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 probings 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.*

Résumé—*Le groupe de travail AITES sur le dimensionnement des tunnels présente ici son deuxième rapport. En rassemblant toutes les informations, qui étaient accessibles entre les pays sur le dimensionnement des tunnels, nous espérons, que les expériences gagnées sur beaucoup de projets des travaux souterrains seront propagées dans tout le monde. Parce que le sol participe d'une grande partie à fournir des moyens de stabilité pour des ouvertures souterraines, des méthodes de dimensionnement comprennent aussi bien les investigations sur le chantier, les essais laboratoires et la surveillance pendant le progrès du travail que l'analyse des contraintes et des déformations. Concernant ce dernier point, des modèles de dimensionnement différents et actuellement appliqués sont présentés, y compris aussi la méthode d'observation. Recommendations pour les détails de revêtement et quelques recommandations nationales sur le dimensionnement des tunnels achèvent ce rapport.*

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 testings 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.

This report is edited by Heinz Duddeck, Animateur of the ITA Working Group on General Approaches in the Design of Tunnels. Present address: Prof. Heinz Duddeck, Technical University of Braunschweig, Beethovenstrasse 51, 3300 Braunschweig, Federal Republic of Germany.

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 consecutive 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.

- Plans for and a description of the projected excavation or driving procedure, including the different cross-sections related to different ground conditions.
- Design documents for the types of excavation methods and tunnel supports likely to be applied, considering, e.g. excavation advance and face support (types and number of anchors, shotcrete strength, closure length, etc.).
- The program for the *in-situ* monitoring of the tunnel by field measurements.
- The analysis of stresses and deformations (for unlined tunnels as well as for single-or double-lined tunnels), and the dimensioning of the tunnel support for intermediate phases and final linings.
- The design for waterproofing or drainage.
- Structural documents for the final design of the tunnel project, including the detailing.
- During and after the excavation, reports on the field measurements and interpretation of their results with respect to the response of the ground and the structural safety of the tunnel.
- Documentation of the problems encountered during the excavation and measures applied, e.g. strengthening the ground or changing the projected type of support, based on monitoring results.

The above sequence of these basic documents also provides the general outline of the design procedure.

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.

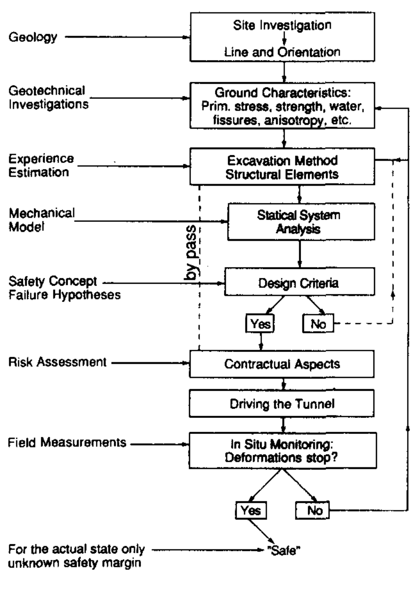


Figure 1. Design process for tunnelling.

(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 $w/w_0 = 1.0$ 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.

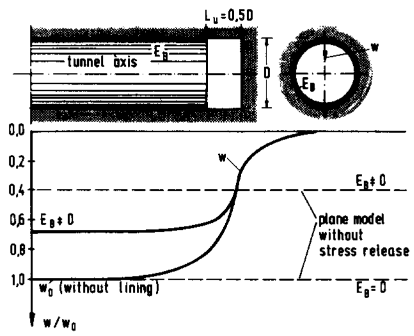


Figure 2. Crown displacement w *along the axis, ahead and beyond the tunnel face.*

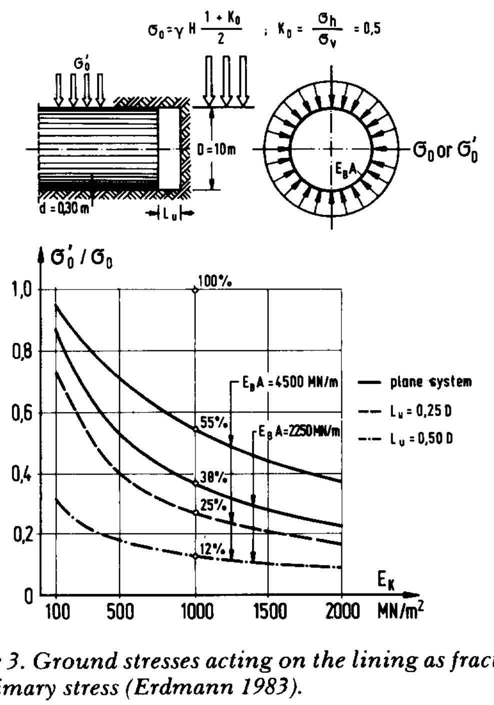


Figure 3. Ground stresses acting on the lining as fractions of the primary stress (Erdmann 1983).

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 $E_BA = 15{,}000 \times 0.3 = 4500$ MN/m and a ground deformation modulus of $E_K = 1000$ MN/m².

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_BA = 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 $L_U = 0.5\ D$, 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 interpretating 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, 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.

2.5. *Design Criteria and Evaluating Structural Safety*

An underground structure may lose its serviceability or its structural safety in the following cases:

— The structure loses its watertightness.
— The deformations are intolerably large.
— The tunnel is insufficiently durable for its projected life and use.
— The material strength of the structural elements is exhausted locally, necessitating repair.
— The support technique (for example, in erecting segmental linings) fails or causes damage.
— Exhaustion of the material strength of the system causes structural failure, although the corresponding deformations develop in a restrained manner over time.
— The tunnel collapses suddenly because of instability.

The structural design model should yield criteria related to failure cases, against which the tunnel should be designed safely. These criteria may be:

- Deformations and strains.
- Stresses and utilization of plasticity.
- Cross-sectional lining failure.
- Failure of ground or rock strength.
- Limit-analysis failure modes.

In principle, the safety margins may be chosen differently for each of the failure cases listed above. However, in reality the evaluation of the actual safety margins is most complex and very much affected by the scattering of the involved properties of the ground and the structure and, furthermore, by the interacting probabilistic characteristics of these properties. Therefore, the results of any calculation should be subject to critical reflection on their relevance to the actual conditions.[1]

National codes for concrete or steel structures may not always be appropriate for the design of tunnels and the supporting elements. Computational safety evaluations should always be complemented by overall safety considerations and risk assessments employing critical engineering judgment, which may include the following aspects:

- The ground characteristics should be considered in light of their possible deviations from average values.
- The design model itself and the values of parameters should be discussed by the design team, which includes all of the experts involved (see Section 2.1, "General Procedure in Designing a Tunnel," above).
- Several and more simple calculation runs with parametric variations may uncover the scattering of the results. In general, this approach is much more informative than a single over-refined investigation.
- The *in-situ* measurements should be used for successive adjustment of design models.
- Long-term measurement of deformations via extrapolation may reveal to a large extent the final stability of the structure, although sudden collapse may not be announced in advance.

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

Zoning. The ground 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 least estimate the stress ratio σ_h/σ_v at tunnel level, where σ_h is the lateral ground pressure and σ_v 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, hydroxydes, or clay minerals should be investigated by mineralogical testing. A special odeometer 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, γ, γd, γz.
- Water content w.
- Permeability k.
- Core recovery.

2. *Mechanical properties* determined by laboratory testing:

- Friction angle ϕu, ϕ.
- Cohesion c_u, c.
- Compressibility m_v, c_v.

3. *Mechanical properties* detemined by field testing:

- Shear strength τ_v (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 arching action in the ground and the lining may develop.

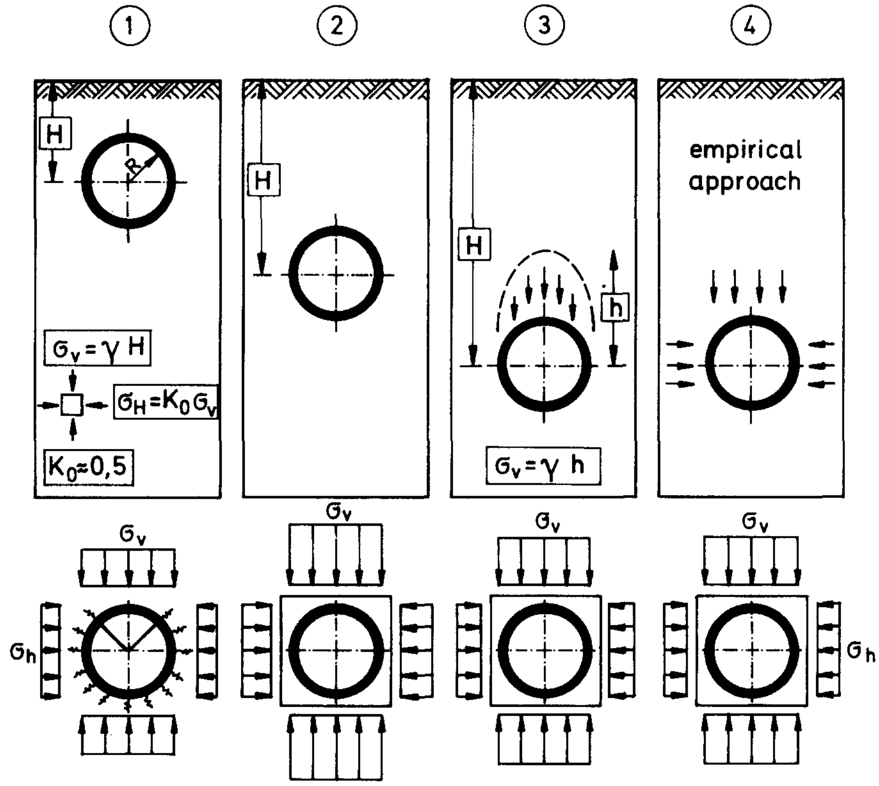


Figure 4. Alternative plane-strain design models for different depths and ground stiffnesses.

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.

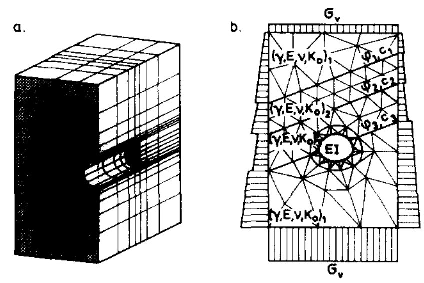


Figure 5a. Three-dimensional continuum model.
Figure 5b. Example of two-dimensional finite-element model.

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 behaviour, etc., may be covered by special assumptions for material laws.

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 β may be smaller than 200:

$$\beta = E_s R^3 / EJ < 200,$$

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,
EJ 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 β 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.

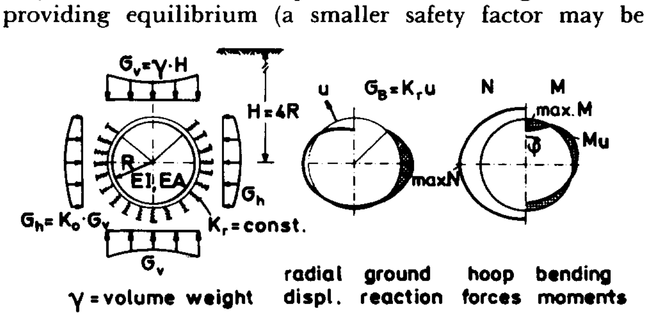


Figure 6. Example of a bedded-beam model for shallow tunnels.

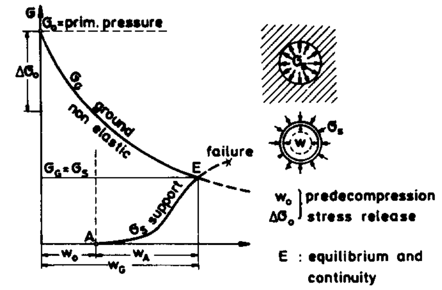


Figure 7. Characteristic curves for the ground and the support for convergence-confinement models (Fenner-Pacher curves).

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 σ_0 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.

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

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.

4.5. *Observational Method*

By combining analytical methods with the empirical approach and the immediate interpretations of *in-situ* measurements, a tunnelling design procedure that is adjustable as the tunnel excavation proceeds may be applied. In this approach, the field measurements of ground movements, displacements and stresses in the lining are used on an ongoing basis to verify or modify the design of the tunnel. More intensively instrumented sections at the early stages of the tunnelling provide the data for these procedures. The interpretation of the measured data yields insight into the ground behaviour as a reaction to the tunnelling procedure.

In applying the observational method, the following conditions must be met:

- The chosen tunnelling process must be adjustable along the tunnel line.
- Owner and contractor must agree in advance on contractual arrangements that allow for modifications of the design on an ongoing basis during the project.
- The field measurements should be interpreted on the basis of a suitable analytical concept relating measurement data to design criteria.
- The interpretation of a particular instrumented section must be used to draw conclusions about the other sections of the tunnel. Hence, the experiences are restricted to those tunnel sections that are comparable with respect to ground conditions, ground cover, etc. (see Section 4.4 "Empirical Approach").
- Field measurement should be provided throughout the entire length of the tunnel in order to check its assumed behaviour.

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

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.

4.6.2. 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 potentials, 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 convergences 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.

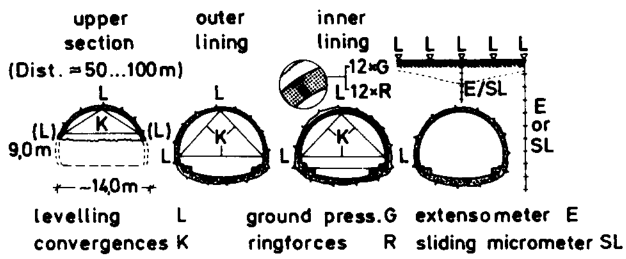


Figure 8. Example of in-situ *monitoring of the tunnel excavation, the preliminary lining, and the surface settlements.*

8) should be the standard method for early information. They are easily applied and are accurate to within 1 mm.

(3) In a few cross-sections, the linings may be equipped 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 diameters behind the face; and end with one reading per month if the time-data curves justify this reduction in measurement readings.

5.3. *Interpreting Results of* In-Situ *Monitoring*

The results of *in-situ* monitoring should be interpreted with regard to the excavation steps, the structural support work, and the structural design model in conjunction with safety considerations.

The actual readings normally show a broad scatter of values. Expectations of reliability may not be met, especially for pressure cells, because stresses and strains are very local characteristics. Deformation and convergence readings are more reliably obtainable because displacements register integrals along a larger section of the ground.

The *in-situ* measurements should be interpreted in consideration of the following:

- The results should verify whether the tunnelling method is appropriate.
- Graphed time-history charts may reveal a decreasing rate of deformation, or uncover danger of collapse.
- Large discrepancies between the theoretically predicted and actually observed deformations may force revision of the design model. However, measurements are valid only for the actual state at the time and the place where they are taken. Long-term influences such as rising water level, traffic vibrations, and long-term creep are not registered during excavation.
- 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* 2:3). 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 cm^3/m of steel;
- At the inner surface, at least 3.0 cm^3/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 the crown.
- 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.

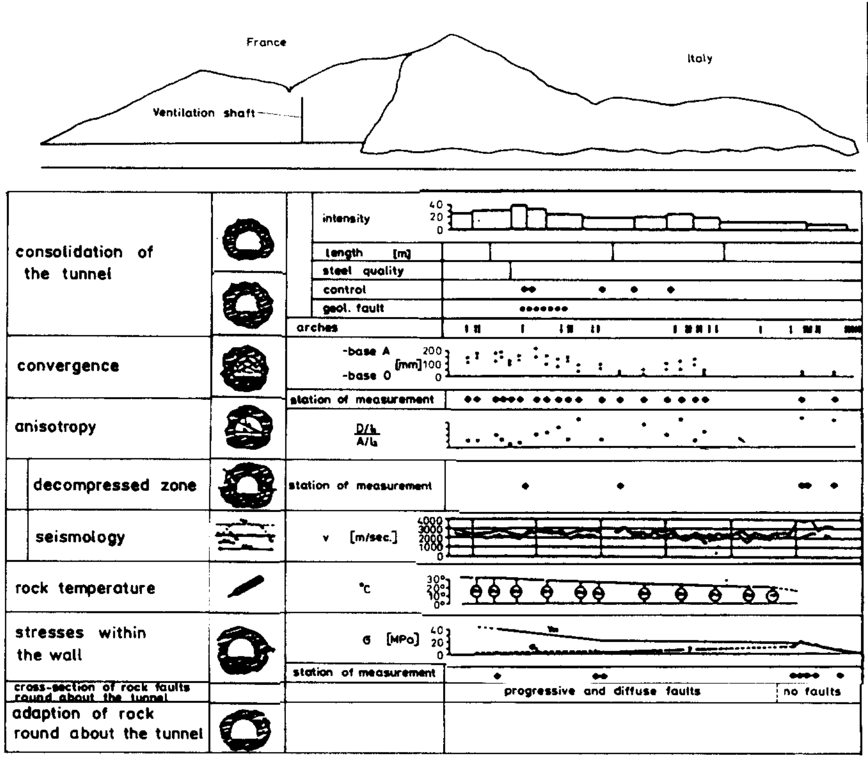
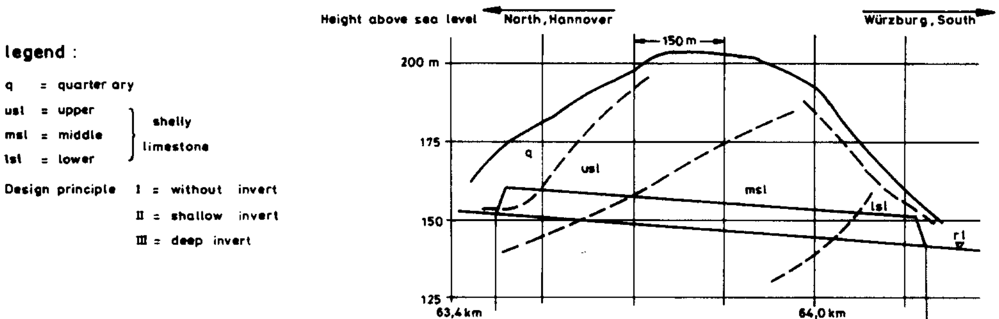


Figure 9. Table of measured data and encountered conditions along a tunnel in France.

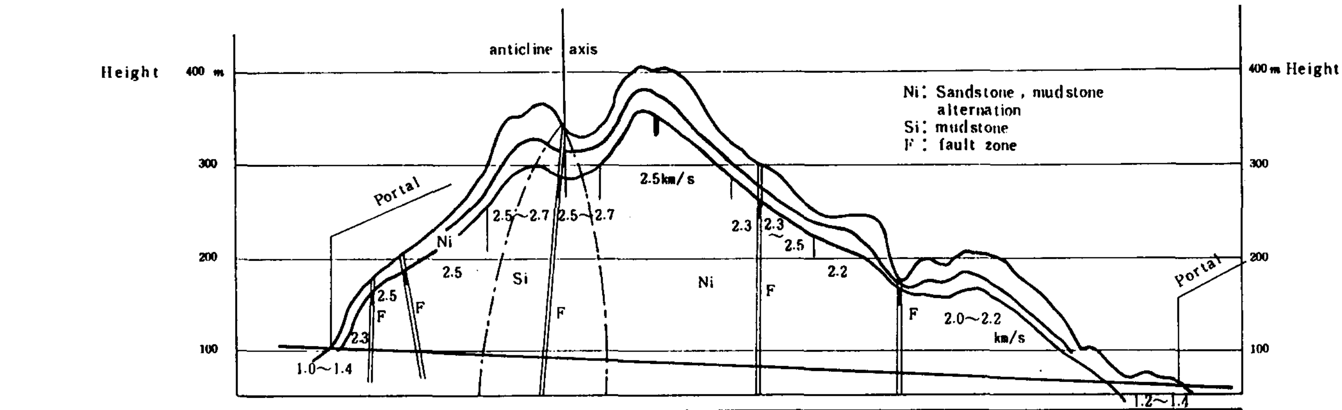


1	Station	km	63,47; 63,50; 63,57; 63,63; 63,68; 63,73; 63,93; 63,95; 64,01; 64,16; 64,19
2	surface height above sea level height of railway line (rl)	m	170,5 / 150,96 ... 153,3 / 142,11
3	Tunneling process; design principle		in open cut | closed crown tunnelling: II | I | in open cut
4	Classes of tunneling excavation according to the StLB		(6.5) 6.6 (8) | (6,5) 6.6 (7) (8) (9) | (6.4) 6.5 (6.6) | (6.3) 6.4 (6.5) 6.6
5	Rock mass compression strengt, uniaxial [1]	MN/m²	0,05 | 1 | 3 | 1 | 0,1 | 1 | 5
6	Ground deform. modulus [1] E_v	MN/m²	150 | 500 | 1000 | 500 | 200 | 500 | 1000
7	Unit weight of rock mass γ	kN/m³	21 | 25 | 24
8	Crown load assumption for bedded beam mod. p_v and $k_o = p_b / p_v$	kN/m²,–	225, 0,5...1,0 | 225, 0,4...0,8 | 150, 0,2...0,5 | 225, 0,4...0,8 | 225, 0,5 . . . 0,8 | 225, 0,4...0,6 | 150, 0,2 . . . 0,4
9	Water pressure at the height of rl for final state	kN/m²	0
10	Special design considevation		← sinkhole potential →
11			
12			

1) average value for design analysis

Technical design conditions (TVR) for the Hopfenberg-Tunnel (Fed.R.Germany) | Longitudinal section with design characteristics

Figure 10. Predicted ground conditions, tunnelling classes and design characteristics along a tunnel of the rapid railway line in Germany.



Kilo meter	41km 500 600 800 · 41km 200 400 600 800 · 42km 200 400 600 800 · 43km 200 400 600 800 · 44km 200
Geological Formation	Ni: Nishiyama Formation · Si: Shiiya Formation · Ni
Rock name	alt. of s.s. and m.s. · s.s. and m.s. (s.p.) · · s.s. and m.s. (s.p.) · s.s. and m.s. · s.s. and m.s. (s.p.) · m.s. · s.s. m.s. (s.p.) · s.s
Seismic Velocity	1.4~1.0 · 2.3 · 1.7 2.5 · 2.0 2.5 · 2.5~2.7 · 1.7 2.5~2.7 · 1.5 2.5 · 2.5 · 1.6 2.3 · 2.3~2.5 · 1.5 2.2 · 2.0~2.2 · 1.2 1.4
Unconfined Compressive Strength (Competence Factor)	σ_c=44~58 kgf/cm² (7.0~9.7) · σ_c=54~64 kgf/cm² (1.0~1.2) · σ_c=61~108 kgf/cm² (1.7~3.0) · σ_c=52~56 kgf/cm² (4.5~4.8)
Water Inflow	a little
Rock Class	I L · II N · I S · I N · II N · I L
Note	· Squeezing Property ·

s.s.: sandstone m.s.: mudstone s.p.: sandstone predominates

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

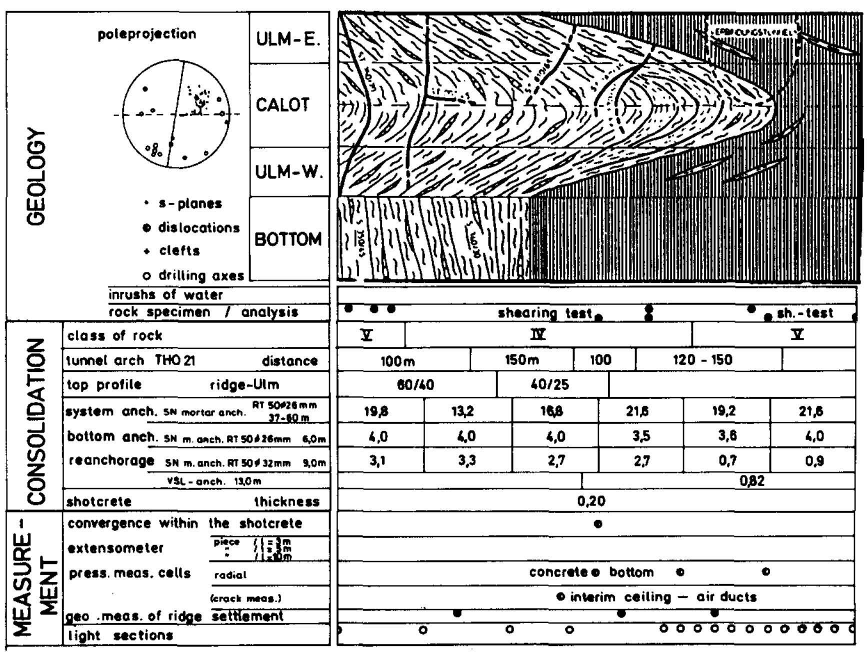


Figure 12. Documentation of geology, ground classes, support, geotechnical field measurements gathered during a tunnel project in Austria.

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

Erdmann, J. 1983. Comparison of two-dimensional and development of three-dimensional design methods for tunnels (in German). Braunschweig, West Germany: Berichte Institut fur Statik, Technical University of Braunschweig.

Gesta, P. 1986. Recommendations for use of the convergence-confinement method. *Tunnels Ouvrages Souterrains* 73: 18-39.

International Society of Rock Mechanics Commission on Classification of Rocks and Rock Masses. 1981. *Int. J. Rock Mechanics Mining Sci.* 18: 85-110.

International Society of Rock Mechanics. 1975. ISRM Recommendations on site investigation techniques.

International Tunnelling Association Working Group on Structural Design of Tunnels. 1982. *Advances Tunnell. Technol. Subsurface Use* 2(3): 153-228.

Note

[1]See, for example, the Swiss SIA Dokument 260 or the corresponding U.S.-ASCE Code.

Appendix. International and National Recommendations on Structural Design of Tunnels.

Although the following selected list of recommendations by national and international organizations is not complete, it nevertheless should provide the reader with sources of additional information regarding the design of tunnels.

Organization/Country	Publication
International Tunnelling Association (ITA).	Views on structural design models for tunnelling. *Advances in Tunnelling Technology and Subsurface Use* 2:3 (1982).
International Society for Rock Mechanics (ISRM)	ISRM recommendations on site investigation techniques, July 1975.
	ISRM Committee on Field Tests: Document No. 1—Suggested Method for Determining Shear Strength
	Document No. 2—Suggested Methods for Rock Bolt Testing
	ISRM Committee on Laboratory Tests; ISRM Committee on Swelling Rocks: Document No. 1—Suggested Methods for Determining the Uniaxial Compressive Strength of Rock Materials and Point Load Strength Index.
	Document No. 2—Suggested Methods for Determining Water Content, Porosity, Density, Absorption and Related Properties, Swelling and Slake Durability Index Properties.
Australia	Australian Standard 1726 - S.A.A. Site Investigation Code.
	Australian Standard 1289 - Methods of Testing Soils for Engineering Purposes.
Austria	ÖNORM B 2203 Untertagebaunorm, Richtlinien und Vertragsbestimmungen, Werkvertragsnorm.
	Projektierungsrichtlinien für Geotechnische Arbeiten. RVS 9.240 u. 9.241, Forschungsges. Srassenwesen. Nov. 1977.
Federal Republic of Germany (in German)	Recommendations for the design of underground openings in rock. *Tunnelbau-Taschenbuch 1980*, Gluckauf-Verlag, Essen (1980), pp. 157-239.
	Recommendations for the analysis of Tunnels in soft ground (1980), *Bautechnik 10* (1980), Berlin, pp. 349-356.
	Recommendations for the Concrete Lining of Tunnels in soft ground (1986). *Bautechnik 10* (1986), Berlin, pp. 331-338.
France	*Tunnels et Ouvrages Souterrains*, Special Issue July 1982, pp. 32-123:
	Réflexions sur les méthodes usuelles de calcul du revêtement des souterrains (Usual calculation methods for the design of tunnel linings).

	Présentation de la méthode de construction des tunnel avec soutènement immediat par béton projeté et boulonnage (Presentation of the tunnel construction method with immediate support by shotcrete and bolting).
	Recommandations sur les conditions d'emploi du boulonnage (Recommendations for conditions of the use of bolting).
	Tunnels et Ouvrages Souterrains 73 (Jan./Feb. 1986), pp. 18–38: Recommendations for use of the convergence-confinement method.
	Tunnels et Ouvrages Souterrains 67 (Jan./Feb. 1985), pp. 32–43: Recommandations relatives au choix d'un type de soutènement en galerie (Recommendations for the selection of tunnel support).
	Tunnels et Ouvrages (1984), pp. 80–97: Recommandations relatives à l'emploi des citres dans la construction des ouvrages souterrains (Recommendations on the use of steel arches as temporary support in tunnel structures).
Japan Tunnel Engineering Committee, Japan Society of Civil Engineering, Japan Tunnelling Association	Standard Specifications for Tunnels: Mountain Tunnelling Method. Nov. 1986. Shield Tunnelling Method. June 1986. Cut-and-cover Method. June 1986.
Switzerland	Recommandation SIA No. 199: Etude du massif rocheux pour les travaux souterrains. 1975. (Also in German)
	Norme SIA No. 198: Travaux souterrains (avancement à l'explosif). 1975. (Also in German)
	Recommandation SIA No. 198/1: Construction de tunnels et de galeries en rocher au moyen de tunneliers. 1985. (Also in German)
United Kingdom	British Standard 1377. Methods of test for soils for civil engineering purposes, British Standards Institution, 1975.
	British Standard 5930, Code of Practice for site investigations, British Standards Institution, 1981.
	Craig, R. N. and Muir Wood, A. M. A review of tunnel lining practice in the United Kingdom. TRRL Supplementary Report 335, 1978.
	Tunnelling Waterproofing. CIRIA Report 81, 1979.
	Dumbleton, M. J. and West, G. A guide to site investigation procedures for tunnels. TRRL Laboratory Report 740, 1976.
United States of America American Society of Civil Engineers (ASCE)	*Guidelines for Tunnel Lining Design.* Ed. by T. O'Rourke. ASCE Technical Committee on Tunnel Lining Design, Technical Council on Research.