driven with continuous bentonite support around the shield in addition to maintaining the face pressure and tail-skin grouting.

Between chainage 4+500 and chainage 4+000 the tunnel boring machines re-entered the Woolwich and Reading clays, progress reduced and settlement increased. Once the settlement exceeded 1% the TBM was stopped and reconfigured to mine clay (i.e. number of picks reduced). When tunneling recommenced, the TBM efficiency improved and volume loss averaged a little over 0.5% for the remainder of the drive though the Woolwich & Reading and London Clays, except at Caledonian Road where special control measures reduced the volume loss to 0.15% under critical utilities.

4.3.7.2. C240 Stratford to Barrington Road. The Wirth EPBM were driven in closed mode through dewatered sand over much of the contract length. Volume Loss was

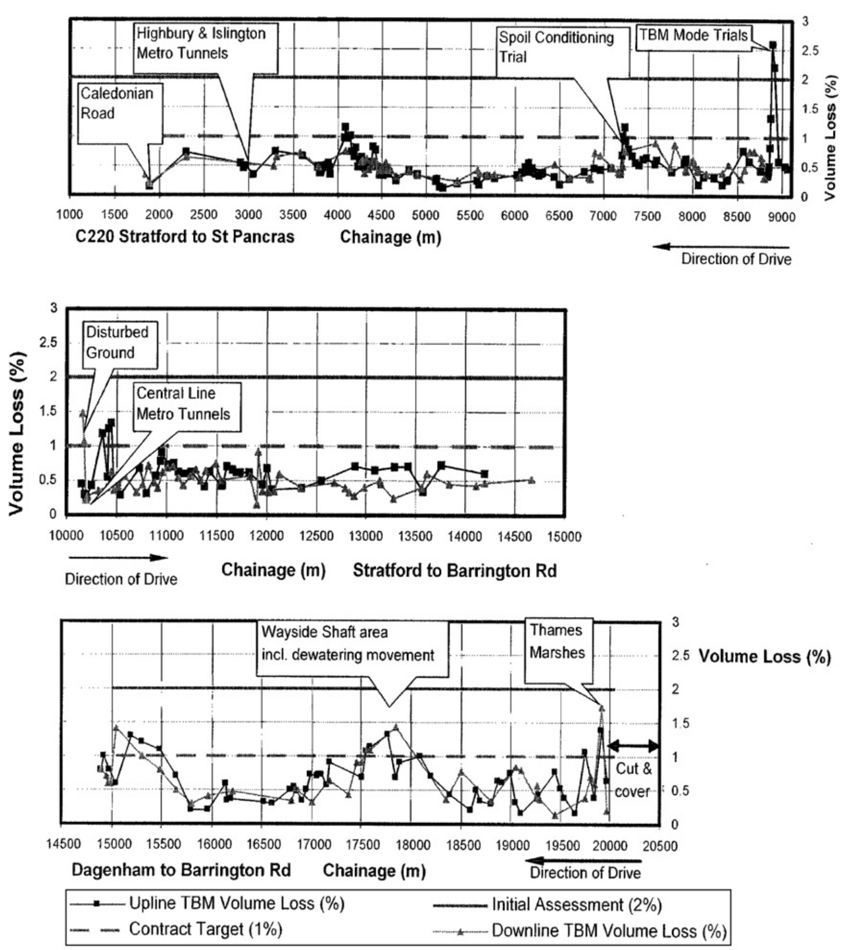

Fig. 13. Volume losses observed on the CTRL.

typically around 0.5%. The majority of the drive was driven without the injection of supporting fluid around the TBM skin. Various problems were encountered in the first 500 m resulting in larger movements at some points, localized surface disruption and damage to property. Nonetheless close control enabled volume losses of around 0.25% to be achieved under critical structures close to the start of the drive.

4.3.7.3. C250 Dagenham to Barrington Road. The first 300 m or so was driven in closed mode with the tunnel crown in alluvium and peat. Considerable problems were encountered here, resulting in variable ground movements and local surface disruption.

After this initial section, the Lovat EPBM were mainly driven in closed mode through London Clay until chainage 17+000. Thereafter the drives passed down through clay into the underlying sands.

It should be noted that, between chainages 18+200 and 17+500, significant settlement effects from local dewatering of the Harwich Formation sands is included in the apparent volume loss graph.

The results from the CTRL project show that, where the EPBM operations were carefully managed, volume losses of 0.25–0.5% were readily achieved. This successful control of the ground should give encouragement to promoters of other works that require tunneling beneath urban areas.

5. Incidence of ground displacements on existing structures

Regardless of the method of construction used, the excavation of a tunnel will generate displacements around the opening that may propagate towards the ground surface. These displacements may differ in their magnitude, spread, as well as direction and speed of propagation and may cause damages to structures located in the vicinity of the tunnel (buildings, structures, carriageways, underground networks, subways, etc.).

It should also be recognized that the displacements of the building and the ground interact with each other, and that the stiffness of existing structures will contribute to reducing the magnitude of tunneling induced displacements.

5.1. Movements induced on existing structures

Experience shows that old masonry structures are subjected to the same deformation as the ground they are founded upon. This is also the case of most constructions founded on footings or isolated shafts.

Conversely, more recent structures (e.g. made of reinforced concrete) which are heavily reinforced will undergo smaller lateral displacements than the foundation ground. The flexural stiffness of these structures results in reduced distortions in comparison to those experienced by the ground, particularly when continuous foundation supports are used (long strip footings, raft).

Stiff structures exhibit a high level of shear resistance and tend to be subject to tilt rather than distortion. This response pattern depends on the building height (number of floors), the number of openings and type of structure (concrete walls, beams and pillars, etc.).

The location of the structure with respect to the settlement trough strongly influences the movements it experiences (extension and hogging over the convex parts of the settlement trough; compression and sagging over the concave parts). This is illustrated in Fig. 14 where some idealized response patterns have been sketched for typical building configurations, either narrow or long, and in relation to their location with respect to the settlement trough.

In summary, it can be expected that a structure located in the vicinity of a tunnel under construction will experience the following movements:

- uniform settlement (or heave);
- differential settlement (or heave) between supports;
- overall or differential rotation;
- overall horizontal displacement;
- differential horizontal displacement in compression or extension.

The main parameters involved in the vertical movement of the structure are described in Fig. 15. where:

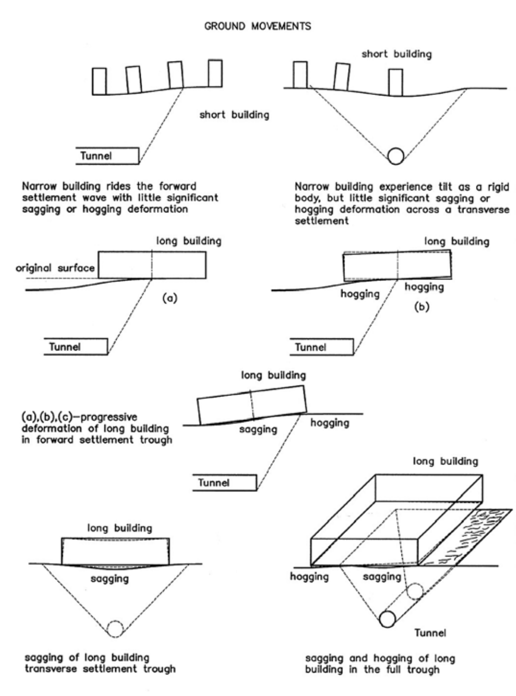

Fig. 14. Typical idealized building response, after (Attewell et al., 1986).

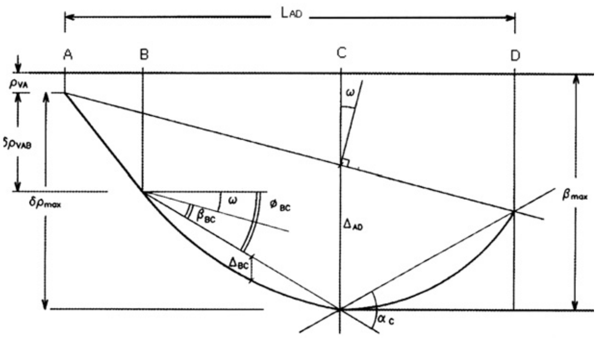

Fig. 15. Vertical movements undergone by the structure.

- L: construction (or element) length in the direction of the settlement trough
- ρ_{va}: absolute settlement at point A
- ρ_{vmax}: maximum absolute settlement
- $\delta_{\rho VAB}$: differential settlement between A and B
- $\delta_{\rho Vmax}$: maximum differential settlement
- ω: tilt.
- Φ_{BC}: rotation of segment BC
- β_{BC}: relative rotation (or angular distortion) of segment BC ($\beta_{BC} = \Phi_{BC\text{-}\omega}$)
- α_c: angular deformation at point C
- Δ_{AD}: relative deflection = maximum displacement relative to the line joining points A and D.
- Δ_{AD}/L_{AD}: rate of deflection.

Note: the relative rotation provides an indication of the shear distortion of the structure; the relative deflection is often correlated to bending distortions.

The main parameters involved in the horizontal movement of the structure are described in Fig. 16.

In this figure:

- ρ_{ha}: horizontal displacement at point A
- ρ_{hb}: horizontal displacement at point B
- ε_{hAB}: horizontal deformation between points A and B; ($\varepsilon_{hAB} = \rho_{ha} - \frac{\rho_{hb}}{L_{AB}}$)

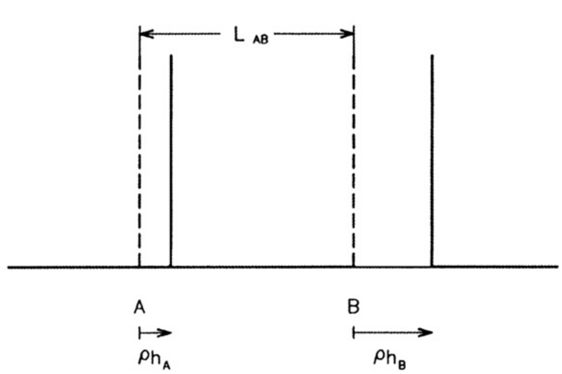

Fig. 16. Horizontal movements undergone by the structure.

5.2. Designation of damages to existing structures

Damages to existing structures fall into three categories:

- architectural damages that affect the visual appearance of the structure;
- functional damages that may be disruptive to the operation.
- structural damages that affect the structural stability.

Damages to structures are caused by cracking of materials with poor tensile strength such as concrete, mortar, plaster and coating (the case of materials involved in underground ducts is analyzed in a separate section). Failure of supporting structures may occur directly as a result of excessive cracking or excessive load transfer onto the reinforcements. To a lesser extent, cracking is harmful to the structure's durability by promoting, for example, steel corrosion.

Crack width therefore appears to be an essential parameter in assessing building damage. Table 1 which is the transcription of the British guidelines can be used in this evaluation of masonry structures (Burland et al., 1977; Burland, 1995; Mair et al., 1996; Burland).

This classification is primarily intended for practical purposes and, as a result, is partially based on repair criteria.

Type 1: Internal cracks can be easily treated during routine renovation works, with some rare external cracks being only noticeable through in depth inspection;

Type 2: Internal cracks can be easily filled but require the masonry to be rehabilitated to ensure sufficient tightness; doors and windows may be slightly malfunctioning;

Type 3: Internal cracks must be opened before filling; external cracks may affect the quality and durability of water-tightness, as well as insulation; cracks may cause significant inconvenience to residents (Serviceability Limit State) such as deformations of door frames, possible pipe breakages, etc.;

Table 1
Classification of visible damage that may affect standard structures

Damage Type	Damage degree	Damage description	Crack width in mm (1)
0	Negligible damage	Micro-cracks	<0.1
1	Very slight damage	Architectural	<1
2	Slight damage	Architectural, to be treated	<5
3	Moderate damage	Functional	5–15, or several cracks >3mm
4	Severe damage	Structural	15–25 (2)
5	Very severe damage	Structural	>25 (2)

Note: (1) crack width is only one aspect of the damaged and cannot be used as a direct measurement.
(2) the number of cracks is also to be considered.

Type 4: Cracking may jeopardize residents' safety (Ultimate Limit State) and structural stability; significant repair works are necessary and may even involve the replacement of wall sections, especially above the opening; doors and windows are twisted, floors are no longer horizontal, supporting beams may be damaged, utilities are broken;

Type 5: The structure may become unstable; it should be partially or totally rebuilt.

This empirical classification applies to classical brick and other masonry structures, rather than modern highly rigid reinforced concrete buildings;

– it does not account for specific structures where cracking may have dramatic consequences, e.g. reservoirs and structures in water-bearing grounds, etc;
– the evolution of damage in types 4 and 5 widely depends on the structural design (e.g. lattice steel structures can be considered particularly resistant);
– it does not take into account of damages that may not be induced by cracking (e.g. deformation or failure of service mains running through the structure).

However it does provide a good assessment for old city buildings which prove the most sensitive and geographically the most likely to be affected by a metro or underground road project.

5.3. Relationships between the displacements of the structure and cracking

The above classification is based on post movement observations and does not relate to the causes of damages. Some correlation can be achieved by introducing the concept of maximum internal extension or critical extension, ε_{crit} (Burland and Wroth, 1975) undergone by the structure (or a component of it) prior to cracking becoming visible. This internal extension may either be due to bending (lateral extension, ε_l) or shear (diagonal extension, ε_d). Fig. 17 illustrates this concept using a comparison of the structure with a thick beam model.

Works (Boscardin and Cording,1989) based on a similar approach have allowed a relationship to be established between the critical extension (ε_{crit}), on the one hand, and the distortion (β) and horizontal extension (ε_h) induced by ground movements, on the other hand. The results of this correlation, as applied to standard structures, are summarized in Table 2.

This critical extension parameter cannot be directly measured and, from this point of view, it could be useful to provide similar ranges of the other two parameters involved in this correlation. Given the number of parameters influencing the behavior of a structure located in the vicinity of underground works, it was decided not to provide such corresponding ranges, due to the risk that particular values get generalized. It is recommended to read carefully reference Boscardin and Cording (1989) for further information.

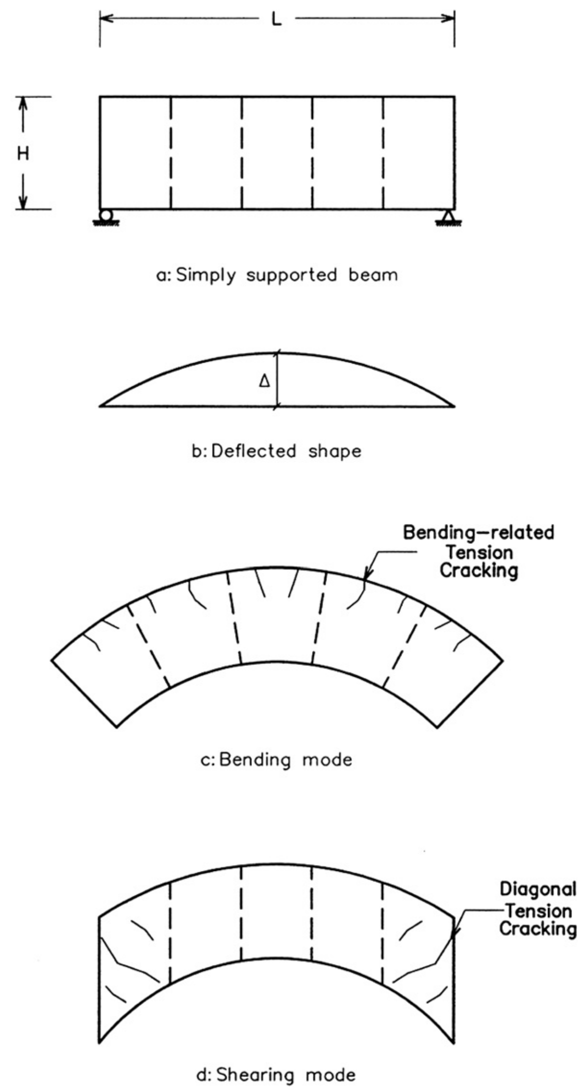

Fig. 17. Thick beam model.

Table 2
Relationship between critical extension and cracking

Damage type	0	1	2	3	4 & 5
ε_{crit} (%)	$\leqslant$0.050	0.050 < $\leqslant$0.075	0.075 < $\leqslant$0.150	0.150 < $\leqslant$0.300	0.300 <

5.4. Relationship between the deformations of the structure and ground movements

A structure subject to the influence of a neighboring excavation, either underground or in open cut, appears to be more sensitive to differential settlements than if subject to its own weight only. This is due to additional deformations imposed by ground movements within the foundation strata. It should be noted that deep foundations too close to the tunnel alignment may result in higher deformations in the structure.

Therefore, the response of the structure strongly depends on its location with respect to the transverse settlement trough (Fig. 14). This will determine the extension to which it is subjected, in particular:

- in the case of a building located above the tunnel alignment, the diagonal extension, ε_d prevails on the longitudinal extension, ε_l which is generally compressive. In the particular case of a low, stiff building located above a narrow trough (sinkhole), ε_l may however reach high values at the base;
- in the case of a building located away from the tunnel, ε_l prevails;
- if the building is in the vicinity of the point of inflection of the settlement trough, the deformations are often complex and severe (hogging).

In view of the practical difficulties experienced in achieving an accurate determination of the distribution of ground movements at the surface and modeling of the actual mechanical characteristics of the structures (see Section 5.7.4), some relationships have been proposed to correlate the structural damage criterion (ε_{crit}) with the average slope of the settlement trough (β_{aver}) underneath the structure. Such a relationship must however be used with care, and is not reproduced in this paper. Details on a more comprehensive method of linking tunneling effects to their consequences are described in Section 5.7.4.

5.5. Limits of structural movements for use in preliminary analyses

It is clear that an absolute settlement criterion is insufficient to solely describe the sensitivity of the overlying building to underground movements, unless a very low limit is imposed.

As a first approach, reference can be made to article 2.4.6 – par. 7 of EUROCODE 7 – Part 1 (ENV. 1997-1:1994) (Eurocode 7), as quoted hereafter, given its novelty at the time of writing this recommendation:

- it is unlikely that the maximum admissible relative rotations for open frames, empty frames and bearing walls or continuous masonry walls be the same but they probably range from 0.5‰ to 3.33‰ to avoid a limit state to be reached within the structure. A 2‰ maximum relative rotation is acceptable for many structures. The relative rotation for which an ultimate limit state is likely to be reached is about 6.67‰;
- for standard structures on isolated foundations, total settlements of 50 mm and differential settlements of 20 mm between adjacent columns are often acceptable. Larger total and differential settlements may be admitted if the relative rotations remain within acceptable limits and if total settlements do not cause damages to utilities, or hogging, etc.
- the above indications on limit settlements apply to common routine constructions. They should not be applied to unusual structures or buildings or those for which the load intensity is highly non-uniform.

Table 3
Range of Serviceability Limit State for standard structures

Damage type	Average slope of settlement trough under building (%)	Maximum settlement of the building (mm)
1	< 2	< 10
2	$2 \ll 4$	$10 \ll 20$

Numerous indications dealing with less common constructions or structures can be found in the literature. In addition, tilt (ω) of a tall building is visible for values of 4‰ and above.

Caution: It must be kept in mind that the impact of tilt on building functionality must be carefully examined, given that a serviceable limit state may be exceeded even without cracking (lift, etc.).

Table 3 provides some correlation between the above defined damage types according to the EUROCODE and British practices (Rankin, 1988).

5.6. Tensile deformations admissible by underground utilities

The term "underground utilities" includes service mains such as drinking water, sewerage, energy (gas, power, oil, etc.) and public or private underground transport infrastructure. This involves various structures in size, design and depth. However, these structures are all characterized by their large length in relation to their transverse size, which is roughly circular.

The response of utilities to undergoing movements is a difficult soil/structure interaction problem.

Large diameter utilities (>2 m) are less numerous, which justifies case-by-case studies to be performed by means of sophisticated modeling techniques to assess the impact of adjacent underground works in such cases. This can in turn lead to an evaluation of the magnitude of admissible movements.

A similar approach cannot be used for a great number of highly sensitive service mains. The sensitivity of these structures to ground movements (horizontal and vertical) widely depends on their lining material (concrete, cast iron, steel, ductile cast iron, PVC, PE, etc.) and gasket characteristics. In comparison with the values displayed in Table 2, the tensile strain criteria respectively associated to the 'Serviceability Limit State' and the 'Ultimate Limit State' of service mains are of the order of 0.03% and 0.1% for cast iron and lining concrete, 0.05% and 0.1% for steel, 0.1% and 0.2% for ductile cast iron and 0.7% and 2.0% for plastic materials.

In fact, the large length of ducts, in comparison to their transverse size and to the width of settlement trough, makes the inner expansion induced by differential settlements relatively limited, typically 1/10th the average duct slope. In addition, the strong longitudinal stiffness of linings, which are generally made of precast pipes connected with or without flexible gaskets and, results in the magnitude of additional deformation caused by horizontal ground displacements remaining limited. It can therefore be concluded that, in most of the cases, only ducts made of brittle materials (cast iron, clayware or concrete) will need to be considered in the determination of admissible ground settlements.

Other than the response of the service main per say, particular attention should be paid to the consequences of differential displacements occurring at joints between the duct and other structures located in the area affected by the underground works.

Additional consideration should be made in the analysis to the fact that maintenance or replacement of some duct sections may generate relatively moderate costs, particularly when these are scheduled concurrently with the tunneling works.

5.7. Design methodology

The proposed approach to study the effects of underground works on existing structures can be broken down into six stages, with geotechnical investigations being dealt with separately.

5.7.1. Phase 1: investigation of existing buildings

This stage consists in surveying and data collection on the nature, configuration and condition of the buildings and utilities together with topographic measurements and technical expert reviews.

It is recommended to properly evaluate the actual condition (zero condition) of each structure prior to the start of the construction works and, when possible, to review the history of the building, particularly in terms of movements already experienced. This task is not easy, but it is essential. In this respect, the use of standard classifications based on cracking levels should contribute to making this investigation task less subjective. It is recommended that a preventive assessment be included at this stage to provide a strong legal base to the records.

5.7.2. Phase 2: information summary

This phase comprises producing a typological classification of the building and utilities according to the nature, function, value, size, design, age and current condition of its individual components. When possible, a zoning will be developed to identify areas of homogeneous characteristics and incorporate geotechnical data collected during the survey.

5.7.3. Phase 3: selection of damage criteria

This phase aims at determining the objectives to be achieved in terms of damage limitation and converting these objectives into straightforward criteria to be used by the designer.

If the preliminary expert review has led to the production of data on the condition of the building prior to construction and cracking reports, it is recommended to use the strains involved in the production of the initial reference condition as evaluation criteria. In this case, the initial recorded strain will deducted from the value of allowable extension induced by construction.

The selection of such criteria should also account for the physical possibility to make measurements at the worksite. Except for particular cases, it is often easier to base the evaluation on an average slope of settlement trough, the geometry of which will be determined on site from surveying records taken on carriageways and buildings.

5.7.4. Phase 4: modeling

Modeling works are intended to correlate the building displacements induced by ground movements to its structural deformations. The deformations of the building are assessed, most of the time, by subjecting its foundations to the excavation induced ground movements with no account of any influence of the structure's stiffness. This simplifying and conservative approach also reflects relatively well the rapid development of settlements in the short term before any adjustment response can be developed within the structure.

Settlement studies conducted during the design phase should enable the engineer and the client to assess the tunneling risks involved in the project. They should therefore make extensive use parametric studies, to evaluate the influence of geotechnical, building, as well as excavation technique parameters.

For given predictions of ground movements, it will often be necessary to quantify their potential effects on brick and masonry buildings and this problem has been considered by Burland (1995) and Mair et al. (1996). Broadly speaking their approach is to calculate the tensile strains in the building and to interpret these in terms of damage "degrees of severity" which are expressed in six categories ranging from "negligible" to "very severe". Each category of damage is described and its ease of repair indicated.

For tunneling works they suggest a three-stage approach:

(1) *Preliminary assessment:* Ground surface settlement contours are drawn (using the empirical predictive methods given above) and if the predicted settlement of a building is less than 10mm it is assumed to have a negligible risk of damage and the assessment process terminated. This is subject to an additional check that no building experiences a slope in excess of 1 in 500 (Note that, for a given settlement, a small shallow

tunnel will be more damaging than a large deep one because the structural distortion will be greater for the former).

(2) *Second stage assessment:* The maximum tensile strain in the building is calculated and a "possible" damage category assigned. Note that this will be a conservative assessment because the building strains are based on "green field" ground movement predictions whereas, in practice, the actual movements may be reduced by the stiffness of the building. This effect could give rise to problems where services enter buildings because of the differential movement of the building and the adjacent ground.

(3) *Detailed evaluation:* This stage is undertaken for buildings where a "moderate" level of damage has been predicted in stage two. It considers the tunneling sequence, three-dimensional aspects, specific building details and soil/structure interaction. The assistance provided by numerical methods may be valuable at this stage and a hybrid (Gaussian/Finite element) approach has been suggested by Potts and Addenbrooke (1997). Protective measures would then be considered for buildings remaining in the "moderate" or higher damage categories.

The effect of tunneling on pipelines has been considered by Bracegirdle et al. (1996). They addressed the important problem of cast iron pipelines, which are particularly vulnerable because of their brittle nature. Again the ground movements are estimated following the equations given by O'Reilly and New (1982) and the tensile strains in the pipe calculated assuming either flexible or rigid jointing of the pipe sections. These strains are compared with various acceptability criteria.

Tunneling works at depth usually produce relatively smooth settlement trough shapes and the angular distortions occurring at the foundations of structures are reasonably predicted by the methods described above. However consideration must sometimes be given to unusual geological conditions which may result in localized differential settlements and unexpected damage to structures. An example of such an occurrence is reported by Friedman (2003).

Settlements caused by longer-term consolidation may tend to be less damaging than the short term movements but should not be overlooked.

5.7.5. Phase 5: determination of the allowable displacement thresholds

The purpose of this stage is to determine the contractual threshold requirements that will have to be met during construction.

The concept of acceptable damages and required ancillary works (preventive or remedial) involves the consideration of non project oriented constraints (human, cultural and legal environment) and economic criteria. These aspects will drive the determination of admissible threshold values. It is not always possible to limit the threshold parameters to a single criterion of admissible movement, unless very stringent requirements are imposed to this criterion. The summary prepared as part of Phase 2 is essential to allow developing contractual criteria that are well adapted to the actual needs in terms of building protection.

Threshold values must never be taken as constant; they should primarily be considered as alarm indicators and should be continuously adjusted in view of the observed response of the building subject to tunneling induced ground movements (it is advisable to allow some tolerance on the threshold values, and preclude the development of solutions with 0.1‰ accuracy!).

In a similar manner, one should develop for each project an alarm (caution) threshold and a stopping threshold.

5.7.6. Phase 6: back analysis and calibration of models with observed data

It is clear that the development of settlement estimates is not an exact science, and that measurements should be made to monitor the construction works and their effects on the surrounding ground and structures (see Section 7).

It is absolutely essential, as part of this process, to check model predictions against data obtained from in situ observations. This process of validating the design assumptions should be a routine part of the construction management plan.

6. Settlement control

It would obviously be more satisfactory to plan before construction starts all the measures required to minimize the impact of the tunneling works. However, this optimal situation is hardly feasible both technically and economically due to the uncertainties that remain at the design stage on the ground response to tunneling and the actual condition of the buildings.

Based on past experience, it is recommended to plan, at the design stage, a reasonable set of preventive measures to be used before and during construction, as well develop contingent remedial solutions to be used in case difficulties are encountered during construction.

A number of techniques developed for limiting settlements or their cause, are described below. The following section focuses on the solutions principles and limitations. One should refer to the relevant specialized literature for further information.

These techniques can be of the preventative or remedial type, and it is generally difficult to really differentiate between the two categories; in fact this distinction is mostly subjective and depends mainly on when the decision is made to implement them.

6.1. Improvement of the overall project conditions

At the preliminary design stage, one should aim at selecting an alignment that offers the most favorable conditions for settlement control. This can be achieved by selecting;

– the largest depth of cover, provided that this does not cause the tunnel drive to encounter ground layers of poorer characteristics;
– ground layers of good mechanical properties for the alignment, provided they are of sufficient thickness (at least one tunnel diameter above the crown). Should this not be the case, it is better to excavate the tunnel underneath a stiffer layer, and rely on the bridging effect this can produce, rather than taking the risk of disturbing its mechanical stability by cutting through it;
– the smallest excavated cross section. This recommendation often leads, for a transportation project, to choosing between a single or twin-tube tunnel. The decision is generally made on the basis of the nature of the grounds to be encountered, the advances in confinement technologies and economical opportunities (e.g. availability of used Tunnel Boring Machines). Although the twin-tube option is often preferable, attention should be paid in that case to ensuring that the distance between both tubes is sufficient to avoid cumulative settlements;
– an as-straight-as-possible alignment if a shield is used.

One should also keep in mind, when selecting the construction method, that increased settlements are often associated with stoppages or reduced rates of face advance.

6.2. Improvement of ground characteristics

Ground improvement can be achieved by altering its mechanical and/or hydraulic characteristics. The following sections are limited to providing some general background on technologies that can be assumed to be well known to practitioners.

6.2.1. Conventional grouting

Grouting of the ground mass can be used to increase its cohesion (consolidation grouting) and reduce its permeability (water-tightness grouting). The effectiveness of the techniques relies on the groutability of the ground [see AFTES recommendation (AFTES, 1988)] and their ease of implementation.

It can be carried out from the ground surface, when permitted by site conditions, or from the tunnel which, in turn, results in reduced rates of advance. In the particular case of shield driven tunnels, this would require specific provisions when manufacturing the machine.

This technique may induce ground heave if uncontrolled fracture is allowed to develop within the ground, which may be the case with shallow urban tunnels where the magnitude of in situ stresses do not allow high grouting pressures to be sustained. Interestingly, there appears to be less concern for the risks associated with heave rather than settlement, although both phenomena can cause damages of the same nature, with cumulative effects when they occur concurrently.

One should be careful with the medium term efficiency of grouting works. In the case of gel grouting performed several months prior to tunneling, some loss of mechanical performance may be experienced as a result of grout deterioration with time (synaeresis).

It must also be remembered that the risk of polluting the water table should be assessed for each product type to be used.

6.2.2. Compaction grouting

In the case of pervious grounds, such as fills, for which conventional grouting would lead to using large quantities of grout without any guaranteed efficiency, or in some loose grounds, considerable improvement of the overall stiffness can be achieved by injecting a dry mortar from boreholes.

This technique allows an overall improvement of the mechanical characteristics of the ground to be obtained. It may be implemented from the surface as part of an underpinning process. Its efficiency must be controlled by means of careful topographic surveying, with adjustments being made on the basis of observed surface heave.

The implementation of grouting works (conventional or compaction) performed concurrently with the tunneling process, is referred to as compensation grouting (Baker et al., 1983; Harris et al., 1994).

6.2.3. Jet grouting

This technique consists in high speed injection of grout into the ground through pre-installed drill pipes. The injection of grout, with various speed levels depending on the technique used (single, double or triple jet with or without pre-washing), de-structures the ground at a distance which varies with the compactness of the ground. The grout mixes with the ground to form a column of stabilized soil. The diameter of the column varies in the range 0.30–1.20 m depending on the technique used and the consistency and nature of the ground.

Ground treatment can be performed through vertical, inclined or sub-horizontal borings. The last option (with single jet) can be implemented from the tunnel face. Particular attention must be paid, when using this approach in fine soil grounds, to potential adverse effects associated with unexpected pressure build up within the ground being excavated at the face (sudden fracture and large heave).

When ground improvement is required, this technique may be used in lieu of grouting in very fine grounds. The efficiency of this technique is well proven, and can lead, when used with a fine drilling mesh, to total ground substitution. There are, however, a number of implementation constraints (power consumption, spoil processing and removal, instantaneous loss of bearing capacity before the grout has set) which require a thorough evaluation to be made before this technique is used.

6.2.4. Ground freezing

The principle of this technique is to build a shell or vault of frozen ground around the area to be excavated. The whole tunnel cross section may also be frozen if the capacity of the system is sufficient. The technique may be used in almost any grounds with permeabilities lower than 10^{-3} m/s.

Freezing can be carried out from the surface or the working face. In both cases, the main difficulty lies both in the control of drilling deviations during the installation of the freezing pipes (which length is limited to 50 m) and of the circulation of large quantities of underground water.

Although this technique can allow ground stability to be improved dramatically, it also requires very tight control because of the potential detrimental effects it can cause within the ground, with heave being generated by seepage of groundwater towards the cold source during freezing and subsequent settlement – and alteration of the ground properties while thawing – at the end of the freezing process.

6.2.5. Drainage

The control of hydraulic gradients that may affect the stability of the tunnel face may be achieved by lowering the groundwater table from the surface or through drainage at the face. This should be implemented as far ahead of the face as possible.

When the ground is likely to undergo consolidation settlements, or could be made unstable as a result of dewatering (karst filing), a risk assessment of the possible consequences of the implementation or lack thereof of drainage must be undertaken prior to using this technique.

6.2.6. Compensation grouting

Compensation grouting is most commonly carried out in association with new excavation, which may be adjacent or beneath existing tunnels. The grouting may be intended to protect overlying structures or the existing tunnels. The effects of excavation and compensation grouting will depend largely on the position of each in relation to the existing structures and the sequence of grouting and excavation employed. Grouting either in advance of or following completion of excavation may reduce these effects.

Compensation grouting can, when carried out in close proximity to existing tunnels, induce modes of deformation in linings that are far more damaging than the elliptical deformation, which usually accompanies the general loading and unloading of tunnels. When carrying out monitoring of tunnel linings during grouting, therefore, it is not sufficient simply to measure diametric change. It is necessary to determine the mode of deformation and, in the case of bolted segmental cast iron linings, determine tensile strains at critical locations. Damage to cast iron linings arising from compensation grouting is usually in the form of tensile cracking of the flanges at bolt positions. Damage in the form of linear cracks along the long axis of the pans of segments may also be seen. Linings where adjacent rings have been 'rolled' are particularly susceptible to damage. Where damage is expected, it may be prudent to release bolts and allow some articulation of the linings.

Damage may also occur where tunnels are lifted by compensation grouting beneath them. In this case, damage may occur as the tunnel linings articulate in the longitudinal direction, and may be concentrated at changes in section, headwalls, etc.

The process of compensation grouting involves the injection of grout into the ground at high pressure. The process is essentially a jacking operation which produces movement of the ground regardless of whether the grout injected is concurrent with tunnel construction or activated afterwards. Thus a reaction force is necessary in order to generate the required (upwards) movement and, consequently, there is unquestionably the potential for loading and deformations to be generated in any tunnel lining or temporary works situated below an area of grout injection.

The controlling factors can be divided into those determined by the design of the grouting facilities and those which relate to the implementation of injections for a given situation.

Design of the grouting system:

- the vertical total stress at the grouting horizon which has a strong influence on the grout pressure in the ground;
- the vertical spacing between the grouting horizon and the tunnel or excavation which influences the spread of load;
- combination with other factors causing loading to a tunnel e.g. close proximity tunnels;
- the properties of the tunnel lining (stiffness, strength, joints).

Implementation of injections is affected by:

- the timing of injections relative to excavation;
- the volume of individual injections;
- the plan location of injections relative to the excavation;
- the properties of the grout particularly with respect to the shape and extent of the grout bulb/fracture formed;
- the quantities of grouting undertaken.

The design of a compensation grouting system should include consideration of the potential effect on the tunnel(s) above which it is to be implemented. Observation is a vital part of any such system and measurements of the effect of grouting on tunnels should be included in the monitoring system.

The form of damage to tunnel linings, which can be caused by compensation grouting, can be either due to excessive deflection or excessive stress causing cracking. In segmental linings, large deformations can often be accommodated by rotation or shear at the joints between segments without inducing high stresses in the linings themselves. Exceptions include rolled joints, junctions, headwalls or temporary internal propping which substantially increase the stiffness of the tunnel structure. Sprayed concrete linings do not have the potential to accommodate large movements without cracking.

If compensation grouting is carried out over a wide area, the total vertical stress cannot exceed the overburden pressure. Segmental tunnel linings are generally robust in comparison to the loads they are subject to at shallow depths and are able to sustain full overburden pressure. Difficulties therefore arise when the grouting is concentrated in a small area and produces localized loadings or deformations within the lining.

The implementation of grouting injections can be modified in respect of the controlling factors listed above to reduce the potential impact on tunnels. For example, if compensation grouting is carried out concurrently with the tunnel advance, exclusion zones can be imposed around the excavation face, i.e. no grout is injected within specified plan distances of the tunnel face. Within limits, the precise location of injections during tunneling has a negligible effect on the efficiency of the grouting in reducing settlements, whilst this can be used to limit the impact of the grouting on the tunnel below.

6.3. Structural improvement of buildings

One way of reducing the sensitivity of existing buildings to tunneling induced ground movements is to reinforce the existing structure, prior to construction. Several options are available, including:

- foundation reinforcement, to reduce the sensitivity to lateral strains;
- front wall stiffeners, elevated belts and floor tie bars, to reduce overall distortions;
- frames at the openings (doors and windows), to resist to local distortions;
- steel ribs in sewer mains and tunnels.

Underpinning may also be used to reduce building settlements. These works are completed prior to tunneling, to allow the building loads to be transferred underneath the excavation level.

6.4. Improvement while tunneling with the sequential method

Generally speaking, reducing the number of excavation stages usually results in lower settlements. Staged excavation typically results in reduced rates of advance, more stages of temporary support installation, additional underpinning and delayed closure of the tunnel liner. All of these consequences more than compensate the expected benefit of achieving smaller individual excavated sections and, as a result, it is necessary to revisit the older belief that: horizontal partitioning of the face = reduced settlement.

Modern excavation and support installation techniques allow such reduction in excavation stages to be achieved, thus contributing to improving the overall rate and safety of advance. Horizontal partitioning remains however in use, particularly with hand mining (small cross section) where support elements of limited weight must be used to facilitate handling and rapid closing of the liner.

Where the ground is potentially unstable, the shape of the excavated cross-section may be altered to improve stability conditions. If necessary, axial or peripheral reinforcement may used at the face. In case of waterbearing ground, other ancillary techniques will be needed to allow controlling hydraulic gradients at the face.

These measures can be planned at the design stage or implemented during construction when unexpected instability occurs. It is clear that the latter will affect the overall rate of advance of the project, with subsequent cost overruns. Introducing any excavation staging in the course of construction is difficult to manage and can be expected to dramatically alter the economics of the project.

6.4.1. Face support

The conventional response to face instability during excavation is to use a bench-and-heading approach, with delayed excavation of the central part of the face. This can be used together with some tapering of the face, although this approach tends to be rarely used because of the construction constraints it generates for liner installation at the crown.

Conversely, using a confining layer of sprayed concrete (with or without reinforcement) is recommended to help control minor instabilities that are likely to develop within the face core.

For more critical cases, face bolting may be used to provide the strength required to ensure the overall stability of the face. This system should preferably be designed to permanently offer a constant confinement capacity (combination of bolts of variable length, as required by the rate of advance) (Fig. 18). The type of face support should be such that it could be easily destroyed by excavation tools (e.g. fiberglass bolts or sub-horizontal jet grouting columns).

6.4.2. Pre-support

When design studies or observations made during construction indicate that serious instability (i.e. involving the stability of the ground above the tunnel crown) may be experienced, more comprehensive measures must be considered.

When ground improvement from the surface is not appropriate (technically or economically), some pre-support must be installed from the tunnel face to support all or part of the ground located ahead of the face. Several methods are available, and can be selected on the basis of the quality of the ground, the excavation geometry (cross section height) and the means available at the worksite.

6.4.2.1. Forepoling. This technique aims at limiting the decompression in the crown immediately ahead of the face. It consists in installing longitudinal bars or steel plates at the periphery of the face, typically over the upper third or quarter of the excavated profile. These bars or plates are assembled to form a short canopy and are often combined with steel ribs, with the canopy sitting on the closest rib installed behind the face (Fig. 19).

The effectiveness of the canopy depends on its length and width. The canopy length is dependent upon the penetration of the bars or plates in the ground ahead of the face; the overlap between two consecutive canopies is about twice or three times the spacing between ribs. This spacing controls the angle of installation; small angles (15°) can only be achieved with specific provisions such as the use of core steel ribs or lattice girders.

Forepoling, also referred to as forepiling, is appropriate for use in coarse alluvial deposits, raveling grounds or highly weathered rocks. In some cases, hollow pipes are used in lieu of bars, with mortar being injected through the pipes after installation to improve the development of arching between the bars.

When the nature of the ground does not allow sufficient arching to develop, steel plates may be used. However, because of the low inertia of the plates, the penetration length can hardly exceed 1.5 times the spacing between consecutive ribs.

With shield tunneling (single shield), forepoling may be improved by jacking high stiffness, sub-horizontal rods from the shield. This technique is rarely used. The canopy formed ahead of the face with this device plays a similar role to that of the cantilever hood used in compressed air shields.

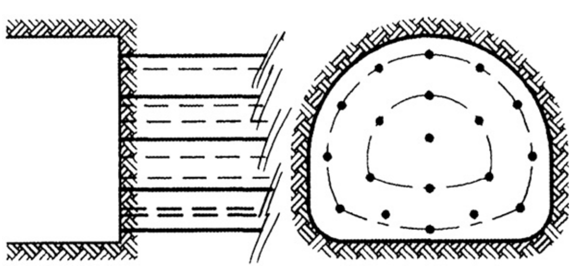

Fig. 18. Principle of face bolting.

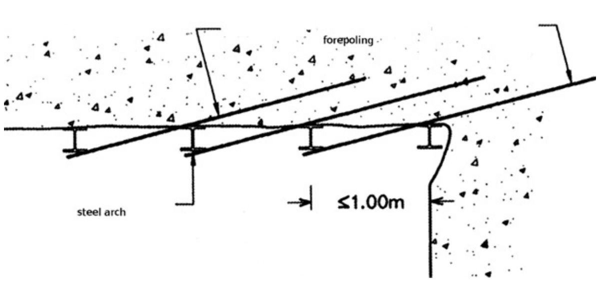

Fig 19. Basic forepoling.

6.4.2.2. Umbrella vault. This system is an extension of the previous one. It is designed to attain a penetration length ahead of the face roughly similar to its height, so that decompression can be limited and subsequent overall failure mechanisms prevented.

The typical umbrella vault, sometimes associated with face reinforcement, makes use of bars (diameter 32 or 40 mm), grouted tubes (diameter 90–250 mm) or jet grouting columns (diameter 30–60 cm). Because of drilling guidance considerations, the vault length does not exceed 12–15 m. In practice, these vaults are of conical shape, so that they can be installed from the face without over-excavation, and overlap with each other (Fig. 20). The overlap length depends on the cross section height and the ground condition; it is recommended to be made no less than 3 m.

Ground decompression during excavation is limited by arching effects that are allowed to develop between the face and the latest installed supporting rib. Whilst the effectiveness of the device depends on its longitudinal stiffness, the quality of contact between the longitudinal elements and their support is paramount.

In the case of very shallow tunnels driven underneath existing buildings, ancillary techniques are required and the approach is adjusted to suit the project and worksite parameters. Such ancillary techniques include:

– installation of parallel steel pipes of high stiffness (diameter 300–600 mm), that are usually filled with concrete and may be secant or even connected. These pipes are often jacked horizontally over some length (less than 30–40 m), using a jacking unit of sufficient stiffness, so that deviations are kept under control. Direc-

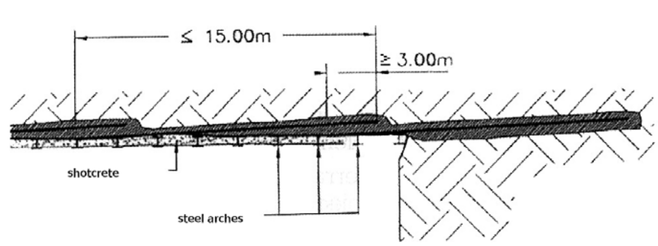

Fig. 20. Typical umbrella vault.

tional drilling may also be used for pipe installation, which allows longer lengths to be achieved with no ground reaction structure required;
– construction of concrete filled secant tunnels using microtunneling (diameter $\leqslant$ 1.20 m) or conventional techniques.

6.4.2.3. Pre-vaults. The pre-vault technique is derived from the concept of the umbrella vault (Bougard et al., 1979; Puglisi, 1991). It consists in building a 15–30 cm thick shell sub-parallel to the tunnel from the face, prior to each excavation stage. The tunnel wall support is thus made of successive pre-vaults, which overlap over some length, as required by the encountered ground conditions (Fig. 21). These pre-vaults may be used as roof support with bench and heading or full face excavation.

The method consists in excavating a slot over the face by means of a longitudinal pre-cutting saw mounted on an arch-shaped structure. As excavation of the slot progresses, the slot is filled with shotcrete. The maximum depth of the slot (currently less than 5 m) depends on the stiffness of the device, as well as on ground conditions, as this will control the stability of the slot walls prior to shotcreting.

Because of the equipment it requires, the pre-cutting technique cannot be introduced in the course of construction, and must be planned from the design stage.

6.4.3. Crown support

Whether ground improvement or reinforcement is required or not prior to undertaking the excavation works, experience shows that, with the sequential method, a large proportion of the overall observed settlements arises from inappropriate support installation. Without pretending to be comprehensive or repeating matters covered earlier in the document (see Section 3.1), the following sections focus on a few situations where particular attention should be required.

6.4.3.1. Support with steel ribs. This support technique is still widely used in France, undoubtedly as a result of practices developed after World War 2 which were inherited from the mining industry. Partly for the reasons developed above and in view of ongoing trends (sprayed concrete, lattice girders and bolts), this approach should loose its predominance in the coming years.

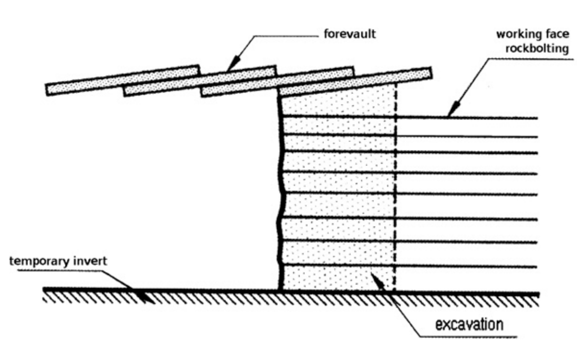

Fig. 21. Principle of the pre-vault technique.

With steel ribs, the primary source of settlement is in defective wedging of the support to the ground. Obviously, steel ribs installed at the face but not secured against the ground will be of no help for limiting the decompression of the ground, and will only provide an illusive sense of comfort as regards settlement control and personnel safety. Contact between the ground and the support must be achieved over the whole rib and its base.

The quality of wedging can be improved by increased wedge density and compressibility. Defective wedging will result in increased ground deformations, with ground tending to fill the gap left with the rib, as well as rib deformations that may be difficult to control given their low bending stiffness if they are not uniformly secured against the ground.

If the rib base is improperly secured, either as a result of an insufficient bearing surface or excessive compressibility of the supports, the loads carried by the rib may lead to punching of the bearing ground. This will in turn produce an overall settlement of the ground, the magnitude of which will depend on the load carried and will be amplified by improper wedging.

The performance of support systems based on steel ribs is also affected by the quality of lagging installed between two consecutive ribs. Obviously the quality of support will be different if wood or steel plates are used or if this support is made of a sprayed concrete shell.

Wood lagging cannot provide any confinement of the ground. In that case, ground loads are primarily transferred by arching onto the ribs, with lagging only taking the remainder of ground weight left unsupported between the ribs. The overall effectiveness of the support system therefore relies on the quality of wedging (see above), with detrimental effects to be expected should local deficiencies in contact conditions occur.

The same is true of steel plate lagging. Confinement of the ground can be improved by filling the gap between the ground and the plates with concrete, but this technique is of limited efficiency in terms of ground movement control due to difficulties in fully sealing the ground-plate interface, particularly at the tunnel crown. As a result, this technique provides little advantage with respect to wood lagging.

These difficulties can be overcome by using shotcrete support. In that case, sealing of the ribs against the ground is improved and additional confinement is provided by the shotcrete shell due to its stiffness and tight contact with the ground. To make the best use of this technique, it is recommended that a first layer of shotcrete be placed against the ground immediately after excavation, and be then used for rib wedging.

Another current trend is to replace steel ribs and lagging with lattice girders and shotcrete.

6.4.3.2. Support bolts. With bolting, the limitation of displacements around the opening, and therefore the limitation of the ground deformations, is highly dependent upon the bolt length (to be adjusted in proportion to the extent of plastic deformations within the ground) and sealing, which are the two parameters controlling the quality of ground confinement that can be achieved.

For better settlement control, it is highly recommended to combine bolting with the immediate placement of a shotcrete shell onto the ground after excavation.

6.4.3.3. Shotcrete. The current trend is for a more systematic recourse to fiber reinforced shotcrete. This is beneficial to settlement control given that less time is required for support installation than when wire mesh reinforcement is used.

6.4.3.4. Expanded concrete segments. In view of reducing ground decompression, it may be advantageous to install the final liner as close as possible to the face, as:

– longitudinal arching may develop more easily between the face and the liner;
– given its stiffness, the liner can contribute to limiting ground decompression.

However, casting the final concrete liner next to the face is difficult to achieve, and some large distance of liner installation behind the face is usually imposed as a result of worksite constraints. A solution may be provided by the expanded segment method (also referred to as the Jacobson method), in which the liner is formed of a sequence of large span vaults made of precast reinforced concrete segments (Fig. 22). These vaults are assembled at a distance from the face ranging from one to two times their width (2×0.8 to 1.2 m). Final vault installation is achieved by expanding the liner against the ground using flat jacks mostly located in the crown area.

With the liner thrust being re-centered as a result of the jacking action, the expanded segment vault allows wider spans to be achieved, thus contributing to lower excavated profiles, thicker overburden covers and reduced earthworks This technique has been successfully used on a number of underground projects in Paris [RER Lines A and B, METEOR, RER Line D, EOLE (Carrnes, 1993)].

It must however be kept in mind that assembling the segment erecting machine will require the construction of an assembling chamber by means of conventional tunneling, with limited efficiency in terms of settlement control.

6.4.4. Underpinning of the upper cross-section

In the case of soft ground excavation, using a bench and heading construction sequence, settlements induced when excavating the lower part of the cross section may be reduced by underpinning the upper liner in such a way that the loads carried by this liner are transferred to the ground beneath the invert.

Depending on the ground conditions and structure of the upper liner, underpinning can be achieved by micropiles, jet grouting columns (Tanis et al., 1994) or, in some occasions, shafts (Fig. 23).

Whatever the option, ancillary measures will need to be taken so that no additional settlement is generated as a result of the preventative methods being used. Other than liner closing, the following should be considered:

– micropiles may need to be pre-loaded by jacking, in which case allowance should be made for the piles shortening during pre-loading;
– jet grouting columns should be installed in such a way that their design strength is achieved prior to being exposed to ground loads; failure to do so may result in large overall settlements.

6.4.5. Invert arch

With poor ground conditions in relation to the loads they are subject to during excavation, it may prove very effective to close the cross section after each major excavation stage. This may be achieved by a temporary invert arch that will be destroyed during subsequent excavation stages. This invert serves three main purposes:

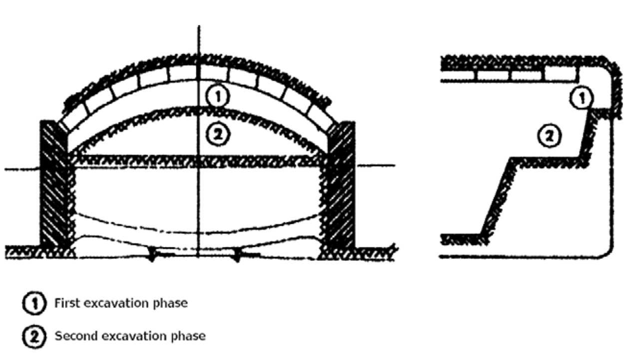

Fig. 22. Principle of the expanded concrete segment method.

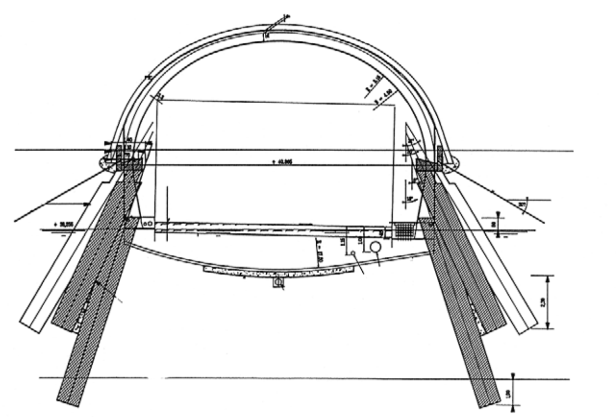

Fig. 23. Principle of underpinning the upper tunnel section.

– convergence limitation at the base;
– confinement of the ground in the invert;
– improvement of the bearing capacity at the base.

This invert may be shotcreted with wire mesh reinforcement, in which case it can also be used as a rolling platform. When steel ribs are used in the vault support, the temporary invert can be made of steel elements connected to the vault ribs. This solution is usually found less effective than a shotcreted invert, because it provides little or no ground confinement unless concrete is poured in the invert which tends to make the steel elements redundant.

6.5. Improvement with shield tunneling

The decision to employ shield tunneling relies on a number of technical and economical factors. This section refers to the case where the use of a pressurized shield with segmental liner installation within the shield skin is made necessary by the poor quality of grounds to be encountered. In this case, all attempts must be made to act upon the settlement sources and prevent ground decompression:

– at and ahead of the face;
– above the shield;
– at the tailskin.

It is important to note that the success of this endeavor will rely on both the technical choices made at the design stage and the presence on site of an experienced crew possessing all the required skills involved in the complex operation of a shield. Although this may prove costly, it must be kept in mind at tender evaluation that this may be the price to pay for preventing cost overruns that would be significantly more difficult to manage should a serious incident occur during construction.

6.5.1. Reduction of decompression ahead of the face

Beyond the selection of appropriate face support (compressed *air*, slurry or Earth Pressure Balance shield), the most fundamental issue is the ability to control of the confining pressure. This is not easy to achieve and requires, in particular, the following:

– achieving a good understanding of the ground conditions ahead of the face (when the required information has not been provided by preliminary investigation, in particular where voids are present in the ground to be excavated, the shield will need to be fitted with some equipment allowing geophysical investigations to be performed through boring ahead of the face);
– equipping the machine with reliable sensors gauges to allow measurement of all variations of key parameters within the working chamber and spoil conveyor, so that guiding of the machine can be continuously adjusted on the basis of the response monitored by the sensors.

6.5.2. Limitation of ground losses along the shield

The annular gap formed along the shield may be reduced:

– by limiting the amount of over-cutting or using variable over-cutting tools (elliptical over-cutting);
– by reducing the total shield length or by installing one (or two) articulation(s) along the shield, although this may create other guidance constraints;
– by allowing, in the shield design concept, for bentonite grouting to be achieved through the shield skin (which may also be used to reduce friction during shield shoving).

It must however be recognized that little flexibility is usually available, given that the machine design options will be restricted by project constraints, technical limitations and compatibility between the different shield functions.

6.5.3. Backfilling of the tail void

Preventing this void from developing is the key issue for settlement control (Carrara, 1995). This requires two preventive actions to be taken:

– longitudinal pressure grouting at the tailskin while:
  - adjusting shield advance in view of the observed grouting levels achieved,
  - using several simultaneous grouting pipes over the shield periphery;
– reducing the tailskin and tail seal thickness as far as compatible with the other machine functions.

This system offers the obvious and essential advantage (in comparison with grouting through the liner segments) to allow filling of the gap as it appears (i.e. as the shield advances). This can be achieved provided that:

– grouting parameters are permanently maintained at the desired level, whatever the rate of shield advance,
– early setting of the grout in the tubes and over the seals is prevented; this may be achieved, for instance, by using grouting products that are not based on cement but offer some cementation capability (e.g. puzzolanic reaction).

7. Observation and monitoring of ground response

This section is not intended to provide details on inspection requirements for settlement control. Only the main principles of instrumentation are discussed hereafter.

7.1. Inspection purpose

Inspection must allow to monitor the deformations and displacements of the ground and surrounding structures, including carriageways and platforms, before, during and after tunneling.

Prior to tunneling works, it is essential to determine the initial condition of all existing structures in the vicinity of the project. This investigation supplements preliminary building investigations and surveys. This information is required to the evaluation of any measurements taken during construction. In addition, in the case of poor condition of the building structure or of limited bearing capacity of the underlying ground, it will provide the owner with an understanding of the building movements out of any influence of construction works.

Measurements taken after construction is complete, will allow checking for the occurrence of long term movements or the return to the initial situation.

During construction, instrumentation allows to check the observed tunneling induced movements against any contractual thresholds and assumptions made in design (Section 8.1).

7.2. Instrumentation choice

The instrumentation approach must be defined in details at the design stage. It must meet the requirements set by the design studies and must be practical to implement (Leca and Clough, 1994).

Engineers should not restrict their views to the most cost-effective equipment, and should include monitoring labor costs in the evaluation. From a practical point of view, it may prove more expensive to deal with frequent monitoring of elementary equipment (e.g. surveying) rather than investing in a comprehensive automatic data acquisition system.

Sufficient details should be provided in the tender documents so that contractors can achieve a proper appreciation of the risk.

In all cases, the engineer should allow for a significant provision for unanticipated specific monitoring that will inevitably be required in urban tunneling.

7.2.1. Monitoring of existing structures

The monitoring equipment should allow the determination of at least three types of movements within the surrounding structures:

- total settlements;
- differential settlements;
- rotations.

Total surface and building settlements can be measured using conventional surveying with a required accuracy in the millimeters. These measurements can easily be made on the outside of buildings or facilities, but prove more difficult internally, particularly with cellar walls or buried utilities.

Differential settlements between two points are obtained by difference between the total settlement values recorded at each point. For the above mentioned reasons, determining differential settlements between building supports is complex, difficult to achieve or even impossible if all settlement markers are to be taken into account.

The effectiveness of the monitoring system in place relies on the ability to take frequent measurements during the most critical constructions phases. The location of settlement markers should therefore be carefully determined to allow this requirement to be met. Should direct measurements on supports be impracticable, provisions should be made to allow monitoring of the settlement trough in the area of influence of the tunneling works.

Direct monitoring of tilt or rotation of a specific structure or parts of structure (windows, lintels, etc.) can be achieved by installing direct monitoring devices such as vertical inclinometers along the bearing elements or horizontal levels on the bearing elements. In the United Kingdom extensive use has been made of electrolevels. They are often linked together in linear arrays to provide real time data on tilt and settlements. This is particularly valuable where it is required to monitor operational railway tunnels.

Recently the introduction of automatic total station monitoring has allowed continuous monitoring of structures both in terms of settlement and horizontal movements.

7.2.2. Ground measurements

Tunneling induced deformations within the ground located between the opening and the surface can be monitored by means of multipoint borehole inclinometers and extensometers.

Correct interpretation of inclinometers requires a fixed reference point. The devices situated on either side of excavation should be anchored deep below the invert level (about one diameter); devices located in the tunnel centerline should be checked for displacements along all three directions. Surveying at the heads of extensometers or deep settlement indicators should be performed at least as often as topographic surveying.

Such devices are expensive, in terms of procurement, installation and monitoring and difficult to implement. For this reason, their location should be carefully assessed. It is however essential not to be over-restrictive in this respect, as the cost of instrumentation remains reasonable in comparison to the cost of construction and even more when compared with construction stoppages.

In addition to the specific monitoring sections associated with the presence of sensitive structures, as identified at the design stage or found in the course of construction, it is absolutely necessary to also provide for monitoring sections of the structure under construction, particularly if it is of linear nature.

The instrumentation plan should at least include one monitoring section for each ground formation. These sections should, as far as possible, be placed in the early part of the project so that sufficient information can be collected within the learning phase and construction parameters adjusted for the remainder of the alignment.

Each section should include at least three pairs of monitoring equipment (settlement indicator + inclinometer), with one being installed in the tunnel center-line and the other two on each side of the tunnel. Experience shows that this is a minimum requirement, and that using two lateral pairs of equipment allows data interpretation to be more reliable, especially when it comes to determining the position of the inflection point of the settlement trough. It is also to be noted that one highly monitored section is more effective than two partially equipped sections.

7.3. Monitoring program

The monitoring program should specify the type, organization and frequency of measurements to be taken, as well as their purpose. Provisions in this respect should be clearly detailed in the tender documents, and adapted by the contractor to their own methods under the engineer's supervision.

The documents should specify, for each work phase, whether continuous monitoring is required. This aspect is very important, as it conditions the choice of the instrumentation system.

8. Contractual aspects

As explained in the previous sections, tunneling inevitably produces ground movements, the magnitude of which depends on the project conditions. In urban areas, the impact of these movements on existing structures should be a major concern for all stakeholders from the design stage through to completion of the works.

In this respect, a comprehensive strategy should be developed by the owner, and provisions made accordingly in the tender documents so that the difficult and costly situations associated with litigation during construction can be prevented.

Based on current practice, a number of contractual strategies can be developed by the owner that fall under one of the following two categories:

- either the contractor is made liable for any damage occurring throughout the tunneling works; this approach may sometimes comprise unrealistic settlement thresholds or 'alibi' criteria;
- or contractual rules for sharing responsibility are applied.

8.1. Usual contractual clauses

It is common practice in France to include in contractual documents for urban underground works, clauses that specify the maximum admissible ground movements in the area of influence of the works. This aims at assigning responsibilities in the case damages are experienced.

As a result, damages related to ground movements falling with the contractual thresholds will be covered by the owner, with the contractor being responsible for damages experienced when the threshold values are exceeded.

The key issue in that case is the determination of the contractual threshold. In some cases, it is set equal to the maximum movement that could be sustained by existing structures, usually factored by a safety coefficient. In other cases, it is arbitrarily fixed by the owner, which may lead to unrealistic requirements.

Usual guidelines require several of the following types of criteria to be implemented alone or in combination:

- total settlement and, in some cases, total heave;
- differential settlement or relative rotation;
- overall tilt;
- extent of the settlement trough.

These guidelines may also indicate the frequency of measurements to be used, or this may be left to be developed in the contractor's Quality Assurance Plan.

Conversely recommendations may be made in terms of an alarm threshold – intended to trigger a review of the construction methods and make the necessary changes when required – and a stoppage threshold.

Finally, one should also be aware of the complexity of the responsibility allocation process when several companies are involved on the worksite. This is particularly true of situations where underground preparatory works are undertaken prior to the commencement of the main contract.

8.2. The position of the different players

During the completion of the underground works, one should differentiate between the players that take part in the construction process and other stakeholders that take a more passive role, although they may happen to become accidental players; the latter includes tenants, landlords or operators of the existing surrounding buildings and facilities.

A special mention should be made of insurers whose position may considerably influence the conclusions of issues arising from damage to existing buildings. Further review would be required of these aspects in the coming years, as practices in this area are still unclear.

Three main players must be considered: the owner, the engineer and the contractor.

8.2.1. The owner

The owner is faced with the challenge of meeting two objectives: minimizing the project costs, on the one hand, and minimizing disruption to local residents, on the other hand. Disruption may occur prior to construction, due to preliminary preventive action, or caused by the tunneling works or required reinforcement of existing buildings.

It may be difficult, at the design stage, for the owner to comment on the likelihood of significant settlements to occur, as other stakeholders may not be acquainted with the reality of tunneling works and may overreact to potential concerns with negative reactions by the residents.

As part of the authorization process (Environmental Impact Statement), the owner has to make commitments at the early stage of the project when a number of uncertainties remain as for the condition of existing structures, with no possibility for detailed investigations because of access permit constraints. As a result, and based on shared responsibility principles, the owner may be inclined to impose very stringent contractual requirements.

This strategy, which results in minimal values of settlement thresholds, may, in some cases, generate adverse effects, either because the imposed requirements prove impossible to meet, which increases the risk of litigation, or because they can only be met at the expense of excessive cost overruns which are not justified in view of the potential extent of damages to be prevented.

8.2.2. The engineer

The engineer is in charge of, among other things, evaluating the potential magnitude of displacements induced by the excavation method that has been selected, as well as the response of existing structures to these displacements.

He only is provided with the role of managing these difficult tasks. However, whatever the effort put in this exercise, predicting ground settlements remains difficult and sometimes uncertain. As often with underground construction, predictive analyses can only provide a general indication as to the likely ground movements.

Two approaches are available, based on the philosophy taken by the owner:

- either making the *a priori* choice to use the criteria wished by the owner, and then adjusting the construction methods to ensure these criteria are met, while resisting the pressure to keep the costs as low as possible,
  or selecting a realistic construction approach, evaluating the excavation induced movements and ensuring that they are sustainable for existing buildings, or else define ancillary techniques to be implemented for the building deformations to remain acceptable.

Attention should be paid in the settlement review conducted by the engineer to a number of elements of different origins, including:

- possible variations in the implementation of construction methods;
- allowance for alternatives proposed by the contractors;
- time dependent phenomena that may develop after completion of the works;
- impact of the work breakdown structure, particularly with phased construction, that may involve different contractors and different contracts.

8.2.3. The contractor

The contractor's approach is based on his experience of similar works and on his knowledge of construction methods.

He is not inclined, during the tender stage, to allow for other precautions than those stipulated by the engineer. This would be incompatible with the bidding schedule that does not provide for the time for additional investigations to be undertaken. In addition, the introduction of additional precautions would contribute to making the bid less competitive, and leave the contractor with little chance to be selected as preferred bidder, at least as long as the concept of best bidder has not been clearly defined, and effectively comes into practice.

Having said this, the contractor cannot ignore that settlement is an issue to be managed and that his expertise is a key element in controlling damages to existing structures. He should not take the risk of underestimating the cost of preventive measures or expecting these measures to be partly relaxed in the course of construction. This is particularly true when a construction alternative is submitted.

One should also bear in mind that, when it comes to litigation, determining the actual liability of each player is an extremely difficult exercise, and that part of the counseling strategy, consists in assessing whether (or not depending which interests are at stake) the observed settlements should have been expected or are the result of construction defects. In the end, experience shows that no player benefits from any contractual uncertainty in this respect.

Disputes in regard to damage caused by tunneling induced settlements should usually be settled without recourse to litigation. However, recently in the UK, an important case was the subject of a complex technically driven trial (New et al., 2005) in the High Court. The Judgment has set a precedent in these matters and serves to clarify the law at least as it stands in the UK, and provides a fully reasoned argument in terms of causation and legal liability in these matters.

9. Possible improvements

It is the interest of all those involved in a tunneling project to use clear and easily applicable rules when it comes to relatively shallow works in the vicinity of existing buildings.

In this section, some options are proposed to simplify design and construction procedures.

The *owner* is usually the only player, together with *the engineer and the related engineering offices,* with sufficient time and budget to study and identify the impact that the construction may have on the environment. He must:

- organize preliminary inspections and studies of the buildings and constructions located in the influence zone of the future tunnel to determine with accuracy their condition prior to construction, and make an evaluation of their ability to sustain movements, thereby limiting the risk for and extent of future litigation;
- commission a comprehensive program of studies to be completed by consultants having expertise in both geotechnical and structural engineering;
- set settlement limits appropriate to the site conditions and nearby structures. The engineer shall define, first, the criteria applicable to each main work phase and, secondly, to the construction process;
- provide, during tendering, all available information, including from the engineer's preliminary studies.

If structural assessments indicate that the proposed work may result in damage, then mitigation measures should be provided. These may be in the form of revised tunneling methods or structural protection by underpinning, ground improvement or compensation grouting.

The contractor must provide assistance throughout the works. All personnel must be made aware of their responsibilities in the control and execution of the construction process. Feedback from manual and automatic survey equipment must be quickly interrogated and immediate action taken if ground movements get beyond design limits.

The contribution of insurance companies may be of three types, the first two of which also apply to *control firms:*

- greater technical clarity when defending their interests, by reverting more systematically to experts specialized in underground works and in soil-structure interaction;
- technical analysis of the risks before finalizing their binding agreements;
- greater diligence in the review of litigious situations, as excessive delays are often found in the completion of damage analyses, thereby impeding the clarification of factual causes and responsibilities. Another consequence of these delays is that it is difficult to control the amount of additional works, which tend to inflate on the basis of the broad argument that safety should prevail.

It is to be mentioned that these proposals are in compliance with the spirit of the recommendations referring to the contractual risk sharing, prepared by the International Tunnelling Association (ITA).

Many major projects throughout the world are currently demonstrating the abilities of the new generation of tunneling machines to control ground movements to limits not previously economically attainable. It is to be hoped and expected that these major advances in tunneling technologies will promote even greater use of sub-surface space in solving the transport, utility and other problems of the cities of the world.

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