Guidelines for the Design of Tunnels

ITA Working Group on General Approaches to the Design of Tunnels

Abstract—This second report by the ITA Working Group on General Approaches to the Design of Tunnels presents international design procedures for tunnels. In most tunnelling projects, the ground actively participates in providing stability to the opening. Therefore, the general approach to the design of tunnels includes site investigations, ground probing and in-situ monitoring, as well as the analysis of stresses and deformations. For the latter, the different structural design models applied at present—including the observational method—are presented. Guidelines for the structural detailing of the tunnel lining and national recommendations on tunnel design are also given. It is hoped that the information herein, based on experiences from a wide range of tunnelling projects, will be disseminated to tunnel designers throughout the world.

1. Scope of the Guidelines

The International Tunnelling Association (ITA) Working Group on General Approaches to the Design of Tunnels was established in 1978. As its first project, the group developed a questionnaire aimed at compiling information about structural design models used in different countries for tunnels constructed prior to 1980. A synopsis of the answers to the questionnaire was published by the International Tunnelling Association in 1982 (ITA 1982).

As a continuation of that first report, the working group herein presents guidelines that attempt to condense the various answers from the first report and include additional experiences in the general approaches to the design of tunnel structures. These guidelines fulfill one of the main objectives of the International Tunnelling Association, namely, to disperse information on underground use and underground structures throughout the world by crossing national borders and language barriers.

Those interested in the subject of tunnel design should also consult published reports of other ITA working groups, e.g. the recent ITA report on contractual sharing of risk (see T&UST 3:2) and ITA recommendations on maintenance of tunnels (see T&UST 2:3). Furthermore, a number of national and international organizations, such as the International Society on Rock Mechanics, have published recommendations on related subjects, such as field measurements and laboratory testing for rock and ground. Some of these publications and reports are listed in the Appendix.

In tunnelling, most often the ground actively participates in providing stability to the opening. Therefore, the design procedure for tunnels, as compared to aboveground structures, is much more dependent on such factors as the site situation, the ground characteristics, and the excavation and support methods used. Recommendations on tunnel design naturally are limited with regard to their consistency and applicability because each tunnelling project is affected by special features that must be considered in the design. Nevertheless, it is hoped that the general outline provided in these guidelines, based on the experience gained from many tunnelling projects, may be of some help for those starting a project.

2. Outline of General Approaches

2.1. General Procedure in Designing a Tunnel

Planning a tunnelling project requires the interdependent participation of the following disciplines, at a minimum:

• Geology.
• Geotechnical engineering.
• Excavation technology, e.g. machine tunnelling.
• Design of the supporting structural elements, including long-term behavior of materials.
• Contract principles and law.

Although the experts in each of these disciplines may be responsible only for their specific area of knowledge, the decision on the main design features should be the outcome of the cooperative integration of all the disciplines. Only thus can it be ensured that the project, in all its details, has been developed in unity, and not as the successive addition of the separate work of each of the experts.

The basics documents for tunnel design should include or cover:

• The geological report presenting the results of the geological and geophysical survey.
• The hydrogeological report.
• The geotechnical report on site investigations, including the interpretation of the results of site and laboratory tests with respect to the tunnelling process, soil and rock classification, etc.
• Information on line, cross-section, drainage, and structural elements affecting later use of the tunnel.

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2.2. Elements of the Structural Design Model for Tunnels

In planning, designing, analysing and detailing a structure, engineers promise that the structure will neither suffer structurally nor collapse during its projected lifetime. Thus, models of the reality are necessary for analysis in order to predict the behaviour of a tunnel during the excavation and during its lifetime. Models are also needed for bidding on projects.

The following main elements involved in the design procedure are shown as a flow-chart in Fig. 1:

1. **Geology and site investigations** must confirm the line, orientation, depth, etc., of the opening, e.g. a cavern.
2. **Ground probing and soil or rock mechanics must be applied** to determine the ground characteristics, e.g. primary stresses, soil or rock strength, faults, water conditions.
3. **Experience and preliminary estimates or calculations** are used to determine the cross-section required and the choice of the excavation method or the tunnel driving machine to be used, as well as the methods of dewatering the ground and the selection of the supporting structural elements.
4. After steps (1)-(3) are completed, the tunnelling engineer must derive, or even invent, a structural model. By applying equilibrium and compatibility conditions to the model, the engineer has to arrive at those criteria that are factors in deciding whether or not the design is safe. Different models may be used for each excavation phase, for the preliminary and the final tunnel lining, or for different ground behaviour, e.g. in discontinuous rock or homogeneous soft soil. Modelling of the geometric features may vary greatly, depending on the desired intensity of the analysis.
5. A safety concept drawn from failure hypotheses may be based on criteria such as strains, stresses, deformation, or failure modes.

The bypass in Fig. 1 indicates that for many underground structures, as in mining or in self-supporting hard rock, no design models at all are applied. In such cases, past experiences alone may be sufficient.

Risk assessment by the contractor as well as by the owner is needed at the time of contract negotiations. Risks involve possible structural failures of the tunnel support and lining, functional failures after completion of work, and financial risks. The contractual aspects also include risk sharing and risk responsibilities.

In-situ monitoring can be applied only after the tunnelling has begun. If the displacements stop increasing over time, it generally may be assumed that the structure is designed safely. Yet monitoring provides only part of the answer to the question of safety, for it does not tell how close the structure may be to sudden collapse or nonlinear failure modes. The results of field measurements and experiences during excavation may compel the engineer to change the design model by adjusting it to real behaviour.

An iterative, step-by-step approach is characteristic of the design of structures in the ground that employ the participating strength of the ground (see loops in Fig. 1). The designer may begin by applying estimated and simple behavioural models. Adjustments based on actual experiences during the tunnelling excavation (such as excavating the initial section in the same ground conditions or driving a pilot tunnel) will bring the model closer to reality and refine it (if refinement is consistent with the overall accuracy attainable). The interpretations of in-situ measurements (and some back analyses) also may assist designers in making these adjustments.

All of the elements of the structural design model in Fig. 1 should be considered an interacting unity. Scattering of parameters or inaccuracy in one part of the model will affect the accuracy of the model as a whole. Therefore, the same degree of simplicity or refinement should be provided consistently through all the elements of the design model. For example, it is inconsistent to apply very refined mathematical tools simultaneously with rough guesses of important ground characteristics.

2.3. Different Approaches Based on Ground Conditions and Tunnelling Methods

The response of the ground to excavation of an opening can vary widely. Based on the type of ground in which tunnelling takes place, four principal types of tunnelling may be defined:

1. For cut-and-cover tunnelling, in most cases the ground acts only passively as a dead load on a tunnel structure erected like any aboveground engineering structure.
2. In soft ground, immediate support must be provided by a stiff lining (as, for example, in the case of shield-driven tunnels with tubbings for ring support and pressurized slurry
for face support). In such a case, the ground usually participates actively by providing resistance to outward deformation of the lining.

(3) In medium-hard rock or in more cohesive soil, the ground may be strong enough to allow a certain open section at the tunnel face. Here, a certain amount of stress release may permanently be valid before the supporting elements and the lining begin acting effectively. In this situation only a fraction of the primary ground pressure is acting on the lining.

(4) When tunnelling in hard rock, the ground alone may preserve the stability of the opening so that only a thin lining, if any, will be necessary for surface protection. The design model must take into account the rock around the tunnel in order to predict and verify safety considerations and deformations.

Especially in ground conditions that change along the tunnel axis, the ground may be strengthened by injections, anchoring, draining, freezing, etc. Under these circumstances, case (2) may be improved, at least temporarily, to case (3).

The characteristic stress release at the tunnel face (Erdmann 1983) is shown in Figs 2 and 3. The relative crown displacement $w$ is plotted along the tunnel axis, where $w/w_o = 1.0$ represents the case of an unsupported tunnel. In medium-stiff ground nearly 80% of the deformations have already taken place before the lining (shown here as shotcrete) is stiff enough to participate.

For a simplified plane model with no stress release, where the full primary stresses are assumed to act on a lined opening, the displacement may be only 0.4 of that occurring in the unsupported case. The corresponding stress release is shown in Fig. 3. The simplified example, considering only the constant part of radial pressure, yields the values shown for a ring stiffness of $E_K = 15,000 \times 0.5 = 4500$ MN/m and a ground deformation modulus of $E_g = 1000$ MN/m$^2$.

Even in the unrealistic case when the full primary stress acts simultaneously on the ground opening and the lining, only 55% of the stress is taken by the lining; in the case of $E_K = 2250$ MN/m, only 38% is taken by the lining. If an open section of 0.25 of the tunnel diameter is left without lining support, the lining takes only 25% of the primary stresses; for $L_D = 0.5D$, it takes only 12% of the primary stresses.

For very soft ground requiring immediate support (as in the case of very shallow tunnels), almost 100% of the primary stresses are acting on the lining. The values change, of course, with other stiffness relationships and other stress distributions than those shown in Fig. 3, with other cross-sections, and other tunnelling methods.

2.4. Site Investigations, Structural Analysis and In-Situ Monitoring

An adequate intensity of site exploration, from which geological and hydrological mappings and ground profiles are derived, is most important for choosing the appropriate tunnel design and excavation method. A well-documented geological report should provide as much information as is obtainable about the physical features along the tunnel axis and in the adjacent ground. The amount of information should be much greater than the information required for entering directly into a structural analysis.

The results of an analysis depend very much on the assumed model and the values of the significant parameters. The main purposes of the structural analysis are to provide the design engineer with: (1) a better understanding of the ground-structure interaction induced by the tunnelling process; (2) knowledge of what kinds of principal risks are involved and where they are located; and (3) a tool for interpreting the site observations and the in-situ measurements.

The available mathematical methods of analysis are much more refined than are the properties that constitute the structural model. Hence, in most cases it is more appropriate to investigate alternative possible properties of the model, or even different models, than to aim for a more refined model. For most cases, it is preferable that the structural model employed and the parameters chosen for the analyses be lower-limit cases that may prove that even for unfavourable assumptions, the tunnelling process and the final tunnel are sufficiently safe. In general, the structural design model does not try to represent exactly the very actual conditions in the tunnel, although it covers these conditions.

In-situ monitoring is important and should be an integral part of the design procedure, especially in cases where stability of the tunnel depends on the ground properties. Deformations and displacements generally can be measured with much more accuracy than stresses. The geometry of the deformations and their development over time are most significant for the interpretation of the actual events. However, in-situ monitoring evaluates only the very local and actual situation in the tunnel. Therefore, in general the conditions taken into account by the design calculations do not coincide with the conditions that are monitored. Only by relating measurement results and possible failure modes by extrapolating can the engineer arrive at considerations of safety margins.

In many cases, exploratory tunnelling may be rewarding because of the information it yields on the actual response of the ground to the proposed methods for drainage, excavation, etc.

Figure 2. Crown displacement $w$ along the axis, ahead and beyond the tunnel face.

Figure 3. Ground stresses acting on the lining as fractions of the primary stress (Erdmann 1983).
TBM driving, support, etc. In important cases a pilot tunnel may be driven; such a tunnel may even be enlarged to the full final tunnel cross-section in the most representative ground along the tunnel axis. For larger projects, it may be useful to excavate a trial tunnel prior to commencing the actual work. More intensive in-situ monitoring of the exploratory tunnel sections should check the design approach by numerical analysis.

3. Site Investigations and Ground Probings

3.1. Geological Data and Ground Parameters

The appropriate amount of ground investigations on site and in laboratories may vary considerably from project to project. Because the types of ground explorations and probings depend on the special features of the tunnelling project, its purpose, excavation method, etc., they should be chosen by the expert team, especially in consultation with the design engineer. The intensity of the ground explorations will depend on the homogeneity of the ground, the purpose of the tunnelling, the cost of boring, e.g. for shallow or deep cover, and other factors.

The geological investigations should include the following basic geotechnical information (see also ISRM Commission on Classification of Rocks and Rock Masses 1981).

### 3.1.1. Tunnels in rock

There should be divided in geotechnical units for which the design characteristics may be considered uniform. However, relevant characteristics may display considerable variations within a geotechnical unit. The following aspects should be considered for the geological description of each zone:

- Name of the geological formation in accordance with a genetic classification.
- Geologic structure and fracturing of the rock mass with strike and dip orientations.
- Colour, texture and mineral composition.
- Degree of weathering.

**Parameters of the rock mass** e.g. in five classes of intervals, including:

- Thickness of the layers.
- Fracture intercept.
- Rock classification.
- Core recovery.
- Uniaxial compressive strength of the rock, derived from laboratory tests.
- Angle of friction of the fractures (derived from laboratory direct shear tests).
- Strength of the ground in on-site situations.
- Deformation properties (modulus).
- Effect of water on the rock quality.
- Seismic velocity.

**Primary stress field of the ground.** For larger tunnel projects, tests evaluating the natural stresses in the rock mass may be recommended. For usual tunnel projects one should at least estimate the stress ratio \( \sigma_1 / \sigma_3 \), at tunnel level, where \( \sigma_1 \) is the lateral ground pressure and \( \sigma_3 \) the major principal stress (usually in the vertical direction), for which the weight of the overlying rock generally may be taken. Tectonic stresses should be indicated.

**Water conditions.** Two types of information about water conditions are required:

1. **Permeability,** as determined by:
   - Coefficient \( k \) (m/s) (from field tests).
   - Lugeon unit (from tests in boreholes).
2. **Water pressure:**
   - At the tunnel level (hydraulic head).
   - At piezometric levels in boreholes.

**Deformability of the rock mass.** In-situ tests are required to derive the two different deformation moduli, which can be determined either from static methods (dilatometer tests in boreholes, plate tests in adits, or radial jacking tests in chambers) or from dynamic methods (wave velocity by seismic-refraction or by geophysical logging in boreholes). Engineering judgment should be exercised in choosing the value of the modulus most appropriate for the design—for instance, by the relevant tangent of the pressure-deformation curve at the primary stress level in the static method.

Properties for which information is needed when tunnel boring machines are to be employed include:

- Abrasiveness and hardness.
- Mineral composites, as, e.g. quartzite contents.
- Homogeneity.
Swelling potential of the rock. The presence of sulfates, hydroxides, or clay minerals should be investigated by mineralogical testing. A special oedometer test may be used to determine the swell test-curve of a specimen subjected first to a load-unload-reload cycle in a dry state, and then unloaded with water.

The following ground water conditions should be given:
- Water levels, piezometric levels, variations over time, pore pressure measurements in confined aquifers.
- Water chemistry.
- Water temperatures.
- Expected amount of water inflow.

3.1.2. Tunnels in soil

The geotechnical description should primarily follow the recommendations given above for rock. Additional special features for soil include:

1. Soil identification (laboratory testing):
- Particle size distribution.
- Atterberg limits \( w_l, w_p \).
- Unit weights, \( \gamma, \gamma_d, \gamma_s \).
- Water content \( w \).
- Permeability \( k \).
- Core recovery.

2. Mechanical properties determined by laboratory testing:
- Friction angle \( \phi_u, \phi \).
- Cohesion \( c_u, c \).
- Compressibility \( m_r, c_y \).

3. Mechanical properties determined by field testing:
- Shear strength \( \tau_c \) (Vane-test).
- Penetration \( N \) (Standard Penetration Test).
- Deformability \( E \) (Plate bearing, Dilatometer).

4. Ground water condition (in addition to those listed in 3.1.1.): permeability, as determined by pumping tests.

3.2. Evaluation of Parameters by Ground Probing and Laboratory Tests

The properties of the ground that are relevant for the tunnel design should be evaluated as carefully as possible. In-situ tests, which cover larger ground masses, generally are more significant than are laboratory tests on small specimens, which often are the better preserved parts of the coring. The natural scattering of ground properties requires an appropriate number of parallel tests—at least three tests for each property (see also the corresponding ISRM recommendations).

Results of laboratory tests must be adjusted to site conditions. The size of specimen, the effects of ground water, the inhomogeneity of the ground on site, and the effects of scattering must be considered. The conclusions drawn from tests also should take into consideration whether the specimens were taken from disturbed or undisturbed ground.

In many cases, the first part of the tunnelling may be interpreted as a large-scale test, the experiences from which may be drawn upon not only for the subsequent excavations but also for predicting ground behaviour. In certain cases, long horizontal boreholes may facilitate ground probing ahead of the face, or a pilot tunnel may serve as a test tunnel that at the same time provides drainage. The on-site investigations provide valuable results for checking the correlation of large-scale in-situ tests with laboratory tests.

Special tests that correspond directly to the proposed tunnelling method may be required, e.g. for the sufficient preservation of a membrane at the face of a bentonite shield.

The evaluation of the parameters should indicate the expected scattering. From probabilistic consideration of normally distributed quantities it can be deduced that a mean value or a value corresponding to a moderately conservative fractile of a Gaussian distribution is more appropriate than the worst case value.

A set of all the parameters describing the ground behaviour of one tunnel section with regard to tunnelling should be seen as a comprehensive unit and should be well-balanced in relation to each of the parameters. For example, a small value of ground deformation modulus indicates a tendency to plastic behaviour, to which corresponds a ratio of lateral to vertical primary stress that is closer to 1.0. Hence, for alternative investigations some complete, balanced sets of parameters should be chosen instead of considering each parameter alone, unrelated to the others.

The available methods for ground probing and laboratory tests, their applicability and accuracy are given in the Appendix.

3.3. Interpretation of Test Results and Documentation

The field and laboratory tests should be given in well-documented reports, in the form of actual results. Based on these reports, an interpretation of the tests that is relevant to the actual tunnelling process and the requirements of the design models for the structural analysis is necessary. At the time the tests are planned, the team of experts referred to in Section 2.1 should decide which ground properties and ground characteristics are necessary for the general geotechnical description of the ground and for the projected design model. Thus, a closer relationship may be achieved between ground investigations and tunnelling design, and between the amount and refinement of tests and the tunnelling risks.

The documents should lay open the rational interpretational way in which design values are derived from test results. This method has proven to be especially useful in the tendering process, because it condenses the relevant data for the description of the ground and for the design of the tunnel on a band along the tunnel axis beneath a graphical representation of the tunnel profile (see the examples in Figs 9–13).

Such condensed tables may be prepared first for tendering and the preliminary design, and then improved through experience gained and incoming monitoring results. However, it should be clearly stated, especially in the contract papers, that much relevant information is lost or oversimplified in such tables, and that therefore the geotechnical reports and other complete documents should be considered the primary documents.

4. On Structural Design Models for Tunnelling

4.1. Alternative Design Models

The excavation of a tunnel changes the primary stress field into a three-dimensional pattern at the tunnelling face. Farther from the face, the stress field eventually will return to an essentially two-dimensional system. Therefore, the tunnel design may consider only two-dimensional stress-strain fields as first approximations.

The design of a tunnel should take into account the interaction between ground and lining. In order to do so, the lining must be placed in closest possible bond with the ground. To preserve its natural strength, the ground should be kept as undisturbed as possible. The deformations resulting from the tunnelling process (see Fig. 2) reduce the primary ground pressure and create stresses in the lining corresponding to that fractional part of the primary stresses in the ground which act on the sustaining lining. The stresses depend on the stiffness relationship of the ground to the lining, as well as on the shape of the tunnel cross-section. The latter should be selected such that an anchoring action in the ground and the lining may develop.
Figure 4 presents four different structural models for a plane-strain design analysis. The cross-sections need not be circular. These four models are explained more explicitly below.

In soft ground, immediate support is provided by a relatively stiff lining. For tunnels at shallow depth (as for underground railways in cities), it is agreed that a two-dimensional cross-section may be considered, neglecting the three-dimensional stress release at the face of the tunnel during excavation. In cases (1) and (2) in Fig. 4, the ground pressures acting on the cross-section are assumed to be equal to the primary stresses in the undisturbed ground. Hence, it is assumed that in the final state (some years after the construction of the tunnel), the ground eventually will return to nearly the same condition as before the tunnelling. Changes in ground water levels, traffic vibrations, etc., may provoke this "readjustment."

In case (1), for shallow tunnels and soft ground, the full overburden is taken as load. Hence, no tension bedding is allowed at the crown of the tunnel. The ground reaction is simplified by radial and tangential springs, arriving at a bedded-beam model.

In case (2), for moderately stiff ground, the soil stiffness is employed by assuming a two-dimensional continuum model and a complete bond between lining and ground. As in case (1), stress release due to predeformations of the ground is neglected. Inward displacements result in a reduction of the pressure on the lining.

Case (3) assumes that some stress release is caused by deformations that occur before the lining participates. In medium-hard rock or in highly cohesive soil, the ground may be strong enough to allow a certain unsupported section at the tunnel face (see Fig. 2). Also, for tunnels having a high overburden, a reduction of the acting crown pressure (represented in Fig. 4 by $h < H$) is taken into account.

In case (4), the ground stresses acting on the lining are determined by an empirical approach, which may be based on previous experiences with the same ground and the same tunnelling method, on in-situ observations and monitoring of initial tunnel sections, on interpretation of the observed data, and on continuous improvements of the design model. If a plane model is not justified—as is the case for caverns, for more complicated geometries of underground structures, or for an investigation directly at the tunnelling face—a three-dimensional model may be necessary (see Fig. 5). The three-dimensional model also may be conceived as consisting of discontinuous masses (block theory) or a continuum with discrete discontinuous fissures or faults.

Figure 5a. Three-dimensional continuum model.
Figure 5b. Example of two-dimensional finite-element model.

4.2. Continuum or Discontinuum Model

For structural design models such as those in Figs 5a and b, the ground may be modelled as homogeneous or heterogeneous, isotropic or anisotropic; as a two-dimensional, i.e. allowing some stress release before the lining is acting, or a three-dimensional stiff medium. The lining may be modelled either as a beam element with bending stiffness or as a continuum. Plasticity, viscosity, fracture of the rock, non-linear stress-strain and deformation...
The design criteria are computed by numerical solutions. From their origins, the finite-element method and the boundary-element method are basically continuum methods. Thus, homogeneous media and stress-strain fields are evaluated best. In general, discontinua such as rock with fissures and faults, and failure modes, which are initiated by local rupture, shear failure, or full collapse, cannot be covered by continuum methods.

A continuum or discontinuum model is appropriate for tunnel structures where the ground provides the principal stability of the opening (as in hard rock) or where the geometrical properties of the underground opening can be modelled only by numerical analysis, e.g. in the case of closely spaced twin tunnels.

4.3. Bedded-Beam Model (Action-Reaction Model)

If the stiffness of the ground is small compared to the stiffness of the lining, a design model such as that shown in Fig. 6 may be employed. In such a case, the active ground pressures are represented by given loads and the passive reaction of the ground against deformations is simulated by constant bedding moduli. The model may be particularly well-suited to the design of linings of shield-driven tunnels. As to applicability, the stiffness ratio $\beta$ may be smaller than 200:

$$\beta = \frac{E_s R^2}{E} < 200,$$

where: $E_s$ is the representative deformation stiffness modulus of the ground,

$R$ is the radius of the tunnel cross-section or its equivalent for non-circular tunnels,

$E$ is the bending stiffness of the lining.

A more correct solution for the bedding is given by a non-zero stiffness matrix for all elements with regard to radial and tangential displacements.

However, in most cases and in view of the unavoidable approximations based on the other assumptions, a simpler approach may be sufficient. Such an approach considers only radial (and, eventually, tangential) bedding, neglecting the interdependence of radial and tangential displacements and beddings. For non-circular cross-sections, the continuum solution reveals that bedding may be increased at corner sections of the lining, with smaller radius of the curvature.

The bedded-beam model may be adjusted to more complex cases, e.g. by reducing the crown load in accordance with stress release at the tunnel face (see Fig. 3) or, for deep tunnels, by assuming bedding also at the crown.

For articulated effective hinges in linings the bending moments are smaller; the deformations may be larger, depending on the ground stiffness. For hinged linings the limit of $\beta$ given above is not valid.

The analysis of the bedded beam yields ring forces, bending moments, and deformations as design criteria for the lining. If the lining ring is completely closed, the bending moments may be considered less important than the ring forces for providing equilibrium (a smaller safety factor may be justified for the bending moments). Allowances also may be made for a plastic rotation capacity of the lining segments.

For tunnels with very pronounced stress release due to inward deformations, e.g. for deep tunnels in rock, a simple approach to design considerations is given by the convergence-confinement model, which is based only on the interaction of the radial inward displacement and the support reaction to these deformations by resisting ring forces and the corresponding outward pressure (see Fig. 7).

The primary stresses $\sigma_x$ in the ground are released with progressive inward displacements. The acting pressure may even increase when rock joints are opening with larger displacements. In self-supporting rock, the ground characteristic in Fig. 7 meets the $w$-axis; because the primary stresses are released completely, a supporting lining is not necessary. Before the supporting members are installed, it is unavoidable—even desirable—that decompression associated with the predeformation $w_0$ will occur. The stiffness of the lining determines where both curves (characteristic lines) will intersect. At this point, equilibrium as well as compatibility conditions are fulfilled. If the ground characteristic is known, e.g. by in-situ monitoring, the predeformation $w_0$ and the stiffness of the lining (including its development over time and as tunnelling advances), and even its plastic properties are very decisive for the actual stresses in the lining. Both curves in Fig. 7 may vary considerably.

In its usual analytical form, the convergence-confinement model assumes constant ground pressure along a circular tunnel lining. Consequently, it yields only ring forces and no bending moments at all. However, it may be extended to cover ground pressures that vary along the tunnel lining (Gesta 1986).

The model may also be applied as a first approximation for non-circular tunnel cross-sections, although the support reaction curve is distinctly different, e.g. for horseshoe-type cross sections. Therefore, it may be helpful to use the convergence-confinement model in combination with a continuum model and in-situ measurements.

Although the convergence-confinement approach is primarily a tool for the interpretation of field measurements, it also may be applied in support of the empirical approach.

4.4. Empirical Approach

The structural elements and the excavation procedure, especially for the preliminary support of the tunnel, may be selected mainly based on experience and empirical considerations that rely more on direct observations than on numerical calculations. This procedure may be especially reasonable if experiences from a successful tunnelling project can be applied to a similar, new one yet to be designed. Such a transfer of information is justified only when:

- The ground conditions, including those of the ground water, are comparable.
- The dimensions of the tunnel and its cross-sectional shape are similar.
- The depths of overburden are approximately the same.
4.6.1. Ground improvement techniques

One disadvantage of prolonged application of the empirical approach is that, lacking an incentive to apply a more appropriate tunnelling design via a consistent safety assessment, the structure may be designed overconservatively, resulting in higher construction costs. The simple empirical approach contributes little to the advancement of the state of the art in tunnelling.

The empirical approach to tunnel design may also be applied to larger projects in only slightly changing ground if provision is made (especially in the tender) for initial experiences to be extrapolated to the subsequent sections along the tunnel axis. Such a situation justifies a measurement programme that is more intensive for the first sections, in order to gain experience.

4.6. Special Design Features

Special considerations may be necessary if unusual ground behaviour is expected or is caused by ground improvements. Some special design features and considerations are discussed below.

4.6.1. Ground improvement techniques

- The tunnelling methods to be employed are the same.
- In-situ monitoring yields results comparable to those for the preceding tunnelling project.

Grouting and injections. Intensive grouting or injections of the ground may improve the ground characteristics considered in the design model. Although in most cases grouting is applied only for closing discontinuities in rock or for strengthening soft ground, in both cases the goal is to achieve better homogeneity.

Drainage and compressed air. Usually the ground is stabilized by dewatering it and by avoiding inflows of water. Ground failure may be avoided if the pore water pressure is minimized. The assumed ground characteristics may be valid only if successful drainage is possible or if water inflow is prevented, as in tunnelling under compressed air.

Ground freezing, improving the ground by freezing changes the ground properties. The time-dependent stress-strain behaviour of frozen ground can be significant. Freezing draws water toward the lining, causing an increase in water volume and heave at the surface. Concreting on frozen ground delays the strength development of the concrete.

4.8.2. Unusual ground behaviour

Swelling ground. Stress release due to tunnelling and/or ground water influx may cause swelling and a corresponding increase in pressure on the lining. In these cases, a circular cross-section or at least an invert arch is recommended. The swelling resulting from a chemical reaction, as in anhydrid, generally is much more pronounced than that due to the physical absorption of water, as in clay.

Underground erosion, mining subsidence, and sinkholes. Tunnelling in ground that is subject to settlements, as in the case of gypsum erosion or mining subsidence, requires special design considerations. A flexible lining that follows the ground movements by utilizing its plastic deformation capacity is more suitable in these cases than is a too-rigid or brittle, failure-prone lining. If the ground has sinkhole potential, a tunnel structure that can be repaired easily may be more economical than a structure designed to allow the bridging of the sinkholes.

5. In-Situ Monitoring

5.1. Purpose of In-Situ Measurements

In-situ monitoring during the excavation and at longer intervals after the tunnel is completed should be regarded as an integral part of the design not only for checking the structural safety and the applied design model but also for verifying the basic conception of the response of the ground to tunnelling and the effectiveness of the structural support.

The main objectives of in-situ monitoring are:

1. To control the deformations of the tunnel, including securing the open tunnel profile. The time-history development of displacements and convergence may be considered one safety criterion, although field measurements do not yield the margins the structure can endure before failing.
2. To verify that the appropriate tunnelling method was selected.
3. To control the settlements at the surface, e.g. in order to obtain information on the deformation pattern in the ground and on that part of settlements caused by lowering the water level.
4. To measure the development of stresses in the structural members, indicating sufficient strength or the possibility of strength failure.
5. To indicate progressive deformations, which require immediate action for ground and support strengthening.
6. To furnish evidence for insurance claims, e.g. by providing results of levelling the settlements at the surface in town areas.

5.2. Monitoring Methods

A programme for monitoring the deformations and stresses during the excavation may comprise the following measurements (see Fig. 8):

1. Levelling the crown (at the least) inside the tunnel as soon as possible. With regard to interpretation of the data, Fig. 2 reveals that often only a small fraction of the entire crown movement can be monitored because a larger part occurs before the bolt can be set. For difficult tunnelling, the distance between two crown readings may be as close as 10-15 m. Levelling of the invert is recommended for rock having swelling potentials.
2. Convergence readings (in triangular settings; K in Fig.
be installed from the surface well ahead of the tunnelling face, with stress cells for reading the ground pressures and ring forces in the lining (G and R in Fig. 8).

(4) Stress cells also should be installed in a few sections of the final second lining if long-term readings are desired after the tunnel has been completed.

(5) Surface levelling along the tunnel axis and perpendicular to it yield settlements and the correlation to measurements inside the tunnel (see Fig. 2).

(6) Extensometers, inclinometers, gliding micrometers may be installed from the surface well ahead of the tunnelling face, yielding deformation measurements within the ground (see Fig. 8). Monitoring of the ground deformations is especially appropriate for checking and interpreting the design model. Therefore, the installation should be combined with convergence readings and stress cells in the same cross-section.

The frequency of the readings depends on how far from the tunnelling face the measurements are taken, and on the results. For example, readings may be performed initially two times a day; then be reduced to one reading per week four times a day; then be reduced to one reading per month if the time-data curves justify this reduction in measurement readings.

The readings may promote visual understanding of the structural behavior of ground and support interaction.

The readings may cover only a fraction of the actual phenomena if bolts and stress cells are installed too late (see Fig. 2).

The tunnel may be considered stable when all the readings cease to increase. However, a safety margin against failure—especially sudden collapse—cannot be deduced from measurement, except by extrapolation.

6. Guidelines for the Structural Detailing of the Lining

On design aspects with regard to maintenance the reader is referred to other recommendations of the ITA (see T&UST 2:8). For concrete linings, the following structural design specifications are suggested.

(1) The thickness of a second lining of cast-in-place concrete may have a lower limit of 25–30 cm to avoid concrete placing problems such as undercompaction or honeycombing of concrete. The following lower limits may be recommended:

- 20 cm, if lining is unreinforced;
- 25 cm, if lining is reinforced;
- 30 cm for watertight concrete.

(2) Reinforcement may be desirable for crack control, even when it is not required for covering inner stresses. On the other hand, reinforcement may cause concrete-placing problems or long-term durability problems due to steel corrosion. If reinforcement in the second lining is provided for crack control, a closely-spaced steel mesh reinforcement may have the following cross-sections in both directions:

- At the outer surface, at least 1.5 cm²/m of steel;
- At the inner surface, at least 3.0 cm²/m of steel.

(3) The recommended minimum cover of reinforcement is:

- 3.0 cm At the outer surface if a waterproof membrane is provided.
- 5.0 cm–6.0 cm At the outer surface if it is directly in contact with the ground and ground water.
- 4.0 cm–5.0 cm At the inner tunnel surface.
- 5.0 cm For the tunnel invert and where water is aggressive.

(4) For lining segments, specifications (1), (2) and (3) above are not valid, especially if the segmented tunnel ring is the outer preliminary lining. For detailing the tunnel segments, special attention should be given to avoiding damage during transport and erection.

(5) Sealing against water (waterproofing sheets) may be necessary under the following conditions:

- When aggressive water action threatens to damage concrete and steel.
- When the water pressure level is more than 15 m above ground level.
- When there is a possibility of freezing of ingressing water along the tunnel section close to the portals.
- When the inner installations of the tunnel must be protected.

(6) In achieving watertightness of concrete, special specifications of the concrete mixture, avoidance of shrinkage stresses and temperature gradients during setting, and the final quality of the concrete are much more important than theoretical computations of crack widths.

(7) Temperature effects (tension stresses) may be somewhat controlled by working joints (as close as 5 m at the portals) and by additional surface reinforcement in concrete exposed to low temperatures.

(8) An initial lining of shotcrete may be considered to participate in providing stability of the tunnel only when the long-term durability of the shotcrete is preserved. Requirements for achieving long-term durability include the absence of aggressive water, the limitation of concrete additives for accelerating the setting (liquid accelerators), and avoiding shotcrete shadows behind steel arches and reinforcements.
Figure 9. Table of measured data and encountered conditions along a tunnel in France.

<table>
<thead>
<tr>
<th>Station</th>
<th>Height above sea level (m)</th>
<th>Tunneling process, design principle</th>
<th>Classes of tunneling excavation according to the StLB</th>
<th>Rock mass compression strength, uniaxial</th>
<th>Ground deform modulus $E_v$ MN/m$^2$</th>
<th>Unit weight of rock mass $\gamma$ kN/m$^3$</th>
<th>Crown load assumption for bedded beam model $P_0$ and $P_0/t_0$, kN/m$^2$</th>
<th>Water pressure at the height $h$ for final state kN/m$^2$</th>
<th>Special design consideration</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6347</td>
<td>Op. Min.</td>
<td>18.5, 8.8 (6)</td>
<td>180</td>
<td>21</td>
<td>225, 255, 255</td>
<td>225, 225, 225</td>
<td>0</td>
<td>Sinusoidal potential</td>
</tr>
<tr>
<td>2</td>
<td>6337</td>
<td>Op. Min.</td>
<td>6.6 (7) (8) (9)</td>
<td>500</td>
<td>25</td>
<td>225, 225, 225</td>
<td>225, 225, 225</td>
<td>0</td>
<td>Sinusoidal potential</td>
</tr>
<tr>
<td>3</td>
<td>6343</td>
<td>Min.</td>
<td>6.6, 6.6 (10)</td>
<td>500</td>
<td>25</td>
<td>225, 225, 225</td>
<td>225, 225, 225</td>
<td>0</td>
<td>Sinusoidal potential</td>
</tr>
<tr>
<td>4</td>
<td>6357</td>
<td>Min.</td>
<td>6.6, 6.6 (11)</td>
<td>500</td>
<td>25</td>
<td>225, 225, 225</td>
<td>225, 225, 225</td>
<td>0</td>
<td>Sinusoidal potential</td>
</tr>
<tr>
<td>5</td>
<td>6337</td>
<td>Min.</td>
<td>6.6, 6.6 (12)</td>
<td>500</td>
<td>25</td>
<td>225, 225, 225</td>
<td>225, 225, 225</td>
<td>0</td>
<td>Sinusoidal potential</td>
</tr>
<tr>
<td>6</td>
<td>6353</td>
<td>Min.</td>
<td>6.6, 6.6 (13)</td>
<td>500</td>
<td>25</td>
<td>225, 225, 225</td>
<td>225, 225, 225</td>
<td>0</td>
<td>Sinusoidal potential</td>
</tr>
<tr>
<td>7</td>
<td>6353</td>
<td>Min.</td>
<td>6.6, 6.6 (14)</td>
<td>500</td>
<td>25</td>
<td>225, 225, 225</td>
<td>225, 225, 225</td>
<td>0</td>
<td>Sinusoidal potential</td>
</tr>
<tr>
<td>8</td>
<td>6353</td>
<td>Min.</td>
<td>6.6, 6.6 (15)</td>
<td>500</td>
<td>25</td>
<td>225, 225, 225</td>
<td>225, 225, 225</td>
<td>0</td>
<td>Sinusoidal potential</td>
</tr>
<tr>
<td>9</td>
<td>6353</td>
<td>Min.</td>
<td>6.6, 6.6 (16)</td>
<td>500</td>
<td>25</td>
<td>225, 225, 225</td>
<td>225, 225, 225</td>
<td>0</td>
<td>Sinusoidal potential</td>
</tr>
<tr>
<td>10</td>
<td>6353</td>
<td>Min.</td>
<td>6.6, 6.6 (17)</td>
<td>500</td>
<td>25</td>
<td>225, 225, 225</td>
<td>225, 225, 225</td>
<td>0</td>
<td>Sinusoidal potential</td>
</tr>
<tr>
<td>11</td>
<td>6353</td>
<td>Min.</td>
<td>6.6, 6.6 (18)</td>
<td>500</td>
<td>25</td>
<td>225, 225, 225</td>
<td>225, 225, 225</td>
<td>0</td>
<td>Sinusoidal potential</td>
</tr>
<tr>
<td>12</td>
<td>6353</td>
<td>Min.</td>
<td>6.6, 6.6 (19)</td>
<td>500</td>
<td>25</td>
<td>225, 225, 225</td>
<td>225, 225, 225</td>
<td>0</td>
<td>Sinusoidal potential</td>
</tr>
</tbody>
</table>

Figure 10. Predicted ground conditions, tunnelling classes and design characteristics along a tunnel of the rapid railway line in Germany.
| Kilo meter | Geological Formation | Ni: Nishiyama Formation | Si: Shikino Formation | Ni | Ni
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rock Name</td>
<td>all of s.s. and m.s.</td>
<td>s.s. and m.s. (s.p.)</td>
<td>s.s. and m.s.</td>
<td>m.s. and m.s.</td>
</tr>
<tr>
<td></td>
<td>Seismic Velocity</td>
<td>1.4~1.6</td>
<td>1.7~2.0</td>
<td>2.5</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>Unconfined Compressive Strength (Competence Factor)</td>
<td>64~68</td>
<td>61~64</td>
<td>2.3</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>Water Inflow</td>
<td>a little</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Rock Class</td>
<td>s.s.: sandstone</td>
<td>m.s.: mudstone</td>
<td>s.s.: sandstone predominates</td>
<td></td>
</tr>
</tbody>
</table>

**Note**: squeezing property

Figure 11. Predicted ground conditions along a tunnel line (example submitted by Japan).

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Figure 12. Documentation of geology, ground classes, support, geotechnical field measurements gathered during a tunnel project in Austria.
7. Examples of Presentation of Tunnel Design Data

Figures 9-12 are national examples of tabulated information on geotechnical conditions and design characteristics given in condensed form along a longitudinal tunnel section. This information may be part of the tendering documents and should be amended with ongoing tunnelling. By gathering the data actually encountered along the tunnel line in a similar table, a comparison can be made between predicted and actual tunnelling conditions.

References


Note

1See, for example, the Swiss SIA Dokument 260 or the corresponding U.S.-ASCE Code.
<table>
<thead>
<tr>
<th>Country</th>
<th>Organizations/Recommendations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Japan</td>
<td>Tunnel Engineering Committee, Japan Society of Civil Engineering, Japan Tunnelling Association</td>
</tr>
<tr>
<td></td>
<td>Standard Specifications for Tunnels:</td>
</tr>
<tr>
<td></td>
<td>Recommendations for use of bolting. Tunnels et Ouvrages Souterrains 73 (Jan./Feb. 1986), pp. 18-38:</td>
</tr>
<tr>
<td></td>
<td>Recommendations for use of the convergence-confinedment method. Tunnels et Ouvrages Souterrains 67 (Jan./Feb. 1985), pp. 32-43:</td>
</tr>
<tr>
<td></td>
<td>Recommendations for the selection of tunnel support. Tunnels et Ouvrages (1984), pp. 80-97:</td>
</tr>
<tr>
<td>Switzerland</td>
<td>Recommandation SIA No. 199: Etude du massif rocheux pour les travaux souterrains. 1975. (Also in German)</td>
</tr>
<tr>
<td></td>
<td>Norme SIA No. 198: Travaux souterrains (avancement à l'explosif). 1975. (Also in German)</td>
</tr>
<tr>
<td></td>
<td>Recommandation SIA No. 198/1: Construction de tunnels et de galeries en rocher au moyen de tunneliers. 1985. (Also in German)</td>
</tr>
<tr>
<td>United States of America</td>
<td>American Society of Civil Engineers (ASCE)</td>
</tr>
</tbody>
</table>