www.aftes.asso.fr

ASSOCIATION FRANÇAISE DES TUNNELS
ET DE L'ESPACE SOUTERRAIN

Organization member of the AFTES

AFTES Recommendations

Settlements induced by tunnelling

GT16R1A1

AFTES

TEXT OF RECOMMENDATIONS ON

SETTLEMENTS INDUCED BY TUNNELLING

Published in french version in 1995

Paper presented by
Y. LEBLAIS,
working group animateur with the collaboration of
D. ANDRE, C. CHAPEAU, P. DUBOIS, J.P. GIGAN, J. GUILLAUME, E. LECA, A. PANTET, G. RIONDY

FOREWORD

1. PURPOSE OF THE RECOMMENDATION

2. TUNNELLING-INDUCED GROUND DISPLACEMENTS

2.1. Face stability
2.2. Displacements spreading to the surface
2.3. Parameters of stability during excavation
2.3.1. Purely cohesive soil(clay)
2.3.2. Cohesion less soils (sand)
2.3.3. Cohesive frictional, soils
2.3.4. Rock type soils
2.4. Convergence of the excavation

3. SETTLEMENT INDUCED BY EXECUTION WORKS

3.1. Case of sequential method
3.1.1. Influence of the face behaviour
3.1.2. Influence of the tempory support
3.1.3. Influence of the work phases
3.1.4. Influence of the lining
3.2. Case of shield-driven tunnelling
3.2.1. Settlements ahead and above the face
3.2.2. Settlements along the shield
3.2.3. Settlements induced by post shield/grout loss
3.2.4. Settlements due to lining deflexion
3.3. Water table incidences
3.4. Incidence of the worksite conditions

4. EVALUATION OF SOIL MASS DISPLACEMENTS

4.1. Displacement computation methods around the underground work
4.2. Evaluation methods of surface settlements
4.2.1. Empirical and semi-empirical methods
4.2.2. Numerical methods
4.3. Basic methodology for estimating surface settlements
4.3.1. Evaluation of volume losses around the face
4.3.2. Spreading of displacements to the surface
4.3.3. Transverse settlement trough and maximum settlement
4.3.4. Displacement and surface settlement
4.3.5. Opposite procedure
4.3.6. Longitudinal settlement trough4.4. Horizontal displacements linked to settlements
4.4. Horizontal displacements linked to settlements

5. INCIDENCES OF SOIL MASS DISPLACEMENTS ON CONSTRUCTIONS

5.1. Movements affecting structures
5.2. Qualification of damages to the structures
5.3. Relationships between structure deformation and cracking
5.4. Relationship between structure deformation and the groud movement
5.5. Limits of the structure movements valid as a first analysis
5.6. Tensile deformations admissible by underground utilities
5.7. Study methodology
5.7.1. Phase 1: Investigation of existing buildings
5.7.2. Phase 2: Information synthesis
5.7.3. Phase 3: Damage criteria choice
5.7.4. Phase 4: Modelling
5.7.5. Phase 5: Determination of the allowable displacement limits
5.7.6. Phase 6: Feedback analysis and determination of models according to observations

6. SETTLEMENT CONTROL

6.1. Improvement of the general conditions of the project
6.2. Improvement of ground behaviour
6.2.1. Grouting
6.2.2. Compaction grouting
6.2.3. Jet grouting
6.2.4. Ground Freezing
6.2.5. Drainage
6.3. Stiffening of buildings
6.4. Tunnelling improvements using the sequential method
6.4.1. Face support
6.4.2. Pre-support
6.4.3. Crown support
6.4.4. Underpinning of the upper half cross section
6.4.5. Arch invert support
6.5. Shield improvement
6.5.1. Reduction of decompression ahead of the face
6.5.2. Reduction of decompression along the shield
6.5.3. Decrease of immediate post shield loss

7. INSTRUMENTATION

7.1. Instrumentation purpose
7.2. Instrumentation choice
7.2.1. Monitoring of the existing structures
7.2.2. Ground measurements
7.2.3. Measurements in buildings
7.3. Method of measurement

8. CONTRACTUAL ASPECTS

8.1. Usual contractual clauses
8.2. The opinion of the different actors
8.2.1. The client
8.2.2. The engineer
8.2.3. The contractor

9. CONCEIVABLE IMPROVEMENTS

BIBLIOGRAPHY

FOREWORD

First, this document is designed to inform direct participants in the construction phase (owners, engineers, design offices, contractors...). It is also directed at informing private and public decision makers, or even local residents, 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 claims to be a first stage. In particular, it shall be revised in due course to present assessment methods on settlements understandable to anybody as well as experienced damage criteria. We may assume that with the support of Owners, who are directly interested in the consequences of their works, there will be much experience feedback from the many worksites under way at the time of writing these recommendations.

1. PURPOSE OF THE RECOMMENDATION

The high density of land use of large building projects shall be not overlooked when constructing new public and private facilities. Neither shall be the wish of city dwellers to reconquer nuisance-free pedestrian areas. Both aspects increasingly lead to use the subsoil, whether it be in terms of frequency or density. Boring new underground facilities interferes with the overlying buildings and, more and more, with the existing underground facilities as there is no actual underground land use blueprints.

Of course, it is trivial to remind that the major characteristic of underground works is to be bored in the subsoil. But the main doubts that designers and constructors shall face will be caused by the ground conditions encountered. These conditions will give local residents a clear insight into the existing underground building, whether it be when it is under construction or throughout its lifespan.

As the foundations set the building to the soil, the building's behaviour to the movements imposed by tunnelling works underneath or nearby its axis depends on its geometry, method of construction and structure condition. There lies also another big source of dubiousness as there are few property owners who know about the deformations previously suffered by their building, not to mention in what conditions the foundations were digged.

It then appeared interesting to provide a document to clarify the soil/structure interaction phenomena during underground (not opencast) excavation as well as the means to evaluate, measure, avert and cure them without forgetting the induced contractual problems.

This is the purpose of these recommendations on settlements induced by tunnelling.

Conversely, this document does not provide calculation recipes on foreseeable settlements for the two following reasons:

- evaluating settlements is greatly based on appraisal and experience and remains a matter of specialists,

- to date, research has been developing quickly, both in France and abroad.

Thus, it is convenient to study every specific case by referring at the very best to the own experience of participants and to the many publications available.

2. TUNNELLING-INDUCED GROUND DISPLACEMENTS

First and foremost, it seems necessary to specify that the relationship between the surface settlements and the work depth is neither simple nor linear. Actually, settlements depend on geological, hydrogeological and geotechnical conditions, on the work geometry and underground position as well as on methods of excavation. However, it is clear that a shalow project is often more harmful and requires special monitoring.

Tunnelling disrupts the initial stress field as well as the hydrogeological conditions. Generally, this stress modification is accompanied by instantaneous displacement of the face towards excavation as well as convergence of the tunnel walls (Fig. 1.). In the specific case of soft grounds, the pore pressure field modification may induce long term displacements.

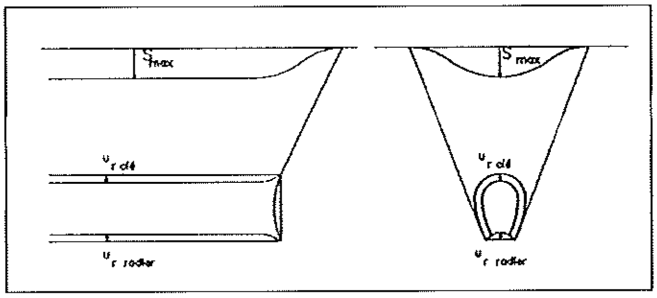


Fig. 1. Displacements of the excavation profiles: basic cross-sections

The magnitude, orientation and location of the soil mass points around the tunnel depend on the soil mechanical characteristics, geostatic stress, surface overloads, hydraulic conditions as well as methods of excavation and support. When the soil mass mechanical capabilities are locally exceeded, there occur many displacements (important magnitude and speed). They often pave the way to yielded zones. This situation is harmful, whether it be for support (dead gravity load) or displacement limitation.

Thus, if the walls are poorly confined, the displacements around the excavation profiles may lead to a fracture zone rear of the face (Fig. 2a.). If the face is not adequately confined, this zone can spread ahead of the face (Fig. 2b.).

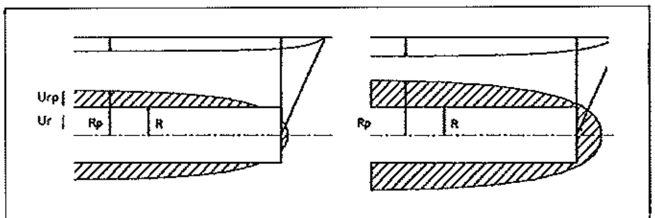


Fig. 2a *Fig. 2b*

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

Knowledge of fracture risks at the working face provides useful data to assess the settlement likelihood as well as an estimate of immediate safety conditions during tunnelling insofar as ground collapses at the face represent one of the main settlement causes and as they are most likely to take place in poor stable conditions.

2.1. FACE STABILITY

The study of the face stability provides indications on the most probable fracture mechanisms and on the parameters to take into account in the study of the soil mass behaviour. Two types of patterns were evidenced according to ground nature.

In the case of clayish soils, the diagram (Fig. 3.) shows that an important part of the soil mass ahead of the working face is affected by the displacements. At the surface, the fracture looks like a crater wider than the tunnel diameter. In this case, experience shows that considerable ground volumes are involved in working face fractures.

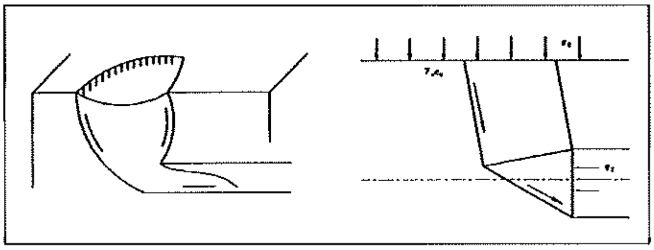


Fig. 3. Face collapse : basic diagram in clayish grounds

In the case of granular soils, the working face fracture creates a chimney of a reduced width above the tunnel alignment (Fig. 4). This second pattern type has been evidenced by centrifugal tests carried out with dry sand.

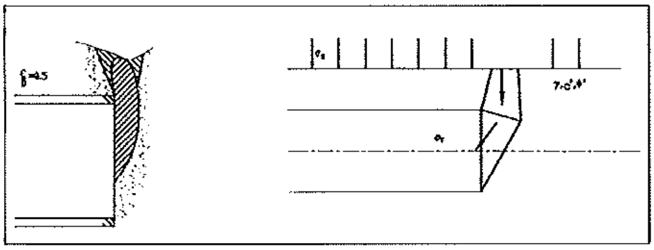


Fig. 4. Face collapse : basic diagram in dry granular soils

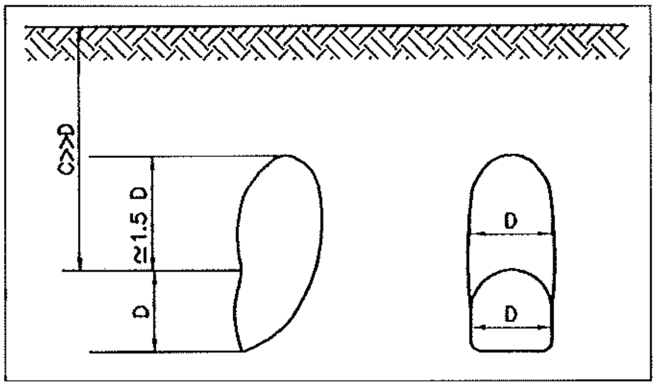


Fig. 5. Secondary pattern. Basic transverse cross-section

These different findings are in line with the results provided by the theoretical studies [5, 6, 13, 23, 24, 25] as well as with observations made on worksites [8]. However, they are based on the analysis of extreme cases and must, of course, be mitigated to take into account the conditions specific to each worksite : inhomogeneous ground layers or water intake. Especially in the case of granular waterbearing grounds, the front face stability will be considerably influenced by hydraulic gradients induced by seepage in the soil mass.

It is convenient to outline that the patterns shown in Figures 3 and 4 correspond to the fracture condition of the ground and illustrate the general tendency of soil mass deformations rather than the displacements to expect during tunnelling.

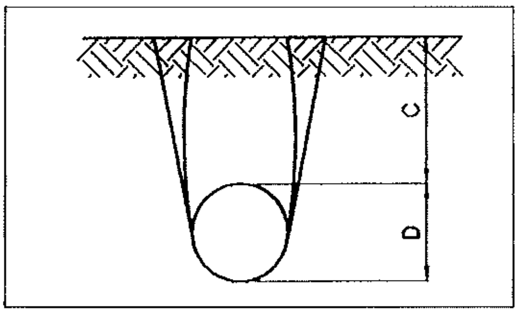


Fig. 6. Primary pattern: basic transverse cross-section

2.2. DISPLACEMENTS SPREADING TO THE SURFACE

From the excavation periphery, the displacements spread to the surface at magnitudes and time intervals depending on the project's geotechnical, geometrical and technological conditions.

To compute the displacements spreading from the tunnel to the surface, the in-situ tests and observations lead to distinguish two patterns shown in transverse cross-sections: the primary pattern and the secondary pattern [36].

The primary pattern (Fig. 5.) occurs when the face is under excavation. It features a deformed ground zone above the gallery. This zone is high about 1 to 1.5 time the tunnel diameter and about as wide as the diameter. Two compressed zones develop laterally following the vertical. When the tunnel is deep enough (C/D > 2.5), the spreading of digging effects towards the surface above the roof is generally reduced [10, 20, 36].

The secondary pattern (Fig. 6.) may occur after the previous one when the tunnel is pretty close to the surface (C/D < 2.5) and confinement is insufficient. This results in the formation of a 'rigid' ground block, bordered by two simple or multiple shear strips that join the tunnel at the surface. The crown displacements and those at the surface, above the gallery axis, are about the same.

This may lead to vertical and horizontal surface displacements throughout the drive termed settlement trough (Fig. 7.).

To make things simple, the 3-dimensional computation is usually obtained by breaking down a transverse trough and a longitudinal trough.

2.3. MAIN PARAMETERS OF STABILITY DURING EXCAVATION

Whatever the subsoil nature, the magnitude and distribution of settlements caused at the surface by tunnelling works depend on the soil mass structure (for instance, alternated inhomogeneous levels), deformability (initial and long term), anisotropy (initial stress ($K_0 \neq 1$), strength and deformability). Of course, the soil mass behaviour is also influenced by the hydrogeological conditions of the worksite. Consequently, the stability time is linked to the ground permeability.

It is clear that a good geotechnical knowledge of the soil mass is essential to estimate these fundamental parameters. The absolu-

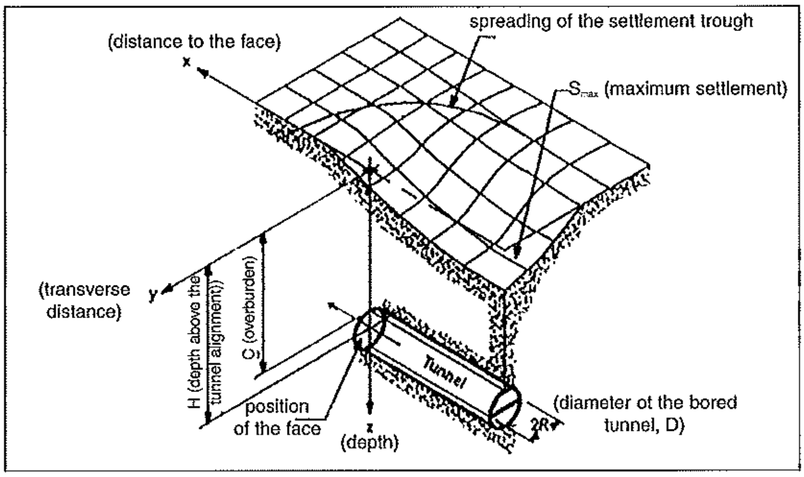


Fig. 7. Three-dimensional settlement trough

te necessity for a soil investigation campaign of good quality shall be emphasized (see on this subject the AFTES recommendation "The choice of parameters and trials useful to the design, size and completion of underground works" [1]).

A few important parameters typical of the ground stability during excavation, not including water seepage, have been evidenced from theoretical and experimental work devoted to the face stability. See Figure 8 for the definition of the different data.

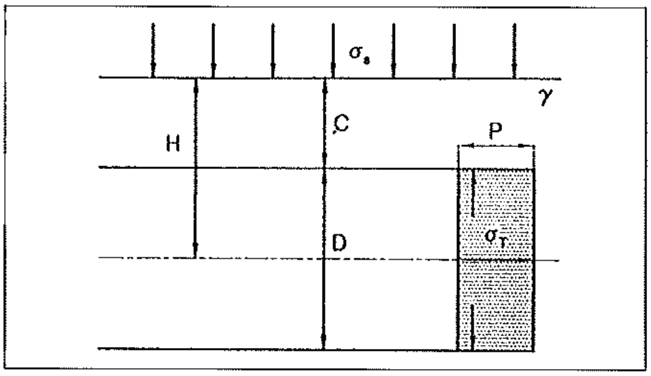


Fig. 8. Stability parameters : notations

2.3.1. Purely cohesive soil (clay)

In the case of tunnelling in clayish ground, the load factor N, defined [3] in the following ratio,

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

H : tunnel axis depth,
γ : ground unit volume weight,
c_u : undrained cohesion of the ground before excavation

appears as the fundamental parameter as regards the instability risks at the face.

Another two parameters shall also be considered:

$\frac{C}{D}$ and $\frac{\gamma D}{c_u}$ C : overburden thickness above the crown
D : excavation diameter

The first ratio reflects the sensitivity of the stability conditions to the work depth whilst the second allows to integrate the likelihood of located fractures on the face.

In the more general case whereby an overload is applied to the surface and the face undergoes support pressure, the load factor N is written as follows:

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

σ_S : overload applied to the surface
σ_T : face support pressure

The observations [38] show that N values ranging from 5 to 7 lead to tunnelling difficulties, or even to breaking of working face. Although these results are to be mitigated in view of the findings of experimental works (with centrifugers) and theoretical works, we can remember that:

- when $N \leq 3$ the overall stability of the work is generally ensured;
- when $3 < N \leq 6$ special attention must be paid to the evaluation of settlement risks since important ground collapse at the face is expected when $N \geq 5$;
- when $6 < N$ on average, the face may fail.

For the other two parameters, the following ranges shall be considered carefully:

$\frac{C}{D} < 2$ need to make a detailled analysis of the face stability

$4 < \frac{\gamma D}{c_u}$ outbrook of yielded parts on the face.

Moreover, special precautions shall be taken if the tunnel is only supported at a certain distance P rear of the face, as the P/D ratio interferes with the stability of the latter [48].

The above parameters, that reflect the soil mass stability condition at the working face, may influence the settlements induced at the surface when the ground is submitted to a stress of nearly the strength limit. Correlations were made between the load factor N and the surface settlements [9].

2.3.2. Cohesionless soils (sand)

In cohesionless soils, the face cannot be stable. However, in these grounds, we often observe the presence of a slight cohesion that may be temporary (capillary tension, for instance).

For these grounds, it is more difficult to draw conclusions on the instability factors insofar as works in this type of ground are only recent. It is clear that the deformability and anisotropy parameters also interfere in the expansion of settlements to the surface [26].

The theoretical studies and tests, conducted without water, seem to indicate that influence of the work depth (C/D) on the face stability conditions is not so important while, on the contrary, the work diameter exercises a strong influence so that the parameters $\frac{\gamma D}{\sigma_T}$ and ϕ' appear determinative in the face stability.

2.3.3. Cohesive frictional soils

If we make a more general analysis of the stability conditions for a frictional, cohesive soil mass (i.e. with a strength characterized by a cohesion, c' and a friction angle, ϕ', four fundamental parameters are identified:

$$\frac{\gamma H}{\sigma_c}, \frac{\gamma D}{\sigma_c}, \frac{\sigma_T}{\sigma_c} \text{ and } \phi' \quad \text{where} \quad \sigma_c = \frac{2c'\cos\phi'}{1 - \sin\phi'}$$

2.3.4. Rock type soils

In rock type soils with a slight overburden, the mechanical strength of the rock mass is scarcely exceeded by the stresses induced by excavation. Stability is linked, above all, to the rock mass quality (stratification, orientation and continuous fracturation, etc.).

2.4. CONVERGENCE OF THE EXCAVATION

In addition to the stability of the excavation face, the convergence of its walls influences the deformations of the soil mass.

It shall be remembered that the essential factor to reduce the excavation wall convergences, which may generate possible settlements, is the immediate installation of an adequatly stiff support the nearest possible to the face, or even ahead of it. On a mere Convergence-Confinement diagram (Fig. 9.), it is obvious that a stiffer support ($K_1 > K_2$) installed closer to the face ($Ur_1 < Ur_2$) will contribute to limiting convergence, while being more loaded.

3. SETTLEMENT INDUCED BY EXECUTION WORKS

Before proposing ways to estimate the displacements due to underground excavation, it proved desirable to identify, from what is currently known, the different settlement sources linked to construction works. Prevention and remedies will be addressed later in this document (§ 6).

Generally speaking, the movements along the excavation alignement start some distance ahead of the face and continue until the support is installed on the ground, or even after. Therefore, we will consider the settlements linked to the methods of excavation at the face, then rear of the face.

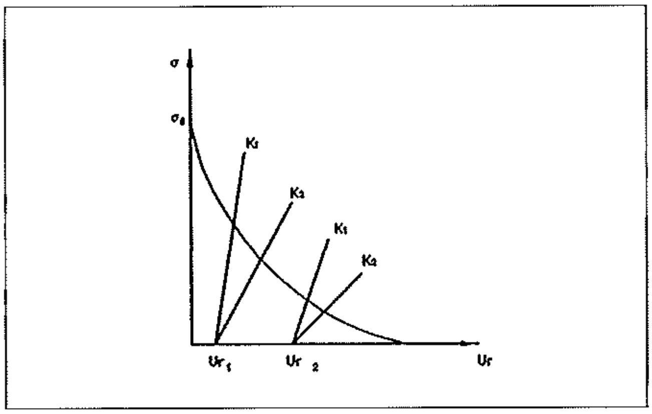


Fig. 9. Influence of the support (stiffness, installation time frame) on convergence

In view of the fundamental evolutions gained through shield tunnelling and the development of such techniques, it seemed necessary to differenciate continuous shield-driven works from that using sequential tunnelling method. The term 'sequential' has been preferred to "traditional" or "conventional" because the latter refers to methods poorly adapted to the control of settlements (arches and wood) and moreover, it does not reflect at all the richness of the recent technical evolutions.

The end of the chapter will roughly deal with settlements induced by water inflow in the subsoil as well as with those induced by the worksite conditions. Finally, it is convenient to underline that the findings below deal with the case of a generic work that we assume is isolated. Indeed, so as to give simple landmarks, we preferred not to complicate this document further, by adding views on the influence of several neighbouring excavations, whether they are simultaneous or not. That may be an additional worsening factor.

3.1. CASE OF SEQUENTIAL METHOD

For works of this type, four major settlement sources can be identified:
- settlements induced by the face behaviour;
- settlements induced by the temporary support nature and conditions of installation;
- settlements induced by the work sequencing of the cross section (phases);
- settlements induced by the permanent lining.

3.1.1. Influence of the face behaviour

Controlling the face behaviour is essential.

The findings regarding the face behaviour clearly show the direct link that exists between the face behaviour control degree and the outbreak of settlements ahead of the excavation face.

3.1.2. Influence of the temporary support

One important result of the project's feasibility study is the choice of a temporary support type. A compromise between the theoretical requirements related to the size and those imposed by the study of excavation methods needs to be found. Two parameters shall be determined:

- the nominal stiffness of the support which must take into account its mechanical capability and installation method, especially wedging;

- the time frame to install the support which depends on the installation distance behind the face.

The combination of these two parameters defines the overall support capability to resist to ground convergence (Fig. 9.) and, accordingly, to limit induced settlements at the surface. Once this theoretical capability is defined, it is still necessary to make sure that it can be achieved through the actual conditions of installation of the support at the worksite.

3.1.3. Influence of the work phases

The work phases may strongly influence the soil mass deformations:

- at the face, according to its area surface;

- in the typical cross section according to the speed at which the support is closed depending on a split transverse cross section and the distance between the face and the support installation;

- in the typical cross section according to the distance at which the lining is installed since it is often, indeed, much stiffer than the support and subject to lesser deformations; its quick installation may contribute to a better longitudinal distribution of the loads thereby limiting ground deformations.

3.1.4. Influence of the lining

The deflection incidence of the lining and, possibly, its wedging on the extrados shall be taken into account, especially in the case of shalow, large spans.

3.2. CASE OF SHIELD-DRIVEN TUNNELLING

Settlements induced by shield tunnelling may be classified into four categories (Fig. 10) :
- settlements ahead and above the face ;
- settlements along the shield ;
- settlements induced by post shield/grout loss ;
- settlements due to deflexion the lining.

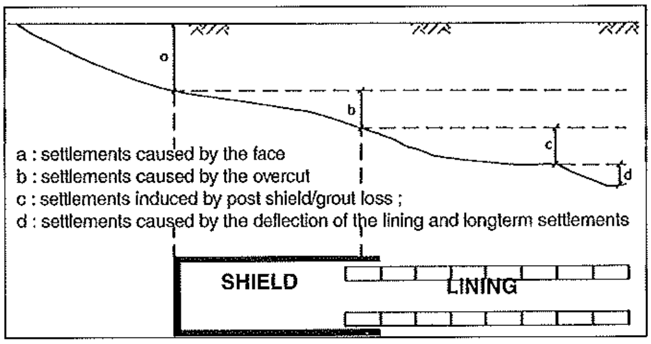


Fig. 10. Evolution of settlements along a shield

3.2.1. Settlements ahead and above the face

Settlements at the face are due to ground displacements forward (face loss) and above the shield towards the excavation chamber. They depend on the chamber confinement, the ground nature and the hydraulic conditions.

3.2.2. Settlements along the shield

When the shield tailskin runs through a stretch where measurements are made, we notice that the ground movements are scarcely stabilised. There is a response time of the surrounding ground that decreases as the roof is thicker. The few observations made seem to show that the spreading of displacements from the tunnel to the surface follows a constant speed for a given ground [36].

Settlements along the shield may be caused by the main following causes:

- overcut due to the tools at the periphery of the cutting wheel that often features an excavation diameter slightly wider than that of the bead so as to reduce friction and facilitate guidance, especially in low radius planimetrical curves;

- shield piloting difficulties, especially their tendency to plough. Thus, to maintain the machine on its pathway, it is generally convenient to constantly maintain a certain leading angle up to avoid it to nosedive (pitch loss). Similarly, the pathway may lead to an horizontal zigzagging. The result is the cutting of a transverse cross section wider than the upright cross section of the shield thereby causing a void (yaw loss) ;

- possible cone-shaped shields;

- bead roughness that may induce, in case of friction and shear in the subsoil, settlements at the crown and ground displacements ahead of the shield.

3.2.3. Settlements induced by post shield/grout loss

At the end of the shield tail, a void may appear between the ground and the segment extrados affected by following phenomena:

- voids along the shield;

- tailskin thickness that varies according to its type (single or double) and the tunnel diameter;

- clearance between the tailskin intrados and the segment extrados to house the tail seal.

The magnitude of surface settlements widely depends on whether the void is properly grouted.

The previous remarks implicitly refer to the case of segment installation within the tailskin. It is very scarcely resorted to expanded segments directly installed on the ground to control settlements as the ground results not confined.

3.2.4. Settlements due to lining deflexion

In the case of a concrete precast segmental lining installed within the tailskin and undergoing the advance thrust of the shield jacks, the necessary thickness is such that the radial deformations of the ring are limited provided the installation to the vault is of good quality.

In the case of more flexible linings (cast-iron segments, for instance), significant deformations may occur due to egg-shaping of the ring thereby inducing additional settlements.

3.3. WATER TABLE INCIDENCES

The experience of underground works is rich with many difficulties or even accidents mainly caused by water inflow. **We must insist that the control of hydraulic conditions is a prerequisite for tunnelling to be performed in good conditions.**

Settlements induced by a water-bearing subsoil can be roughly classified into two categories which, in fact, are not independent.

The first category includes settlements which appear almost immediately with excavation.

Ground water lowering, prior to boring (drains) or as a consequence of boring, may cause immediate settlements not only in horizons or lenses of compressible soils but also in some fissured soil mass. The incidence of lowering varies according to depth and range:

- when localised, they often generate high differential settlements harmful to surrounding buildings;

- when spread, they are generally not very severe (Auber station, line A of Réseau Express Régional (RER) - the Paris express railway network -, St Lazare railway station, Est-Ouest Lien Express (EOLE) - the Parisian East-West underground link).

Water inflow at the face may induce settlements caused by:

- a hydraulic gradient effect inducing ground mechanical deterioration at the face and on the tunnel walls thereby increasing deformations;

- preexisting mechanical instabilities making a worsening factor (washed out karsts, etc.);

- worsened mechanical qualities of the sub-plates inducing, especially when the sequential method is used, sagging of the support and ground confinement losses at the invert.

The second category includes long term settlements, with special sensitivity in soft, compressible grounds. As a result of excavation, some areas of the ground undergo increased deviatory stress and locally, it results in the outbreak of pore overpressures. If the face is completely sealed off, it may cause an identical phenomenon at a wider scale. In view of unavoidable drainage to be conducted throughout tunnelling and/or the work lifespan, either through more permeable soil beds or through the work itself, a consolidation phenomenon will concern the whole drained soil mass, with higher extent in the areas where pore pressures are strongly reduced.

3.4. INCIDENCE OF THE WORKSITE CONDITIONS

This chapter includes the settlements induced by the general worksite conditions, especially vibrations induced by boring and muck removal machines, whether the sequential method or a shield is used. Settlements of this type have been observed during boring works in loose grounds of different types, or in sound grounds but with a poor quality fill layer above.

4. EVALUATION OF SOIL MASS DISPLACEMENTS

4.1. DISPLACEMENT COMPUTATION METHODS AROUND THE UNDERGROUND WORK

To date, the theoretical determination of the displacement field around the underground work remains a delicate issue. It is particularly difficult to give a math representation of the phenomena observed during tunnelling as many factors must be taken into account as well as the three-dimensional pattern of the displacements spreading in the soil mass.

The solution to such a mechanical problem requires, in particular, to determine at the very best the equations representing the intrinsic behaviour of materials (rheological law of the soil condition, lining and possibly grouting products). Actually, several theoretical studies have shown the influence of the behaviour law on the displacement determination around the gallery and within the soil mass.

In France, the convergence study of the excavation profile is dealt with by the Convergence-Confinement method [35]. Let's remind that this method gives a plane deformation-based approach to the three-dimensional issue in tunnelling using a fictitious support pressure dependent on the deconfinement ratio, λ. This ratio integrates the face behaviour, the support position in relation to the face, the method and the work completion quality. The recent findings also allow to integrate the influence of the support stiffness.

The balance of a soil mass disrupted by excavation works (assimilated to a continuous environment subject to external efforts) can be described using two classical families of resolution methods according to how complex is the case under study:

- analytical methods ;

- finite elements methods (FEMs).

Analytical methods are based on simplifying assumptions, both for geometry, lithology (uniqueness of the homogeneous-assumed layer), behaviour laws and the definition of boundary and initial conditions. The scientific literature provides many analytical formulations [9, 12, 40, 44, 52]. Most of the time, the authors were interested in the definition of the new stress field caused by excavation. Because of the computation complexity, few of them showed interest in determining the distribution of the displacement field and the time effects.

On the othe hand, digital FEMs allow to take into account heteregeneous layers, more sophisticated behaviour laws and initial and boundary conditions more similar to the actual conditions or even to the time effect. They are particularly very effective in the study of continuous environments, especially for non-linear problems and complex phasing and geometry. However, three-dimensional computations are still complicated and resorting to too simplifying bidimensional displays may prove less efficient.

4.2. EVALUATION METHODS OF SURFACE SETTLEMENTS

If we leave apart the reduced model approaches reserved to research, there is a distinction between two major method families.

4.2.1. Empirical and semi-empirical methods

These supposedly light methods consist in estimating the surface settlements from a small number of parameters taking into account:

- the excavation dimension and depth;
- a coarse definition of the ground nature;
- the volume loss or the convergence generated by tunnelling work.

The simplest of them consists in making a pseudo-elastic computation and expresses the maximum surface settlement S_{max} as follows :

$$S_{max} = k \cdot \lambda \cdot \frac{\gamma R^2}{E} \text{ with}$$

k : factor dependent on the stress in the soil mass, on its nature and configuration as well as on experience ;

λ : deconfinement ratio ;

R : excavation radius ;

γ : average volume weight of the ground ;

E : average elastic module of the soil mass .

It is clear that this overall ground modelling is often too simplifying as :

- it cannot be rigorously applied to a shallow work (stress uniformity around excavation, admitted if $H \geq 3D$);

- the depth has no explicit influence ; actually, increased deformations in the soil mass, associated with increased stress as depth itself increases, are compensated for by reduced deformations due to the distance from the surface ;

- it expresses a direct proportionality between settlements and deconfinement due to excavation which is often far from reality (§ 4.3.3.).

However, this modelling is useful to underline the essential parameters required for determining settlements :

- tunnel cross section (R^2);

- soil mass deformability (E);

- methods of excavation and tunnelling quality (λ);

- feedback from experience gained (k).

In practice, it is most of the time resorted to empirical methods based on analytical approaches or finite element computations and designed from experience feedback. In general, these methods are light and allow to conduct many parametric studies on the construction influence along the whole stretch. They are thus very useful during the preliminary studies and may even be sufficient to the whole study when the boring site is already well known and the parameters correctly designed accordingly.

This pragmatic approach designed by Peck [38] has developed in Britain above all from the high number of studies on tunnelling works in the London Clay homogeneous horizon [2, 18, 28, 32, 34].

4.2.2. numerical methods

These methods aim at achieving displacements on every point of the soil mass around the excavation and allow to take accurately into account the characteristics of both the construction and the subsoil (geometry, initial stress, behaviour laws, excavation phases, etc.). Within these, FEMs place the emphasis on bidimensional computations in a plane perpendicular to the work axis, in line with analytical approaches and the utilization of the Convergence-Confinement concept.

These methods are very powerful but more complex. It should be underlined that this type of computation also aims at providing strain in the support and the lining. In so doing, even though they are developing quickly, beyond simplified and therefore preliminary models, modelling taking into account all geotechnical, geometric and excavation method data can only be reserved to cross sections carefully chosen.

For shallow works, these methods sometimes show improper spreading of the excavation effects to the surface. Actually, they are naturally not very appropriate for assessing the fracture processes. In cohesionless grounds especially, FEMs for bidimensional models tend to subdivise deformations into a too wide set of elements and this may result in a too wide spreading of the deformations leading to overevaluate the width of the settlement trough and underevaluate its vertical magnitude. Research under way (soil behaviour laws, initial stress condition, true three-dimensional computations, fine meshing) will make it possible to further improve modelling in the future.

It must be remembered that there is always a wide discrepancy between the apparent accuracy of the results obtained with high computation capacity systems and the poor accuracy of the working hypotheses, especially in terms of deformability or work phases. Hence, it is absolutely necessary to test the model sensitivity to the different working hypotheses to avoid misestimations, sometimes serious, and not get lost in unproductive discussions.

Thus, the introduction of secondary parameters in the analysis of soil mechanics behaviour, such as dilatancy for instance, must be envisaged very cautiously. Actually, in the absence of a common determination mode, accepted by everybody, and in view of a variable effect introduction according to the different computation codes, the use of such parameters may result in adverse effects as they may be given excessive importance when their use is too closely connected with the apparent accuracy resulting from the powerful computation used.

It is noticeable that these methods allow, when necessary, to make an interaction modelling of the subsoil, underground work and overlying building. Finally, it must be underlined that the parametric use of these theoretical models as a basis for a retroactive analysis of real cases is very rich whether it is for determining the geomechanical parameters and empirical approaches or for clarifying the interpretations made from in-situ measurements.

4.3. BASIC METHODOLOGY FOR ESTIMATING SURFACE SETTLEMENTS

Taking the outbreak chronology of the phenomena, the proposed procedure consist of three main stages:

(1) evaluating volume losses generated by tunnelling around the excavation face (V_e);

(2) evaluating in what proportion these losses will develop to the surface (V_s);

(3) choosing the settlement trough shape,

determining its width (2B),

deducting the trough depth i.e. the maximum settlement (S_{max}).

4.3.1. Evaluation of volume losses around the face

The Convergence-Confinement method reduces the determination of volume losses around the excavation face (V_e) to the convergence of the tunnel walls. In the case of a tunnel tube driven in a homogeneous and isotropic soil, there is a certain number of analytical resolutions to the problem, that also provide a fairly good preliminary approach to non-circular tunnels using the equivalent radius notion.

Within the approach, the essential parameter is the deconfinement ratio, λ that integrates the volume loss developing at the face and nearby the face.

In the case of ***sequential method*** the deconfinement ratio value is generally fixed for each excavation phase in conjunction with the relevant support phase.

In the case of a ***shield-driven excavation,*** if an overall value of the ratio is sufficient for the lining size, the determination of partial ratios is necessary to give an account on the effect of the different settlement sources (§ 3.2.). This is awkward and requires good experience feedback to determine the spreading of volume losses according to the observations made on the spreading of settlements. As an example, until a recent date, the following distribution of settlements to the surface was observed:

- 10 to 20 % caused by the face ;
- 40 to 50 % caused by the void along the shield ;
- 30 to 50 % caused at the end of the tail seal.

In view of the current technological and methodological evolutions and at the light of the observations made on recent worksites in difficult geometric and geotechnical conditions (extension to Vaise of the Lyon metro's line D; Cairo metro's line 2), it is clear that :

- the absolute value of the observed settlements clearly tends to be reduced (10 to 20 mm) ;
- these percentages are changing and settlements at the tail seal exit may only stand for a small part of the total settlements considering the technological improvements that have been introduced (§ 6.5.3.).

4.3.2. Spreading of displacements to the surface

This second stage consists in determining the settlement trough volume (Vs) that will spread to the surface or to a given depth.

The simplest hypothesis consists in considering the soil as incompressible. In this case, the settlement trough volume equals the volume lost around the excavation profile. Actually, this hypothesis highly depends on the ground nature and the overburden above the tunnel. The hypothesis will be checked especially if the ground is clayish and the overburden is thin.

While there are few increases of settlement volume, there are many volume reductions between the tunnel and the surface. As an example, they may be caused by:

- an important overburden above the tunnel softening deformations up to 80 %;
- a stiffer layer over the boring horizon (slab effect);
- an overlying dilating horizon (dense sand).

It is clear that there are as many specific cases as completed works. Therefore, it is difficult to give general relationships between the settlement trough volume and the ground volume lost around the tunnel. Refer to the abundant bibliography and, as an example, to Fig. 11. computed from measurements made on a few French sites excavated with closed-face shields.

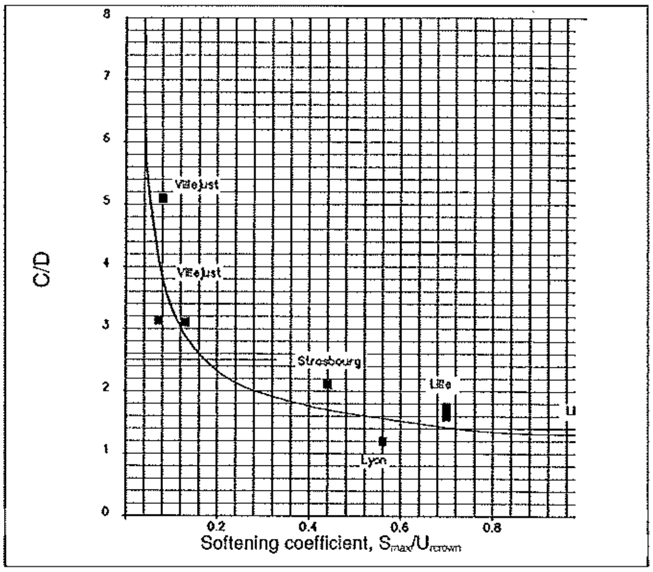


Fig. 11. Softening coefficient according to the geometry of the tunnel bored with a shield

The time frame for settlements to appear at the surface and be stabilised varies very much according to the project configuration. In the absence of a synthesis still not available in France to date, we advise, in this first stage of our recommendations, that the reader refer to the existing literature on feedback gained from experience.

4.3.3. Transverse settlement trough and maximum settlement

The abundant research conducted in the United States and in the UK [2, 16, 46] have shown that the shape of transverse settlement trough is generally fairly well represented by a Gauss curve (Fig. 12.).

B	: half trough width
i	: distance of the inflection point of the settlement trough to the plane
V_s	: trough volume
S_{max}	: maximum settlement at the surface
R	: excavation radius
H	: depth of the tunnel axis

In this case, the settlement S_y at the distance y is given by :

$S_y = S_{max} . \exp\left(-\frac{y^2}{2.i^2}\right)$ (in particular for $y = i$ $S_y \cong 0{,}61\ S_{max}$)

This allows to express the trough volume in relation to its width and to the maximum settlement at the middle:

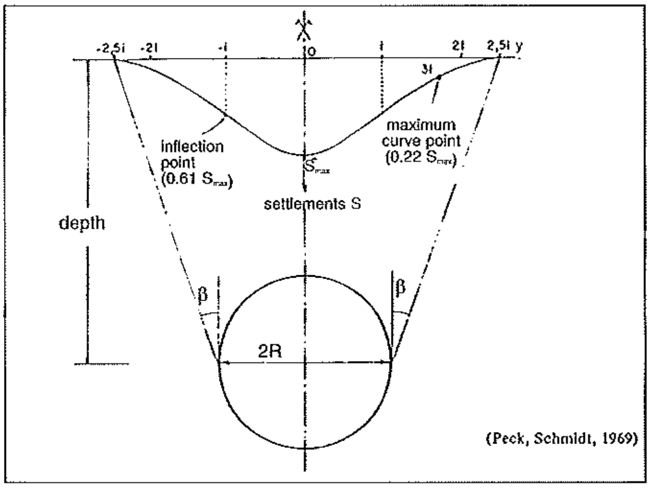


Fig. 12. Idealized transverse settlement trough

$$V_s = B.S_{max} = \sqrt{2\pi}.i.S_{max} \cong 2.5\, i\, S_{max} \qquad \text{hence}: S_{max} = \frac{V_s}{2.5\, i}$$

The maximum settlement is determined from two parameters:

- the trough volume, itself related to the volume loss around the excavation;

- the distance of the point of inflection of the trough to its centre (or its half width).

Within this framework, several researchers tried to empirically relate i/R or B/R to H/R and to the ground nature [2, 38]. Here again, see the bibliography.

In the light of observations on sites and digital studies, it appears that the trough width widely depends on the ground characteristics and the project geometry (C/D) and much less on deconfinement which, on the contrary, strongly influences S_{max}.

4.3.4. Relationship between crown displacement and surface settlement

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

Several researchers have proposed formulas to calculate $S_{max}/U_{r\,crown}$ roof according to H/R and a parameter varying with the ground nature [43]. Each formula has been designed in a specific environment that should be in mind in case of use. In particular, the choice of the parameter associated with the ground deserves attention because it can integrate many other factors.

It should be remembered that another direct evaluation type of surface settlement can be carried out from a typical pseudo-elastic computation (§ 4.2.1.).

4.3.5. Opposite procedure

It may be interesting to start from what is admissible at the surface (cf. § 5.) to go back to the volume loss that can be tolerated above the tunnel alignment. This opposite procedure considers 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. Longitudinal settlement trough

The text above deals with the settlement determination after excavation. It is also convenient to pay attention to settlements ahead of the face as if their magnitude is less than S_{max}, the longitudinal trough is perpendicular to the transverse trough. During excavation, this results in the building undergoing strains in the direction of advance.

4.4. HORIZONTAL DISPLACEMENTS LINKED TO SETTLEMENTS

Settlements are accompanied by horizontal displacements that also damage the existing buildings (Fig. 13.).

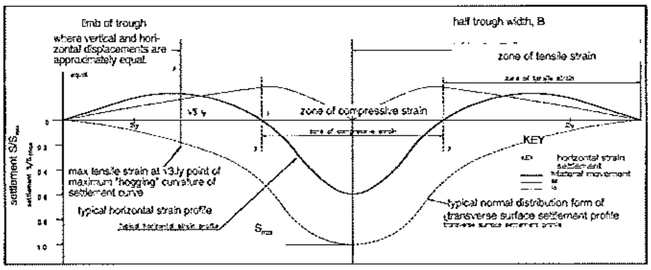


Fig. 13. Vertical and horizontal deformations and displacements

Still speaking from the hypothesis of a normal distribution of ground settlements, the main following values will be retained:

- horizontal displacement at the distance y : $\delta_{hy} = \frac{k}{(H-z)}\, y\, S_y$

- horizontale deformation at the distance y : $\varepsilon_{hy} = \frac{k}{(H-z)} \left(\frac{y^2}{i^2} - 1\right) S_y$

5. INCIDENCES OF SOIL MASS DISPLACEMENTS ON EXISTING STRUCTURES

Whatever the method of excavation used, displacements around the excavation develop in the soil mass and may spread to the surface. These displacements, according to their magnitude, expansion, direction and spreading speed may cause disorders in the buildding located in the vicinity of the tunnel (buildings, engineering structures, carriageways, underground networks, subways, etc.).

It should be remembered that there is an interaction between the building and ground movements and that the construction stiffness tends to reduce the structure displacements compared with the ground displacements alone.

5.1. MOVEMENTS AFFECTING THE CONSTRUCTIONS

Experience shows that old masonry structures deform such as the foundation ground. So is the case for most of the constructions founded on footings or isolated shafts.

On the contrary, the more recent structures, in reinforced concrete for instance, that are laterally supported by peripheral tie beams deform laterally less than the foundation stratum. The stiffness to bending of these beams induces a distortion of these structures more reduced than that of the soil, all the more as the foundation supports are continuous (long strip footings, invert).

Stiff constructions show good shear strength and tend to be subject to inclination rather than distortion. This capability depends on their height (number of floors), the number of openings and the structure type (concrete shells, beams and poles, etc.).

The position of the construction on the settlement trough strongly influences the movements to which it is subject (extension and hogging on the convex surface of the trough; compression and sagging on the concave surface). As an example, Fig. 14 gathers some idealized diagrams on the behaviour of buildings, either narrow or long, according to their configuration in relation to the settlement trough.

Thus, it is likely that a structure nearby the tunnel under construction will undergo the various following movements:

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

The main parameters of the vertical movement in a construction are defined in Fig. 15.

with:

- L : construction (or element) length in the direction of the 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 : construction tilt
- ϕ_{BC} : BC segment rotation
- β_{BC} : relative rotation (or angular distorsion) of the BC segment ($\beta_{BC} = \phi_{BC} - \omega$) ;
- α_C : déformation angulaire en C
- Δ_{AD} : relative deflection = maximum displacement relative to the line joining points A and D
- Δ_{AD}/L_{AD} : Deflection ratio

Note: the relative rotation is a measurement of the shear distortion of the structure; the relative deflection and bending distortions are often correlated.

The main parameters of the horizontal movement of a construction are defined in Fig. 16

- ρ_{hA} : horizontal displacement in A
- ρ_{hB} : horizontal displacement in B
- ε_{hAB} : horizontal deformation between AB ; ($\varepsilon_{hAB} = \dfrac{\rho_{hB} - \rho_{hA}}{L_{AB}}$)

5.2. QUALIFICATION OF DAMAGES STRUCTURES

Usually, damage to constructions is classified into three categories:

- architectural damage that damages the visual appearance;
- functional damage that disrupts utilization;
- structural damage that damages stability.

Damage to constructions is caused by cracking of materials with poor tensile strength such as concrete, mortar and, a fortiori, plaster and different coats (materials making underground ducts are analysed in a case dealt with separately). The failure of supporting structures may occur directly as a result of excessive cracking or excessive load transfer onto the reinforcements. To a lesser degree, cracking is harmful to the structure durability by favouring, for example, steel corrosion.

The crack width is then the important parameter to assess the damage. To do so, we will refer, in the absence of a French sour-

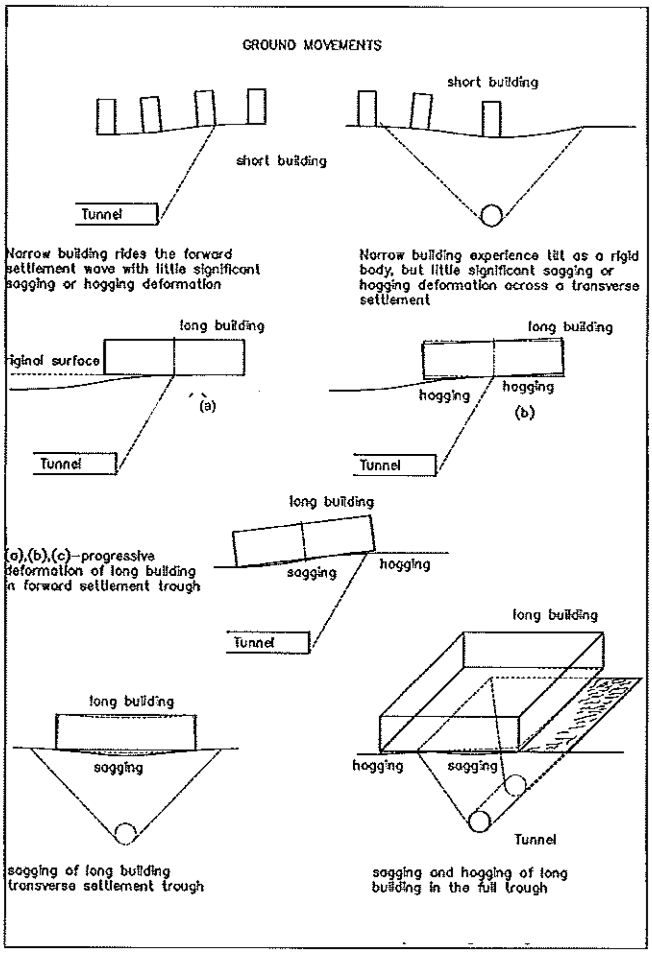


Fig. 14. Several idealized behaviours of buildings from [2]

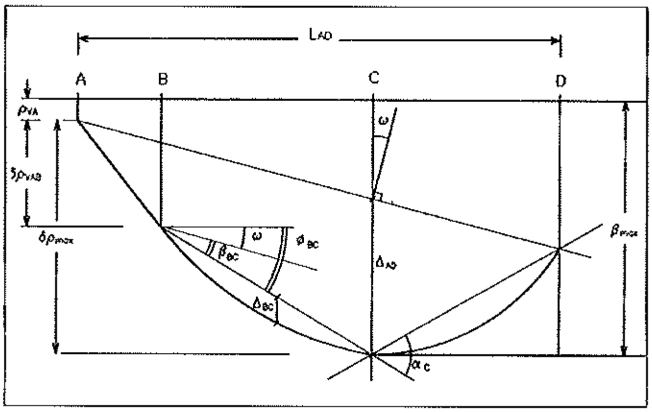


Fig. 15 Vertical movements undergone by the construction

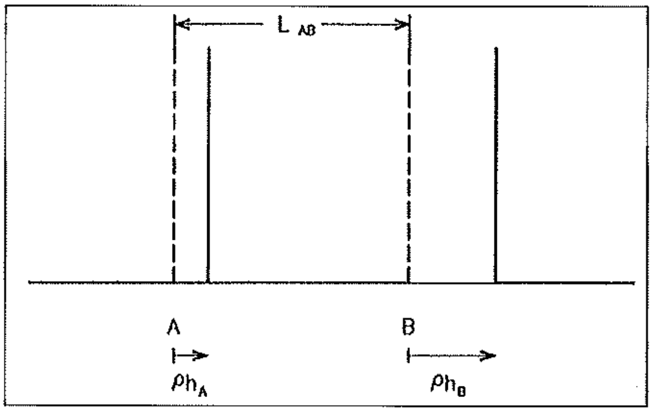


Fig. 16. Horizontal movements suffered by a construction

ce to date, the classification shown in Table 1 which is the transcription of the rules adopted by the British [54].

Damage type	Damage degree	Damage description	Crack width in mm (1)
0	Negligible damage	Microcracks	< 0.1
1	Very slight damagee	Esthaetical	< 1
2	Slight damage	Esthaetical, to be treated	< 5
3	Moderate damage	Functional	5 to 15, or several cracks > 3 mm
4	Sereous damage	Structural	15 to 25 (2)
5	Very serious damage	Structural	> 25 (2)

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

Table 1. Classification of visible damage that may affect a common construction

This classification is first designed for practical use and in this purpose, it is partially based on the repair easiness:

Type 1: Internal cracks can be easily treated during a normal refreshing of the decoration; the rare external cracks only are visible by conducting an indepth inspection;

Type 2: Internal cracks can be easily filled up but they require internal roughcasting ; cracks may be visible outside and require repointing of the masonry to ensure tightness;

Doors and windows can slightly rub;

Type 3: Internal cracks must be open before filling them up; external cracks may be harmful to the tightness lifespan and quality as well as thermal isolation;

Cracks may cause important inconvenience to residents (Serviceable Limit Condition) reflected as deformations of door frames, possible pipe break, etc.

Type 4: Cracking may threaten the residents' safety (Ultimate Limit Condition) and the structure stability;

Important repairs are necessary and they may even involve the replacement of wall sections, especially above the openings ; doors and windows are twisted, the floors are no longer horizontal, supporting beams may be damaged, utilities are broken;

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

This empirical classification may be deemed too simplifying since:

- it undoubtedly refers to classical brick and rubblework buildings, with or without supporting framework, rather than recent, very rigid reinforced concrete buildings;
- works with very harmful cracking shall be considered separately i.e. reservoirs or works in waterbearing grounds, etc. ;
- evolution of damage in types 4 and 5 widely depends on the structure design (latticed steel structures are particularly resistant);
- it does not take into account damage not induced by cracking such as the consequences of deformed or fractured service mains running through the structure.

On the contrary, it provides a good assessment for old city buildings which prove the most sensitive and geographically the most likely concerned by an undergound metro or road route.

5.3. RELATIONSHIPS BETWEEN THE CONSTRUCTION MOVEMENTS AND CRACKING

The above classification is based on a posteriori observations and is not linked to the causes of the disorders. A link has been created by introducing the maximum internal extension or critical extension, ε_{crit} [55] undergone by the construction (or one of its components) prior to the outbreak of visible cracks. This internal extension may either be due to bending (lateral extension, ε_l) or shear (diagonal extension, ε_d). Fig. 17. illustrates it by assimilating construction to a thick beam.

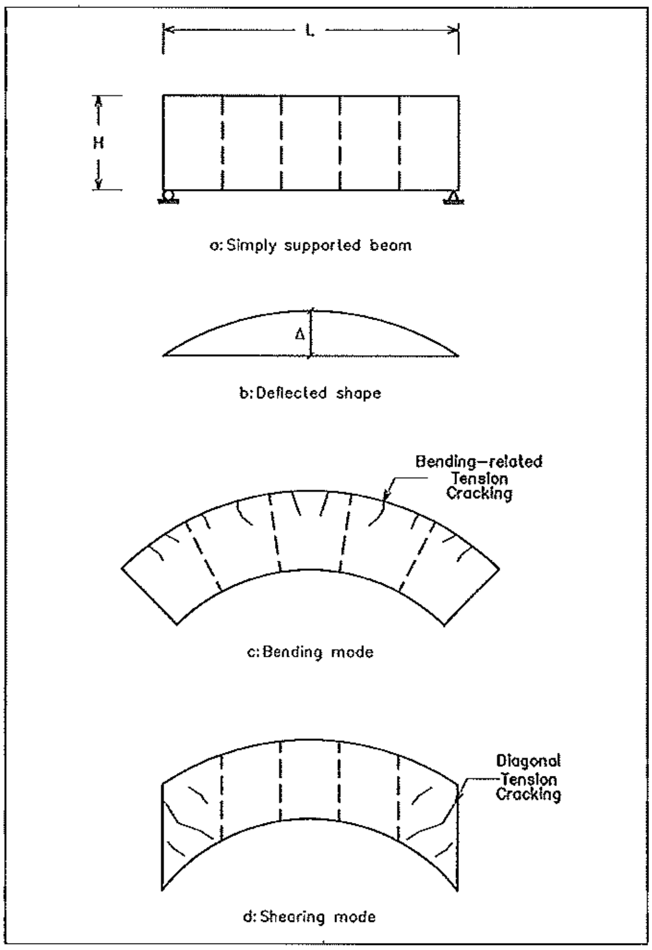


Fig. 17. Thick beam analogy

Work [53] based on the same modelling allowed to describe a relationship between, first, the critical extension (ε_{crit}) and, secondly, distortion (β) and the horizontal extension (ε_h) imposed by the ground movements. For common constructions, the values of Table 2 represent this relationship.

Damage type	0	1	2	3	4 et 5
ε_{crit} (‰)	≤ 0,50	0,50 < ≤ 0,75	0,75 < ≤ 1,50	1,50 < ≤ 3,00	3,00 <

Table 2. Relationship between critical extension and cracking

This critical extension cannot be directly measured. It thus would have been interesting to give the corresponding ranges of the other two parameters in the relationship. Considering the number of parameters influencing on the behaviour of a construction near of underground works, it was decided not to give such correspondaing ranges, which would have generalised particular values.. To get further information, it is recommended to read carefully reference [53].

5.4. RELATIONSHIP BETWEEN THE STRUCTURE DEFORMATIONS AND THE GROUND MOVEMENTS

A construction undergoing the influence of a neighbouring excavation, either underground or open cast, show more sensitivity to the outbreak of differential settlements than if subject only to its own weight. This is due to additional deformations imposed by movements in its foundation stratum. It is noticeable that deep foundations too close to the tunnel alignment may make the risk of structure deformation bigger.

Therefore, the behaviour of a construction strongly depends on its position on the transverse settlement trough (Fig. 14.) that conditions the extension to which it is submitted, in particular:

- in the case of a building located above the tunnel alignment, the diagonal extension, ε_d blank prevails on the longitudinal extension, ε_l which generally is a compression. In the particular case of a low, stiff building on a narrow trough , ε_l blank may be important 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 difficulty to determine accurately in practice the distribution of ground movements to the surface and to make a modelling of the real mechanical characteristics of the structures (see § 5.7.4.), we propose in annexe 1, so as to link tunnelling effects to their consequences, a relationship between the damage criterion to constructions (ε_{crit}) and the average slope of the settlement trough (β_{aver}) under the foundation stratum of the construction.

This approach is also based on the thick beam analogy. It leads to retain, as a first approximation, the correspondences shown in annexe, without bias on the deformation behaviour or the real cracking pattern.

5.5. LIMITS OF THE CONSTRUCTION MOVEMENTS VALID AS A FIRST ANALYSIS

It clearly appears that an absolute settlement criterion is insufficient to describe alone the sensitivity of the overlying building to the displacements transmitted through the subsoil, unless we want to impose a very low limit.

As a first analysis, refer to article 2.4.6. - par. 7 of EUROCODE 7 - Part 1 (ENV. 1997-1:1994) [56], quoted hereafter in extenso, as regard its novelty at 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 condition of the structure. A 2 ‰ maximum relative rotation is acceptable for many structures. The relative rotation for which an ultimate limit condition is likely is about 6.67 ‰;

- for common constructions on isolated foundations, total settlements of 50mm and differential settlements of 20mm between adjacent columns are often acceptable. Wider total and differential settlements may be admitted if relative rotations remain within acceptable limits and if total settlements do not cause problems to utilities, or hogging, etc.;

- the above indications on limit settlements apply to common routine constructions. It is convenient not to apply them to unusual structures or buildings or those for which the load intensity is extensively not uniform.

Many indications about less common constructions or structures shall be found in the bibliography. In addition, tilt (ω) of a tall building is visible from the 4 ‰ value.

Caution : tilt effects of the building on the functionality shall be inspected as well as even without cracking, a serviceable limit condition may be exceeded (lift, etc.).

The damage types defined above shall be related to the EUROCODE indications and the British practices [59] as shown in Table 3.

Damage types	Average slope of the settlement trough under the building (‰)	Maximum settlement of the building (mm)
1	≤ 2	≤ 10
2	2 < ≤ 4	10 < ≤ 50

Table 3 - Range of serviceable limit condition for common constructions

5.6. TENSILE DEFORMATIONS ADMISSIBLE BY UNDERGROUND UTILITIES

The concept of underground utilities involves service mains such as potable water, sewerage, energy (gas, power, oil, etc.) and public or private underground transport infrastructures. The structures involved are very different as for their size, design and depth. However, their long length is the general geometric feature in relation to their transverse cross section which is roughly ring-shaped.

The behaviour of utilities undergoing movements of the soil mass through which they run is a tough problem of soil/structure interaction.

There are not many large diameter utilities (> 2m). They justify case by case studies by means of sophisticated modelling to assess the impact of a nearby underground excavation. The value of admissible movements shall then be determined.

This is different for a great number of highly sensitive service mains. Their sensitivity degree to ground movements (horizontal and vertical) widely depends on the nature of their lining (concrete, cast iron, steel, ductile cast iron, PVC, PE, etc.) and their gaskets. As a comparison with the values given in Table 2, the expansion limits corresponding respectively to the 'Serviceable Limit Condition' and the 'Ultimate Limit Condition' of the service mains equal 0.3 ‰ and 1 ‰ for cast iron and lining concrete, 0.5 ‰ and 1 ‰ for steel, 1 ‰ and 2 ‰ for ductile cast iron and 6.7 ‰ and 20 ‰ for plastic materials.

In fact, the relative long length of ducts, both associated with their cross section and the settlement trough size, makes the inner expansion induced by differential settlements relatively limited, about 1/10th the duct average slope. In addition, the strong longitudinal stiffness of linings, generally precast rings installed with or without flexible gaskets, causes horizontal ground displacements to generate only slight additional deformations. We can infer that, in most of the cases, ducts in 'fragile' materials only (cast iron or concrete) shall be considered to determine the admissible settlements.

In addition to the study of the common part of a service main, the consequences of the differential displacements of the duct and structures to which it is connected in the influence zone of the planned underground tunnel shall be carefully examined.

Moreover, the study shall take into account that the cost to maintain or partially replace ducts may be relatively low especially if it is scheduled according to tunnelling.

Annexe 1 - Coarse relationship between the construction deformations and the ground movements

Construction	low height ≤ length	high height > length
in the vicinity of the tunnel alignment (compression zone)	$\varepsilon_{crit} = 1/3\ \beta_{aver}$	$\varepsilon_{crit} = 1/2\ \beta_{aver}$
away from the tunnel alignment (extension zone)	$\varepsilon_{crit} = \beta_{aver}$	$\varepsilon_{crit} = 2/3\ \beta_{aver}$

The results of this simplified approach may appear hardly realistic in view of observations undertaken in similar cases. In all cases, anyone shall be critical at the results obtained.

5.7. STUDY STEPS

The proposed procedure to study the incidence of an underground structure project on existing constructions can be broken up into six phases. Geotechnical investigation shall be dealt with separately.

5.7.1. Phase 1, Investigation of the existing building

It is a data survey and collection phase on the nature, configuration and condition of the building and utilities together with a topographic measurement and technical expertise campaign.

It is convenient to properly define the actual condition (zero condition) of each construction prior to work debut and, if possible, to review the previous history of the building and in particular, the movements already suffered. Although essential, it is not the easiest task. Endorsement by everyone of this identical classification based on cracking should allow to make many inventories of fixtures less subjective.

It is recommended that this phase include a preventive emergency proceeding to give records a strong judicial base.

5.7.2. Phase 2 : Information synthesis

This phase consists in adopting a typologic classification of the building and utilities according to the nature, purpose, value, size, design, age and current condition of the elements making up the whole thing. If possible, a division into homogeneous zones also integrating the geotechnical data collected during the survey shall also be associated.

5.7.3. Phase 3: Damage criteria choice

The purpose is to clarify the objectives to be reached in terms of damage limitation and to convert these objectives into accurate criteria, useful to the designer.

If the previous inspection campaign provided data on the condition of the building prior to work and led to cracking reports, it is convenient to deem one of the proposed ε_{cm} limit values as the initial reference condition. In this case, the admissible expansion value throughout tunnelling shall be reduced by the initial value.

When the criterion is chosen, the physical possibility to make measurements at the worksite so as to check this criterion shall be taken into account. Except for particular cases, it is often easier to take as a base an average slope for a similar trough, the geometry of which will be determined on-site from the topographic readings on carriageways and buildings.

5.7.4. Phase 4: Modelling

Modelling must link the ground movements undergone by the building to the related deformations.

The deformations of the building are assessed, most of the time, by subjecting it, through the foundations, to the ground movements resulting from excavation without taking into account the reciprocal influence of the structure stiffness. This simplifying and conservative approach reflects fairly well how quick settlements develop in the short term without structure adaptation.

Settlement studies conducted during the design phase shall enable the engineer and the client to assess tunnelling risks associated with their project. They then shall widely resort to parametric studies; whether it be for geotechnical variables, building variables or the incidence of modified boring methods.

So as to limit the number of detailled examinations, graduation criteria shall be defined in modelling. As an example, one can proceed as follows:

- in the first stage, the settlement values calculated in virgin grounds will be applied to the building. All buildings situated in the zone where the average trough slope is less than 2 ‰ and settlements less than 10 mm will not be studied further except those with an obviously critical zero condition;

- in the second stage, the shortlist of buildings will be classified according to their predictable cracking condition under the action of excavation; the buildings entering categories 1 and 2 will not be studied further;

- in the third stage, the remaining buildings, classified into category 3 and beyond, will be examined one by one according to their condition and position in relation to the project; according to the assessment made by the designer, a soil-structure interactive modelling shall be made.

5.7.5. Phase 5: Determination of the allowable displacement limits

In this stage, the objective is to determine the contractual values that will have to be respected during works.

The constraints not related to the project (human, cultural and legal environment) and the economic criteria both determine the nature of the admissible damage and the further possible work (prevention and remedies). The value of the proposed limits shall take it into account.

It is not always possible to limit the prescriptions to the respect of a single criterion of admissible movement, unless by proving too restricting when fixing this criterion. The summary included in phase 2 is particularly important to get contractual criteria well adapted to the reality of the constructions needing protection.

Limit values must never be considered as invariants; they are first of all monitoring indicators that need to be continuously reconsidered according to the actual behaviour of the above building during tunnelling work (a tolerance range should be determined as it is not required to validate solutions with a 0.1 ‰ accuracy !).

In this connection, a **warning limit** and a **work halt limit** shall be defined for each project.

5.7.6. Phase 6: Feedback analysis and determination of models according to observations

It is clear that these settlement analyses are not a matter of exact sciences. It is thus necessary to schedule follow-up measurements of the work and related incidence (see § 7).

It is absolutely essential, in the study process, to check the a-priori forecasts by analysing the results of the on-site observations.

6. SETTLEMENT CONTROL

Obviously, it would be more satisfying to predict, prior to the work start, all the precautions to reduce tunnelling effects to a minimum. However, this optimal situation is not easy, whether it be technically or economically, due to uncertainties, in the study phase, related both to the ground behaviour during excavation and the above building condition.

The current experience feedback recommends to schedule, throughout studies, the reasonable prevention measurements to implement prior or after work as well as the range of possible solutions to apply in case of difficulties throughout excavation.

Several methods to limit settlements, or their cause, are described hereafter. The solution principles and their limitations are reminded only. Refer to the specialized literature for further information.

It is difficult to clearly classify the methods between prevention and solution since the distinction is most of the time subjective and depends above all on the moment when the decision is made.

6.1. IMPROVEMENT OF THE GENERAL CONDITIONS OF THE PROJECT

During the preliminary design of the project, one shall investigate at the very best the geometrical settlement conditions by looking for:

- the thickest overburden provided deep tunnelling does not lead to cross worse geological horizons;

- tunnelling in good mechanical quality layers provided their thickness is sufficient (one tunnel diameter at least above the crown); should this not be the case, it is better to excavate under the stiff layer and benefit from a slab effect rather than disturbing it by holing through it;

- the smallest transverse cross section. This recommendation often leads, for a tunnel, to choose between a single-tube or twin-tube solution; the response varies according to the crossed grounds, evolution of the face sealing technology and budget changes (for instance, a secondhand shield becomes available). If the twin-tube solution is often recommended, the distance between both tubes must be sufficient to avoid cumulative settlements ;

- the least winding route for shield-driven tunnel.

When the method of excavation is chosen, we shall also keep in mind that settlement magnitudes are often bound to the phases of work to be halted or slowed down.

6.2. IMPROVEMENT OF GROUND BEHAVIOUR

Ground behaviour improvement may be obtained by modifying the soil mechanical and/or hydraulic characteristics. We shall only remind hereafter the general data on techniques supposed well-known by engineers.

6.2.1. Grouting

Massive grouting of the ground may increase cohesion (consolidation grouting) and reduce permeability (watertightness grouting). Effectiveness depends on the grouting capability of the ground and grouting conditions (see AFTES recommendation [61]).

It may be carried out from the surface, should the site allow it, or from within the tunnel which reduces the number of cycles. In the particular case of a shield drive, necessary arrangements must be scheduled when the machine is assembled.

This technique may induce ground heave risks in case of uncontrolled failure, especially for shallow, inner-city routes where the geostatic stress does not allow high grouting pressures. Oddly enough, actors are much less sensitive to heaves than settlements whilst the damage caused are of the same nature as heaves come in addition to settlements.

It shall be convenient to watch for the mid-term behaviour of grouting. Actually, in the case of gel grouting undertaken several months before work start, the product degradation (synaeresis) may make the mechanical treatment less efficient.

We shall remind that pollution risks of the water table must be examined according to the type of product used.

6.2.2. Compaction grouting

In the case of open grounds, such as fills, for which classical grouting would lead to use big quantities of grouting products without ensuring effectiveness or, in some cases of little compact grounds, a noticeable improvement of the overall stiffness may be obtained by grouting a dry mortar from boreholes.

This technique improves as a whole the mechanical characteristics of the grounds. It may be used from the surface and possibly as a building underpinning process. Effectiveness shall be controlled by a stringent, topographic follow-up that may be adjustable according to the outbreak of surface heaves.

When grouting is made during tunnelling simultaneously with the drive, it will be termed compensation grouting [62, 67].

6.2.3. Jet grouting

The method principle consists of the very high-speed spraying of a grout jet from a set of drill pipes previously drilled in the ground. The grout jet, more or less thin and rapid depending on the techniques (simple, double or triple jet with or without prewash), destructures the ground in-place at a variable distance according to the compactness of the latter. The grout mixes with the destructured ground to create a stabilised soil column in place. The column diameter varies from 0.30m to 1.20m according to the technique used as well as the ground nature and contents.

The treatment may be carried out from vertical, inclined or subhorizontal trial borings. In the latter case (simple jet), the treatment may be applied from the tunnel face but it is convenient, in fine grounds, to pay attention to the adverse effects of unwanted pressurization of the cavity being cut (violent failure and important heave).

When ground improvement is desired, this technique may replace grouting when the ground is too fine. The technique proved effective and according to the used lattice, it may lead to a total substitution of the grounds in place. However, the constraints to use it (energy consumption, processing of excavated material and muck removal, momentaneous bearing capacity loss before grout setting) make necessary to thoroughly think about it before using it.

6.2.4. Ground freezing

The principle is to build a shell or a frozen ground vault around the tunnel extrados. According to the power capacity of the freezing system, the entire tunnel cross section may be frozen. The technique may be used in almost any grounds featuring permeabilities of less than 10^{-3} m/s.

Whether freezing is carried out from the surface or the working face, the main difficulty lies both in the control of tunnelling deviations to install freezing tubes (featuring a range limited to 50 m) and in the control of important groundwater inflow.

If ground has improved tremendously as far as stability to tunnelling is concerned, vigilance is required because this technique may cause, in view of the migration of pore water to the freezing source, heaves during freezing as well as long term settlements once freezing is completed and alteration of the characteristics of the unfrozen grounds.

6.2.5. Drainage

Control of destabilising gradients towards the face may be obtained by undertaking a general groundwater lowering from the surface or drainage operations from the face. Arrangements made shall make control possible as far ahead of the face as possible.

In the case in which the ground is likely to undergo consolidation settlements or in which dewatering would be a factor for destabilisation (karstic filling), the resort to drainage shall be preceded by an assessment of the possible consequences, with or without drainage.

6.3. STIFFENING OF BUILDINGS

In order to reduce the overlying building sensitivity to movements induced by excavation, it may be interesting to reinforce, prior to tunnelling, the existing structures. As an example, we suggest:

- chainages at the foundations to reduce sensitivity to lateral expansions;

- front wall stiffeners, elevation belts and floor tie bars to reduce overall distortions;

- frames above openings (doors and windows) to resist to local distortions;

- steel arches in main sewers and tunnels.

To reduce the settlement effect, foundation underpinning can also be envisaged under some buildings, prior to tunnelling, to bring down loads under the excavation level of the future tunnel.

6.4. TUNNELLING IMPROVEMENTS USING THE SEQUENTIAL METHOD

Generally, a reduced number of work phases is likely to reduce settlements. Actually, splitting the cross section reduces the overall advance speed, increases the duration of the temporary bearing phases, requires underpinning and causes delay in the cross section closure. In all, that may hamper tunnelling work instead of benefitting from the splitting of the working face. It is thus convenient to reconsider the following old idea: cross section horizontally split = reduced settlement.

Modern excavation and support installation means allow to reduce the number of phases and contribute to improving the overall speed and safety throughout the drive. This horizontal splitting, however, remain useful in particular when manual means are used (small cross section). Actually, it is thus convenient to install a very light support and to secure it as fast as possible.

When instability is feared, the soil mass balance may be improved by acting on the cross section shape. If necessary, the face may also be reinforced at the surface, in the vicinity and/or in the soil mass. In case of excavation in waterbearing grounds, the arrangements made shall be accompanied by the measures necessary to the control of hydraulic gradients.

These measures shall be scheduled right from the design phase or implemented during tunnelling if unexpected instability occurs. It is clear that, in the second case, the worksite advance will be slowed down thereby causing budget overruns. Managing an emergency splitting of the tunnel cross section will be more difficult and may lead to a complete upheaval of the project budget.

6.4.1. Face support

Usually, in case of instability appeared throughout works, the first measure consists in leaving in the middle of the face an unexcavated stabilising counterfort called central core. The face may be inclined at the same time, however this is scarcely done because it induces important geometrical constraints for installing the support rings on the crown.

The additional spray of a shotcrete layer which may be reinforced is however recommended because it allows to confine minor instability likely to develop towards the face core.

In some cases more critical, a face consolidation by means of inclusions shall give the wide strength required to ensure stability. It is desirable that the system be designed to rely permanently on a constant sealing off (combination of variable length inclusions defined according to the excavation pitch) (Fig. 18). Inclusions will preferably be destroyable by the boring machine (fiber glass bolts or subhorizontal jet grouting columns for example).

6.4.2. Pre-support

When the project studies or the observations made during excavation expect serious instability i.e. involving widely the grounds situated above the crown, stiff measures shall be taken.

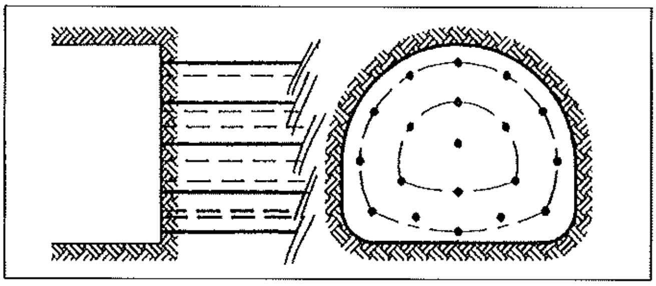


Fig. 18. Basic face support.

If ground improvement from the surface is not a good solution (technically or economically), it is required to install a pre-support on one part of the stretch that shall be installed from the face to be effective ahead of it. Several methods are used for this purpose according to ground quality, excavation geometry (cross section height) and the means available on the worksite.

Forepoling

This technique aims at limiting the decompressions at the crown immediately ahead of the stretch being excavated. It consists in installing longitudinal bars or steel plates at the periphery of the face, most of the time on the upper third or quarter part of the circumference. These bars or plates, often associated with steel arches, make up a short length hood that leans on the last arch installed immediately against the face (Fig. 19).

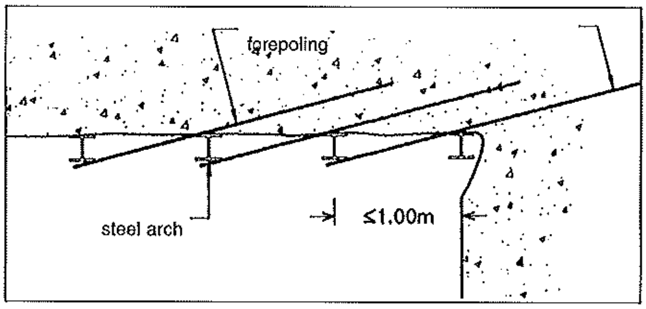


Fig. 19. Basic forepoling

The effectiveness of the hood depends on its length and widening. The hood length is dependent on the penetration of inclusions in the ground and, in general, the successive hood overlap is about twice or three times the distance between the arches. This distance conditions the installation angle of the elements as a low angle (15°) shall only be obtained if special arrangements are made such as core steel arches or lattice girders.

Forepoling also called forepiling is convenient for coarse alluvial deposits, slips or very fractured rocks. In some cases, the bars are replaced by perforated tubes in which mortar is injected after installation to improve arching between the bars.

In the case of grounds for which it is not possible to rely on arching or lattice work, steel plates may be used. However, due to poor inertia, their penetration length hardly exceeds one and a half the distance between the arches.

In the case of a simple shield, forepoling may be improved by installing high inertia, mobile subhorizontally-jacked rods. This technique is hardly used nowadays. The cantilever hood that often moves on the upper part of compressed air shields plays a very similar role.

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, to limit decompressions and to protect personnel from the potential fractures undergone by the entire excavated cross section.

The classical umbrella vault, sometimes associated with a face reinforcement, is made up of either bars (Ø 32 or 40 mm) or grouted tubes (Ø 90 to 250 mm) or jet grouting columns (Ø 30 to 60 cm). In view of the deviations during pipe roofing, the vault length will not exceed 12 to 15 m. In practice, these vaults are conical to be drilled on the face without overexcavation and they overlap (Fig. 20). This overlap depends on the cross section height and the ground nature; it is recommended that it be not less than 3 m.

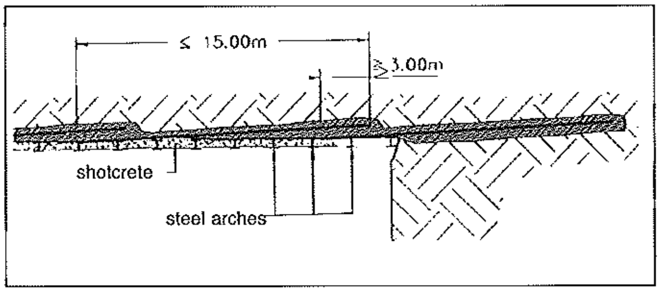


Fig. 20. Basic classical umbrella vault

Ground decompression during a stretch excavation is limited owing to the creation of a longitudinal canopy between the face and the last installed arch of the previously excavated stretch. Obviously, whilst the effectiveness of the device depends on its longitudinal stiffness, the installation quality of the longitudinal elements is absolutely essential.

In the case of very shallow crossing under buildings, the solutions shall be reinforced and cunningly adapted to the project and worksite parameters. As an example, we can quote the following techniques:

- parallel steel tubes with high inertia force (Ø 300 to 600 mm), most of the time filled with concrete and sometimes joining or even connected. These tubes are often horizontally pipejacked along a length not exceeding 30 to 40 m, from a pipejacking frame featuring a very stiff reaction to limit deviations. They are sometimes installed using directional drilling which allows to avert the reaction of the soil mass and to admit longer lengths;

- tangential or secant galleries driven using microtunnellers (Ø ≤ 1.20 m) or by traditional means and filled with concrete.

Forevault

The forevault method derives from the umbrella vault concept [63, 69]. Prior to each earthwork cycle, a 15 to 30 cm thick shell subparallel to the tunnel generators is cast from the face. The support is then made up of successive 'forevaults', the overlap of which depends on the ground conditions (Fig. 21). These prevaults may be installed in half upper cross section or full cross section tunnelling.

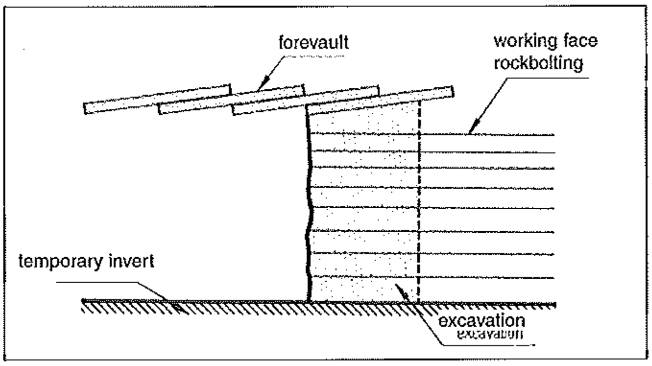


Fig. 21. Basic forevault

The method consists in excavating a slit by means of a longitudinal pre-cutting saw mounted on an arched support structure that moves along the extremities of the tunnel profile. As excavation advances, the slit is filled with shotcrete. The maximum depth of the slit depends on the stiffness of the device (for the time being, it does not exceed 5 m), as well as on the ground quality that conditions stability prior to shotcreting.

The pre-cutting solution cannot be used at a moment's notice. It must have been envisaged right from the project design phase.

6.4.3. Crown support

Whether the ground required improvements or consolidation prior to underground excavation or not, experience shows that, when using sequential method, settlements may mostly come from the poor conditions of installation of the support. Without intending to be exhaustive nor reverting to already developed topics (see § 3.1.), we shall insist hereafter on a few cases requiring monitoring.

Support with steel arches

This support mode is still very frequent in French projects, undoubtedly as a continuation to habits adopted after World War 2 inherited from mining. Partially for the reasons developed above and in view of evolutions under way (shotcrete, lattice girders and rockbolts), this support type should loose its leadership in the near future.

In the case of a steel arched support, the main settlement sources are linked to wedging of arches on the face. Obviously, **an arch installed on the face but improperly wedged does not allow at all to seall off the ground.** In this case, settlement control as well as personnel safety cannot be ensured. Wedging must, moreover, be effective on the whole arch profile as well as on its base.

The high density of wedges around the profile and their compressibility make wood wedging of even better quality. Improper wedging shall cause ground deformations since the ground tends to fill the free space as well as uncontrolled deformations of arches featuring a very low bending strength if they are not uniformly secured to the ground.

If the arch base is improperly wedged, either due to insufficient bearing surface or wedge compressibility, the load transmitted shall lead to punching of the foundation stratum. This results in an overall settlement which is tougher as profile wedging is inefficient and the magnitude of which is dependent on this load.

The performances of a steel arched support are also linked to the nature of filling between the arches. There is an important distinction between simple sheeting, wooden or sheet metal, and shotcrete shells.

Wooden board sheeting does not ensure specific sealing of the excavation. In such a case, arching allows for the successive arches to support the ground and sheeting is only there to bear the weight or thrust of the dead ground situated under the relieving arch. The overall effectiveness is then directly linked to arch wedging (see above) but local unsealing prove very harmful to control settlements.

Metal sheeting equally proves ineffective. To cast concrete between the sheeting support and the ground is a practice that allows to confine roughly the ground but experience proves its lack of effectiveness to fill completely the void between the sheets and the ground, especially in the roof as installation difficulties occur. Therefore, there is no significant improvement as compared with the previous case.

To solve these faults, shotcrete is recommended as a shotcrete shell improves steel arch wedging and provides specific sealing due to its stiffness and contact conditions with the ground. To take the best advantage from this effectiveness, it is recommended to install, immediately after excavation, a first shotcrete layer on which the steel arches will be wedged.

Another trend tends to replace steel arches and sheeting by lattice girders associated with shotcrete.

Rockbolting support

In the case of rockbolting support, the limitation of displacements around the excavation profile, and therefore the limitation of rock mass deformations, is narrowly linked both to the choice of the bolt length (depending on the plastic radius) and to their proper anchoring, thereby ensuring a proper sealing of the soil mass.

For a better limitation of settlements, it is strongly recommended to use rockbolting associated with the immediate installation of a shotcrete shell.

Shotcrete

The current trend tends to resort more frequently to fibre reinforced shotcrete. This is favourable to reduce settlements as time is saved to install a latticed, welded structure.

"Active vault" (vault with major span width)

To reduce ground decompression, it may be interesting to install the ultimate vault of the construction the nearest possible to the face as:

- a longitudinal relieving arch may develop easily between the face and the vault;

- in view of its stiffness, the vault contributes to limit unsealing.

However, casting the vault the nearest possible to the face is very difficult and the worksite constraints require a long distance between the face and the nearest unmoulded resisting ring. A solution may be the so-called 'active vault method (also called Jacobson method) featuring a vault with major span width made up of a set of adjacent arches, each composed of precast reinforced concrete segments (Fig. 22.). These arches are assembled at a distance from the face ranging from one to two times their width (2 x 0.8 to 1.2 m). Their installation is completed by prestressing with flat jacks, most of the time situated at the crown.

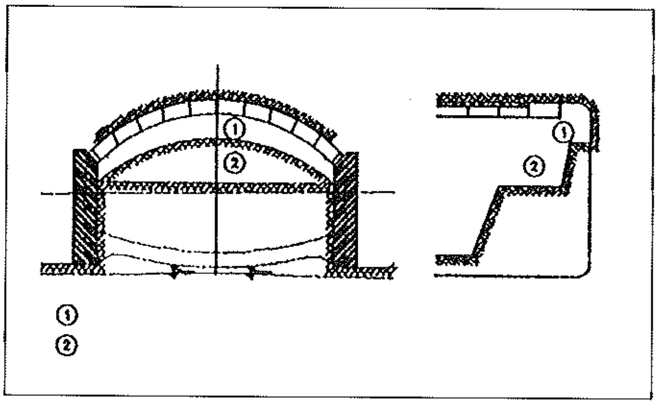


Fig. 22. Basic "active vault" (vault with major span width)

In view of the fact that the normal effort is recentered through jacking, the 'active arch' features a wider span, thus contributing to a lower face, thicker overburden above the construction and reduced earthworks. This technique has been used with success in Paris (RER lines A and B, METEOR, RER line D, EOLE [65]).

We shall keep in mind that installing the segment erector requires a classical assembly chamber, thus less performing in relation to settlements.

6.4.4. Underpinning of the upper half cross section

In the case of soft ground excavation, in an horizontally divided cross section, settlements induced by the lower mid cross section excavation may be reduced by underpinning the loads induced by supporting the upper parts of the excavation so as to transfer them under the level of the future invert.

According to ground nature and the structure to be underpinned, underpinning may use micropiles, jet grouting columns

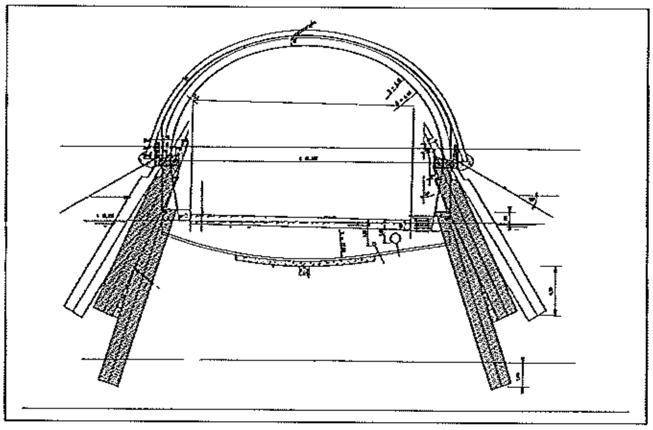


Fig. 23. Basic underpinning of the upper half cross section

[70], or shafts to a lesser degree (Fig. 23.).

Whatever the chosen solution, it shall be convenient to take any measures so that the implementation of preventive arrangements does not itself generate settlements; thus, in addition to the necessary installation:

- micropiles shall sometimes be installed using jacking and their elastic shortening under loads shall always be taken into account;

- jet grouting columns shall be grouted so that are not superimposed columns that still have not reached full maturity; on the contrary, overall settlements are to be feared under the load to be underpinned.

6.4.5. Arch invert support

When grounds are of very poor quality in relation to stresses induced by digging operations, it may prove very effective to close the cross section after each major excavation cycle. This may be obtained by a temporary arched invert that will be destroyed during the next excavation cycles. This invert ensures three main functions:

- blockage of convergences at the footings;

- ground sealing at the invert;

- improvement of the bearing capacity at the footing base.

This invert may be shotcreted with a latticed, welded structure. Traffic of machines and trucks is then possible. When the vault support comprises steel arches, steel section countervaults linked up to the arches may be installed. This solution often proves less effective than shotcreting because the invert is then poorly confined, or even not confined at all, unless concrete is added which thus makes steel countervaults almost useless.

6.5. SHIELD IMPROVEMENT

The choice of a shield excavation mode depends on many technical and economical factors. We will here consider the case in which a closed-face shield with segment installation under the tailskin is required in view of the low mechanical quality of the grounds to be crossed. In this case, it shall be attempted to act on the identified settlement sources and therefore to fight against the ground decompression:

- at the face and ahead of the face;

- above the shield;

- at the exit of the tail.

It is important to remind that the success will both come from the technical choices and experimented personnel at the worksite with all skills involved in the complex operation of a shield. Of course, the cost is high but at time of selecting the company, it shall be convenient not to forget ever that this is the price to pay to avoid budget overruns in case of serious incidents during tunnelling.

6.5.1. Reduction of decompression ahead of the face

Beyond the type of sealing most adapted to such or such situation (compressed air shield, slurry shield or EPB shield), which is not the topic of this document, it is convenient to remind that controlling the pressure in the cutting chamber is essential. This objective is not easy to reach and requires, in particular, the following:

- being aware of the best data on the ground ahead of the face (if preliminary investigation did not provide complete knowledge of the subsoil, in particular in the case of grounds with voids, the shield will have to be equipped with an additional geophysical investigation and trial boring system);

- equipping the machine with reliable gauges to measure all the variations of the significant parameters in the cutting chamber and on the muck conveying system and slaving it to the indications given by these gauges.

6.5.2. Reduction of decompression along the shield

This annular space may be reduced:

- by limiting the overcut at the very best or by adopting slaved overcut tools (elliptical overcut);

- by reducing the total shield length or by installing one (or two) articulation(s) which may create other guidance constraints ;

- by scheduling, during shield assembly, the possibility of backfilling the void with bentonite through the tailskin (when necessary, bentonite will also allow to reduce friction).

However, there is not much room to manoeuvre since the machine design depends on the project constraints, on technology limits and must ensure compatibility between the different shield functions.

6.5.3. Decrease of immediate post shield loss

Avoiding this void is the key issue to control settlements [66] and requires two measures:

- longitudinal pressure grouting through the tailskin while:
  • subjecting shield advance to the actual grouting undertaken,
  • using several simultaneous grouting lances on the profile ;

- reducing the tailskin and tail seal thickness as long as it is compatible with the other machine functions.

This system offers the obvious and essential advantage (compared with grouting through segments) to allow filling of the void as it appears i.e. as the shield advances. To get full effectiveness, it is required that:

- the grouting parameters be permanently maintained at the desired level, whatever the shield drive speed;

- the grouting product setting risk in the tubes and seals be controlled; this may be obtained, for instance, by using grouting products without cement but offering a cementation capability (puzzolanic reaction for example).

7. INSPECTION

The detailled definition of the required inspection to follow settlements is not the topic of this document. Only the major principles that must regulate the definition of such an instrumentation are discussed hereafter.

7.1. INSPECTION PURPOSE

Inspection must allow to follow deformations and displacements in the ground and in the neighbouring zones, including carriageways and strips, before, during and after excavation works.

Prior to tunnelling work, it is essential to get a **zero condition of the zone neighbouring movements** in the tunnel to be driven. This condition supplements the preliminary studies on the building and its earlier movements. This information is required to confirm any measurements taken at the worksite. In addition, in the case of a poor overlying building quality and/or foundation strata of poor bearing capacity, instrumentation gives the client a knowledge of the possible evolutions of the building dead load without any influence of the future work.

Once tunnelling is completed, the measurements taken allow to check the likelihood of deferred movements or the return to the previous situation.

During tunnelling work, instrumentation allow to determine the movements induced by tunnelling in relation to the limits included in the contract (§ 8.1).

7.2. INSTRUMENTATION CHOICE

The instrumentation device shall be designed in detail during the study phase as it must meet accurate objectives resulting from design studies and it must not prove impossible to implement [72].

The engineer shall not restrict its views to the most cost-saving system in terms of supply of materials but shall broaden its analysis to labour costs for tunnelling. Taking very frequently simple measurements (topography for example) may add much more to the cost of the project than the initial installation of an automatic supply system.

The corresponding quantity in tender documents must be explained so that bidding companies can evaluate the risk.

In all cases, the engineer shall foresee a significant provision to the investigation budget to deal with, when necessary, the specific monitoring not scheduled into the project that inevitably occur throughout inner-city excavation works.

7.2.1. Monitoring of the existing structures

The measuring devices shall allow to determine at least three types of movements in the neighbouring zones:

- absolute settlements;

- differential settlements;

- rotations.

Absolute surface settlements and on the overlying building are measured using classical topography, with a required millimeter accuracy. The measurements are easy to make on points outside buildings or facilities but prove much more difficult when the measuring points are situated inside, especially on cellar gable walls or buried utility tunnels.

Differential settlements between two points are determined by the difference between absolute measurements above the considered points. For the reasons above mentioned, determining differential settlements between the structure supports makes monitoring complex, difficult or even impossible above all supports of each involved structure.

The effectiveness of the installed measuring system requires the possibility to often repeat the measurements throughout the tricky worksite phases. The representative measuring points require then careful selection. Should direct measurements on supports be impossible, it is at least required to design an adapted device to monitor the settlement trough in the influence zone of tunnelling work.

Direct monitoring of tilt, or rotation, of some specific structures or parts of structure (windows, lintels, etc.) shall be ensured by installing on these structures direct measuring devices such as vertical inclinometers along the bearing elements or horizontal bubble levelling instruments on the bearing elements. The use of electronic clinometric gauges widely spreads in Great Britain for taking local rotation measurements.

7.2.2. Ground measurements

Ground deformations between excavation and the surface will 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 will be deeply anchored under the invert level (about one diameter) whilst the devices situated upright of it shall require three-dimensional monitoring. Levelling measured at the readout heads of deep extensometers and settlement measuring devices shall be monitored at least as often as the current topographic points.

Such devices are expensive whether it be for supplying, installing or monitoring and prove difficult to adjust. Their location shall then be carefully determined. But it is essential not to make irrelevant savings on this instrumentation which remains cheap as compared with the construction cost and even cheaper as compared with cost if work is halted.

In addition to the specific measurement stretches related to existing sensitive structures, known at the start of the project or appeared during tunnelling, it is absolutely necessary to equip current stretches with such devices, especially in linear tunnels.

At least one measurement stretch shall be equipped each time a significant ground configuration appears. These stretches shall be placed, if possible, upstream of the route, beyond however the running length of the work so as to collect as soon as possible data to process to improve methods on the remaining route.

Each stretch shall include at least three pairs of devices (settlement measuring device + inclinometer), one in the alignment and the other two on either side of the tunnel. Experience shows that this is the minimum and that two lateral pairs of devices give a more reliable interpretation, especially to determine the position of the inflection point of the settlement trough. A highly equipped single stretch is better than two partially equipped ones.

7.2.3. Measurements in the construction

Only a part of volume losses around the tunnel may be measured. Apart from some specific cases, the volume loss occurring ahead of the face only is accessible when it develops to the surface.

These losses may be indirectly evaluated by means of convergence measurements at and rear of the face and borehole extensometer measurements from the gallery.

7.3. METHODS OF MEASUREMENT

The instrumentation plan shall specify the type, organization and frequency of measurements as well as their purpose. These arrangements shall be clearly detailled in the tender documents and it is up to the company to adapt them to its own methods, under the control of the engineer.

For each work phase, it shall be specified whether continuous measurements are required. This aspect is very important because it conditions the choice of the instrumentation system.

8. CONTRACTUAL ASPECTS

We saw above that tunnelling always induce ground movements of variable magnitudes according to the cases. In inner-city areas, the impact of these movements on existing constructions shall be one of the major worries of the several actors, from the start of design studies to completion of tunnelling.

Beyond this preoccupation, the client shall consider these phenomena when drafting the contractual rules and insert them into a coherent strategy to avoid, throughout tunnelling works, tricky and costly situations for all actors which might lead to matters of dispute always delicate to sort out.

Now and according to clients, there are several approaches focused on the two following points:

- either the company is liable to any damage occurring throughout tunnelling; this approach comprises unrealistic settlement criteria or 'alibi' criteria ;

- or contractual rules for responsibility sharing are applied.

8.1. USUAL CONTRACTUAL CLAUSES

It is the custom in France to include in the contracts for inner-city underground works clauses that specify the maximum admissible ground movements by the environment of the construction to be excavated. This aims at determining the responsibility of the parties as regard the possible disorders that could be generated.

The result is that the client will pay for the repair costs following possible damage caused by ground movements less than the contractual values whilst the contractor will be charged with those exceeding these values.

The whole issue is then focused on the determination of this value admitted in the contract. In some cases, it represents the maximum movement value that the buildings can bear without damage, a possible margin of safety being earmarked to this value. In other cases, it only reflects the authority of the client and may be completely unrealistic.

The usual rules require several of the following types of criteria, with or without combination:

- absolute settlement and sometimes absolute heave;
- differential settlement or relative rotation;
- general tilt;
- surface of the settlement trough.

These rules sometimes set the frequency of measurements or put it back to the drafting of the company's Quality Assurance Plan.

Conversely, a warning limit - from which an analysis of the work conditions shall be made to change, if necessary, the methods used - is scarcely specified. Neither is a limit for work suspension.

Finally, we shall underline the complexity of the responsibility problem when several companies follow one another at the worksite. This is the case, for example, when preparatory work is conducted in each gallery before the main worksite is open.

8.2. THE OPINION OF THE DIFFERENT ACTORS

During tunnelling, we can distinguish actors and onlookers. The actors are direct participants to the construction phase whilst onlookers, although sometimes accidental actors, are local residents i.e. tenants, landlords or operators of the existing buildings and the facilities neighbouring the construction.

We should undoubtedly mention underwriters who considerably influence the outcome of the difficulties caused by damage to the existing buildings. To date, as practices still are unclear in this field, they will be discussed by AFTES in the second stage of its workgroup on settlements.

There are three main actors: client, engineer and contractor.

8.2.1. The client

The client has to deal with the difficulty of conciliating two objectives. It consists, first, in minimising spending and secondly social disruption to local residents. Inconvenience may occur as preventive measures prior to tunnelling work are taken and during excavation as tunneling is actually taking place as well as consolidation operations of the existing buildings.

When the project is being designed, it may be awkward for the client to mention in a file the likelihood of settlements of significant magnitude in front of people not acquainted with the requirements of the technique and very sensitive, in inner-city environment, to the reaction of local residents.

The client fulfills contractual obligations taken throughout administrative proceedings prior to the state approval at a moment when dubiousness on the exact condition of the building still prevails. In the absence of an access permit and worried by the responsibility sharing principle, the client may be tempted to delineate a-priori very stringent contractual limits.

This strategy which consists in minimising limit settlements may reveal, in certain cases, adverse effects either because the imposed values prove impossible to respect, which increases the risk of litigation, or because it appears that these values may only be respected at the expense of excessive budget overruns, which prove irrelevant as compared with the importance of the disorders one tries to avoid.

8.2.2. The engineer

The engineer is in charge of assessing, among other things, the movements induced by the excavation methods he selected as well as the behaviour analysis of the building subject to these movements.

He is the only person to have the necessary time to undertake these difficult tasks. However, despite all the possible approaches, predicting ground settlements remains awkward and uncertain. As is often the case in tunnelling, **only a rough estimate may be given.**

There are two approaches infered by the philosophy of the client. They consist:

- either in adopting a-priori criteria that satisfy the client and then in adapting the tunnelling methods to meet these criteria while withstanding at the same time the pressure induced by the budget of the project ;

- or in designing a realistic tunnelling method, infering the movements that will be generated and making sure that the existing buildings will tolerate them or, should this not be the case, defining the preliminary work to preserve them.

In his settlement analysis, the engineer shall not overlook elements of very different origins such as:

- the possible variations in the implementation of construction methods;

- anticipating alternatives that the companies may submit;

- the deferred effects that may continue, or appear, after work is completed;

- the consequences of work organisation and especially the successive intervention phases involving sometimes different companies and contracts.

8.2.3. The contractor

The contractor's approach is founded on his experience of similar works and on the control of the means to involve.

When drafting his bid, he is not in a position to take into consideration other precautions than those predicted by the engineer. Actually, he does not have the time to conduct all the required studies before the deadline to submit his bid. In addition, the introduction of additional precautions would lead to make the bid less advantageous, which would let him few chances to be shortlisted as successful tenderer, at least as long as the best bid concept is not clearly defined in the tendering procedure, or has not become common practice.

Now, the contractor cannot ignore the settlement issue and forget that his expertise strongly influences the likelihood of disorders. He shall not take the risk of underestimating the cost of the precautions to take by expecting these precautions to be reduced throughout tunnelling. This is particularly true when he submits a tunnelling alternative.

We must remind that in case of litigation, it is always very difficult to assess the exact liabibility of the participants and that one of the actions of the litigants consists in determining, according to the interests they defend, if the observed settlements are normal or caused by improper excavation. In the end, experience shows that this unclear situation is not an advantage for the actors involved.

9. ENVISAGEABLE IMPROVEMENTS

It is the interest of every actor involved in a shallow underground project in the vicinity of existing buildings to use clear and easily applicable rules which do not aim at reflecting accurately a reality difficult to understand but rather to clearly specify everybody's responsibilities.

The result of this paper is to make some proposals to tend to greater clearness in the construction phase.

The ***client*** is the only person, 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 environment. He must:

- conduct 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 works, infer from it their capability to undergo movements thereby limiting the future litigation;

- commission the most complete studies to participants competent in geotechnique and structures;

- set settlement limits in line with the situation analysis. The engineer shall define, first, the criteria applicable to each main work phase and, secondly, to the construction process;

- provide, during tendering, all data he is aware of, including the results of his preliminary studies. This information could be gathered in a **Synthesis report on the project environment,** based on the geological, geotechnical and hydrogeological synthesis report described in fascicule 69 of the state general technical specifications ;

- supply, when the studies show that the existing buildings cannot but suffer damage, any required improvement prior to tunnelling work.

The contractor must provide assistance throughout works. Beyond the implementation of required means to bring his mission to a successful end, for studies as well as for works, he must awake all personnel to place them in a position to immediately estimate the incidence on settlements of any modification in the works initially defined so as to limit them at the very best.

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

- greater technical clearness when defending their interests, by reverting more frequently to experts specialised in underground works and in soil-structure interaction;

- technical analysis of the risks before clinching their binding agreements;

- faster analysis of litigious situations since there is too often too much delay in undertaking the damage analysis, thereby making impossible to clarify the objective causes or responsibilities nor to limit additional works. These works frequently prove exaggerated for the sake of safety.

It is noticeable that the above proposals are in compliance with the spirit of the recommendations referring to the contractual risk sharing, designed by ITA, the International Tunnelling Association.

Lastly, it is necessary that an overall assessment of movements and possible damage observed be carried out at the end of each worksite in relation to the project conditions and the methods used. This overall assessment must be available to the tunnelling community to give everybody better knowledge of the problems, thereby contributing to a better design of future projects. To do so, the publication at the end of each worksite, for example in Tunnels & Ouvrages Souterrains, the bimonthly review of AFTES, of a summary or an article jointly written at least by the engineer and the contractor, may become an habit. This article may be written by trainee engineers or students (in the framework of their Travaux de Fin d'Etudes or DEA degree since it is compulsory to be graduate).

Feedback seems essential as there is a long time to go when engineers will properly control all parameters in the calculation of a settlement prediction. Moreover, beyond the purpose of this recommendation, it could be interesting for AFTES to think about setting up a national **coordination to do the summaries necessary to the state-of-the-art evolution and make them available to actors involved in undeground works.** This would contribute to avoiding calculation and theory-based predictions thereby making possible to elaborate a French doctrine that would be of utmost interest.

The authors of this recommendation hope that its first revision shall benefit from the proposed improvements.

BIBLIOGRAPHY

CHAPTERS 2, 3 AND 4

[1] AFTES-Groupe de travail N°. 7, (1994), Recommandations pour le choix des paramètres et essais géotechniques utiles à la conception, au dimensionnement et à l'exécution des ouvrages creusés en souterrain, AFTES-TOS, N° 123, pp. 129-145.

[2] Attewell P.B., Yeates J., Selby A.R., (1986), Soil movements induced by tunnelling and their effects on pipelines and structures, Blackies § Sons Ltd, London.

[3] Broms B.B. and Bennermark H., (1967), Stability of clay at vertical openings, ASCE, Journal of Soil Mechanics and Foundation Engineering Division, SMI, Vol. 93, pp. 71-94.

[4] Broms B.B. and Shirlaw J.N., (1989), Settlement caused by earth pressure shields in Singapore, Colloque international Tunnels et Microtunnels en terrain meuble : du chantier à la théorie, Paris, pp. 209-219.

[5] Chambon P. et Corté J.F., (1989), Stabilité du front de taille d'un tunnel faiblement enterré : modélisation en centrifugeuse, Colloque international Tunnels et Microtunnels en terrain meuble : du chantier à la théorie, Paris, pp. 307-315.

[6] Chambon P. et Corté J.F., (1990), Stabilité du front de taille d'un tunnel dans un milieu frottant, approche cinématique en calcul à la rupture, Revue Française de Géotechnique, N° 51, pp. 51-59.

[7] Chapeau C., (1986), Résultats de mesures au passage du tunnelier du métro de Lyon, AFTES-TOS, N° 78, pp. 257-267.

[8] Clough G.W. and Leca E., (1993), EPB Shield Tunneling in Mixed Face Conditions, ASCE, Journal of Geotechnical Engeering, Vol. 119, N° 10, pp. 11640-1656.

[9] Clough G.W., Schmidt B., (1981), Design and Performance of Excavations and tunnels in Soft Clay, Soft Clay Engineering, Brand and Brenner Eds, Elsevier, Chapter 8, pp. 569-631.

[10] Cording E.J. et Hansmire W.H.,(1975), Displacements around soft ground tunnels - General Report, 5th Pan American Conference on Soil Mechanics and Foundation Engineering, Buenos Aires, Session IV, pp.571-632 ; traduit en français par J. Kérisel, Les déplacements autour des tunnels en terrain tendre, (1977), AFTES-TOS, N° 8 et 12.

[11] Davis E.H., Gunn M.J., Mair R.J., Seneviratne H.N., (1980), The stability of Shallow Tunnels and Underground Openings in Cohesive Material, Géotechnique, Vol. 30, N°. 4, pp 397-419.

[12] Dormieux L, de Buhan P et Leca E., (199), Estimation par une méthode variationnelle en élasticité des déformations lors du creusement d'un tunnel : application au calcul du tassement de surface, Revue Française de Géotechnique, N° 59, pp. 15-32.

[13] Dormieux L. et Leca E., (1993), Stabilité du front de taille d'un tunnel dans un milieu sans résistance à la traction, Symposium International Sols indurés et Roches tendres, Athènes, Vol. 2, pp. 1409-1415.

[14] Fujita K., (1982), Prediction of surface settlements caused by shield tunnelling, 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City, Vol. 1, pp. 239-246.

[15] Fujita K., (1989), Special lecture B : Underground construction, tunnel, underground transportation, 12th International Conference on Soil Mechanics and Foundation Engineering, Rio, pp. 2159-2176.

[16] Glossop N.H., (1980), Ground deformation caused by soft ground tunnelling, PhD Thesis, University of Durham.

[17] Hanya T., (1977), Ground movements due to construction of shield-driven tunnel, 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, pp. 759-770.

[18] Kimura T; and Mair R.J., (1981), Cenrifugal testing of model tunnel in soft clay, 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Vol. 1, pp. 319-322.

[19] Konda T., (1987), Tunneling and Excavation in Soils, 8th Asian Regional Conference on Soil Mechanics and Foundation Engineering, Kyoto, pp. 151-170.

[20] Leblais Y. and Bochon A., (1991), Villejust Tunnel : slurry shield effects on soils and lining behaviour and comments on monitoring requirement, Tunnelling '91, London, IMM, pp. 65-77.

[21] Leca E., (1989), Analysis of NATM and Shield Tunneling in Soft Ground, PhD Thesis, Virginia polytechnic Institute and State University, Blacksburg.

[22] Leca E., Atwa M., Rat M., Humbert P., (1993), Analyse des écoulements hydrauliques autour des tunnels, Journées internationales de l'AFTES, Toulon, pp. 55-64.

[23] Leca E. and Dormieux L., (1990), Upper and lower bound solutions for the face stability of shallow circular tunnels in frictionnal material, Géotechnique, Vol. 40, N° 4, pp. 581-606.

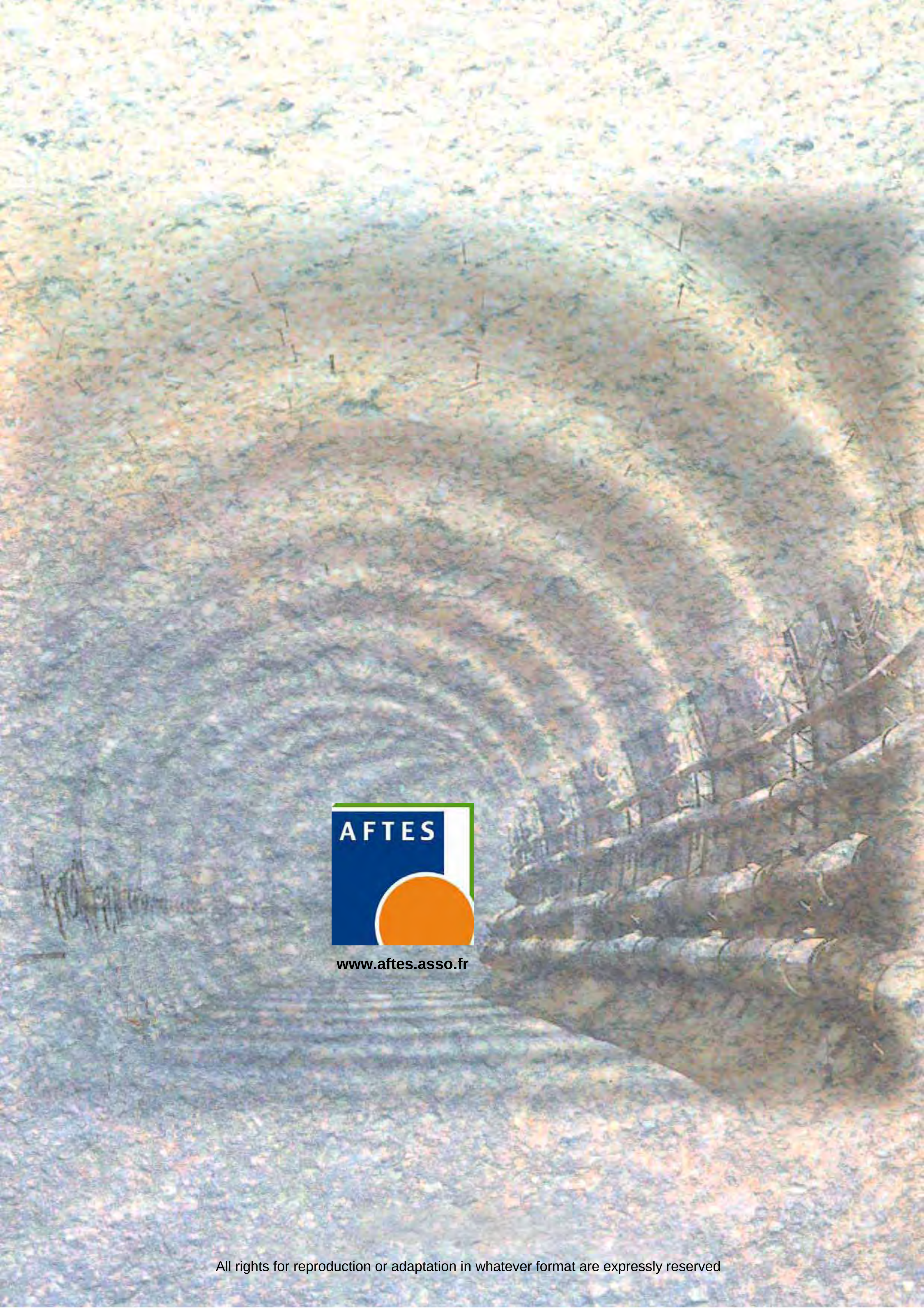
[24] Leca E. and Dormieux L., (1992), Contribution à l'étude de la stabilité du front de taille d'un tunnel en milieu cohérent, Revue Française de Géotechnique, N° 61, pp. 5-16.

[25] Leca E. et Panet M., (1988), Application du calcul à la rupture à la stabilité du front de taille d'un tunnel, Revue Française de Géotechnique, N°. 43, pp. 5-19.

[26] Lee K.M.and Rowe R.K., (1989), Deformations caused by surface loading and tunnelling : the role of elastic anisotropy, Geotechnique Vol. 39, N° 1, pp. 125-140.

[27] Lee K.M., Rowe R.K. and Lo K.Y., (1992), Subsidence owing to tunnelling. I. Estimating the gap parameter, Canadian Geotechnical Journal, Vol. 29, pp.929-940.

Notes :



AFTES

www.aftes.asso.fr