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Lessons Learned from Field Measurements in Tunnelling

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Abstract— *Over the past two decades, various numerical methods of analysis have become popular in the field of geotechnical engineering. However, the accuracy of numerical analyses varies considerably, primarily because of the uncertainties involved in modelling ocmplex geological formations with the complex geomechanical characteristics of soils and rocks. Field measurements carried out during construction can be used to overcome this difficulty. The author reviews ways of using measurement results to improve numerical analyses, including the determination of a "hazard warning level" for each measurement item prior to the start of construction, the use of back analysis. The importance of choosing a proper model is also discussed.* © 1998 Published by Elsevier Science Ltd

1. Introduction

Numerical analyses such as the finite element method (FEM), boundary element method (BEM), and distinct element method (DEM) have become popular in the geotechnical engineering field. These numerical approaches facilitate the consideration of complex geological and geomechanical characteristics in the design of geostructures such as tunnels, underground caverns, foundations of structures, slopes, etc., and the monitoring of the stability of the structures during construction. As a result, these numerical analyses make it possible to achieve a rational design of geostructures.

It is well known, however, that the actual behavior of structures quite often differs from that predicted by numerical analyses. This difference is due mainly to the fact that many uncertainties are involved in the modeling of complex geological formations with the complex geomechanical characteristics of soils and rocks. The initial state of stress also causes difficulties in numerical analyses. To obtain high accuracy in numerical analyses, input data such as geological and geomechanical parameters, initial state of stress, underground water table, permeability of the ground, etc., should be properly determined. This, however, is not an easy task, even though various kinds of advanced exploration techniques have been developed and are already available in practice.

In order to overcome this difficulty, field measurements are carried out during construction. The design parameters used in the original design of the structures can then be assessed on the basis of the results of the field measurements, and, if necessary, the original design and construction/excavation method can be modified. This design/construction method is called the "observational method" (Terzaghi and Peck 1948). With this method, however, a question may arise as to how to interpret the results of field measurements that were made in order to assess the design parameters used in the original design, as well as to monitor the stability of the structures during construction.

As far as monitoring is concerned, the stability of structures can be assessed by comparing the measurement results with their allowable values, and it is obvious that the structures are safe if all the measured values remain smaller than the allowable values. These allowable values are often called "hazard warning levels." Although this approach may be suitable for monitoring the stability of structures during and/or after the construction period, it can hardly be used in assessing the adequacy of the design parameters used in the original design. This difficulty is simply due to the fact that the design parameters cannot be assessed directly from the measured values without doing any analysis of the measured values.

The measurement results must be properly analyzed to assess the design parameters. For this procedure, called "back analysis", the input data are measured values such as displacements, strains, stresses, and pressures, while the output results are material constants, loads, initial state of stresses, permeability, and even boundary conditions. This is exactly the reverse calculation procedure as compared to "forward analysis" commonly used in structural analyses, in which the input data are material constants, loads, initial stresses, and permeability, while the output results are displacements strains, stresses, and pressures.

It is obvious that back analysis is an essential tool in the observational method for assessing design parameters. With regard to engineering practices, it is worth mentioning that back analysis should be carried out immediately after taking the measurements so that the original design and construction methods can be assessed and modified if necessary, without any serious delay during the construction/excavation period.

The observational method for assessing the adequacy of

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the design and construction methods, as well as for monitoring the stability of structures, is shown as a flow chart in Figure 1.

2. Hazard Warning Level

It is recommended that a hazard warning level be determined for each measurement item prior to the start of construction. This will make it possible to assess the stability of the structures immediately after the taking of measurements simply by comparing the measured values to the hazard warning level. When the measured values remain smaller than the hazard warning level, the stability of the structures is confirmed. Even if the measurement values are still smaller than the hazard warning level, however, engineers should always pay attention to what happens after the lapse of a certain period of time (see Fig. 2). If the measured values are predicted to become greater than the hazard warning level after a certain period of time, then the engineers must take some action to stabilize the structures and to modify the original design.

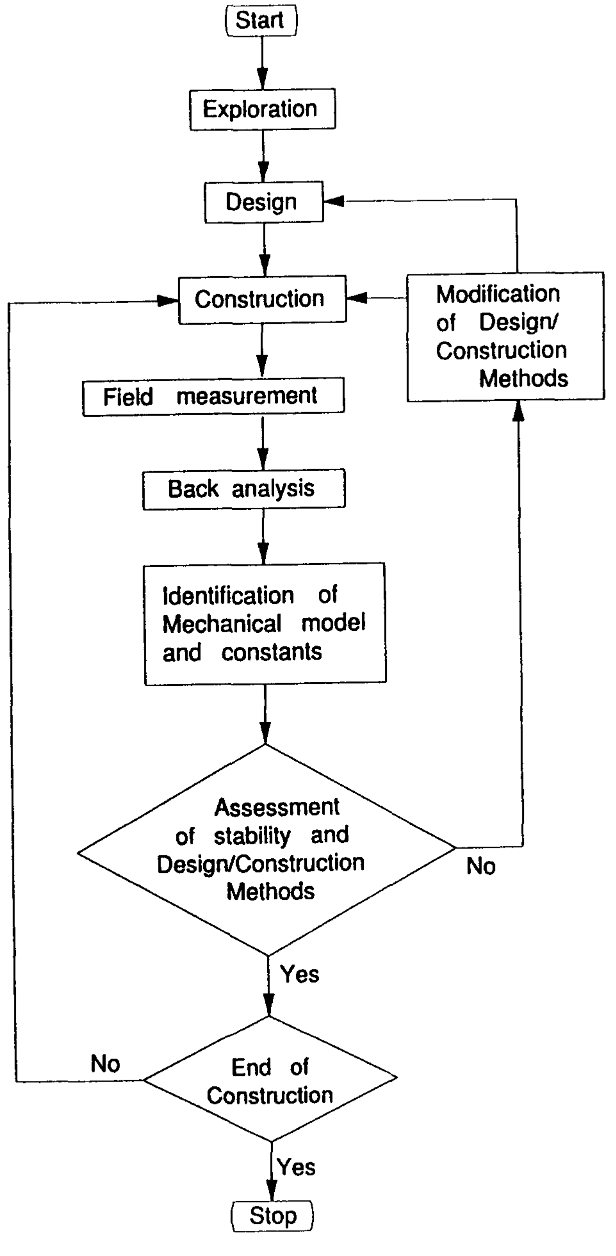


Figure 1. Procedure of assessing the design and construction methods.

The next question is, how is the hazard warning level determined? To answer this question, the author has proposed the critical strain (Sakurai 1981), which can successfully be used for assessing the results of displacement measurements in tunnels, such as crown settlement, convergence, and extensometer and inclinometer measurements.

The definition of the critical strain ε_0 is given as follows:

$$\varepsilon_0 = \frac{\sigma_c}{E} \tag{1}$$

where σ_c is uniaxial compressive strength and E is Young's modulus. It should be noted that the critical strain is always smaller than strain at failure. Various rocks and soils were tested in the laboratory to determine the critical strain. The results are shown elsewhere (Sakurai 1981).

The question may now arise as to how to extend the results obtained from laboratory tests on small specimens to large-scale in-situ soils and rocks. As far as soils are concerned, the critical strain obtained from laboratory tests may be almost the same as that for in-situ soil masses. However, the case of in-situ rock masses requires more discussion.

The critical strain of in-situ rock masses is connected to that of intact rocks by the following equation:

$$\varepsilon_{0R} = \frac{\sigma_{cR}}{E_R} = \frac{m\sigma_c}{nE} = \left(\frac{m}{n}\right)\varepsilon_0 \tag{2}$$

where m and n are reduction factors of uniaxial strength and Young's modulus, respectively, in extending the results obtained from the laboratory to in-situ. Both uniaxial strength and Young's modulus of in-situ rock masses decrease from the values of intact rocks, because of the existence of joints. Thus, the reduction factors m and n range between 0 and 1.0. It should be noted that both the reduction factors m and n for soils must be approximately 1.0. This is the reason why laboratory soil tests are popular in engineering practices.

The values of m and n were determined by operating both laboratory tests and in-situ tests (plate bearing tests and direct shear tests). The ratio of the two ranges was found to be between 1.0 and 3.0, depending on the rock types (Sakurai 1983). This is surprising, in that the critical strain of in-situ rock masses is almost the same order of magnitude as that of intact rocks, though both uniaxial strength and Young's modulus of intact rocks largely differ from those of in-situ rock masses. This is because the effects of joints are canceled out by taking the ratio of the two, although the uniaxial strength and Young's modulus are both greatly influenced by the existence of joints.

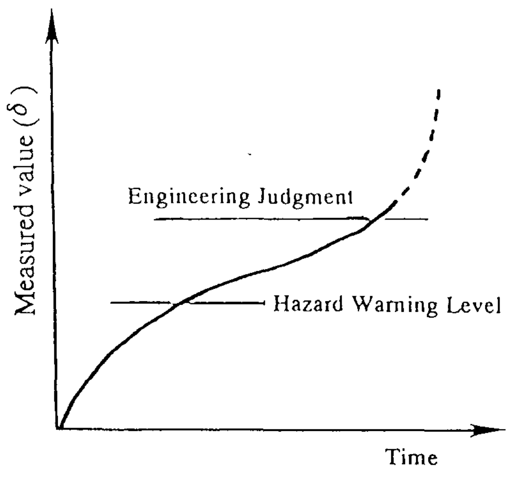


Figure 2. Schematic diagram for measured value in relation to hazard warning level.

In engineering practice, therefore, it may be possible to use the value of critical strain of intact rocks as a hazard warning level for monitoring the stability of tunnels. It should be noted that if we adopt this warning level, the factor of safety from 1 to 3 is automatically included, because the critical strain of in-situ rock masses is always one to three times greater than that of intact rocks. In addition, it is worth mentioning that laboratory tests revealed that the critical strain is not much influenced by various aspects of the environment, such as moisture, temperature, etc. (Sakurai et al. 1994). This is also a great advantage for the critical strain, when it is used in practice.

In order to verify the applicability of the hazard warning level described above for assessing the stability of tunnels, some displacement measurements were carried out. The strains occurring around tunnels as a result of excavation are calculated from measured displacements by Eqs. (3) and (4).

$$\varepsilon_\theta = \frac{u_c}{a} \tag{3}$$

$$\varepsilon_r = \frac{u_1 - u_2}{l} \tag{4}$$

where

u_c is the measured value of crown settlement;

u_1 and u_2 are the displacements measured at the measuring points 1 and 2, respectively, by extensometers installed inwards from the tunnel surface;

a is the tunnel radius; and

l is the length between the two measuring points along the extensometers.

The strains calculated by Eqs. (3) and (4) are plotted in relation to the uniaxial strength of soils and rocks as shown in Figures 3 and 4. The two dotted lines indicate the upper and lower bounds for the critical strain obtained from laboratory tests (Sakurai 1981). The numbers given beside the data indicate the sort of difficulties encountered during the excavation of tunnels, while the data with no numbers are those for tunnels excavated with no serious problems. The types of difficulties are classified as follows:

1) difficulties in maintaining tunnel face;
2) failure or cracking in shotcrete;
3) buckling of steel ribs;
4) breakage of rock bolts;
5) fall-in of roof;
6) swelling at invert; and
7) miscellaneous.

It is seen from these figures that when the strains occurring around the tunnels were smaller than the lower bound of the critical strain, all the tunnels were stable in such a way that they could be excavated with no problems. When the occurring strains reached the upper bound of the critical strain, many different sorts of difficulties occurred. This evidence is exactly that which we expected from the characteristics of critical strains.

Considering the above-mentioned applicability of critical strain, the author has previously proposed a hazard warning level for strain and displacement that can be used in monitoring the stability of tunnels. The hazard warning level is classified into three stages in relation to the degree of stability, as shown in Figure 5. The hazard warning level of settlement at the tunnel crown can then be determined from the corresponding level of strain by using Eq. (3). As an example, the hazard warning levels for crown settlement at a tunnel with a radius of 5 m are also shown in this figure (Sakurai 1993).

3. Back Analysis and Modeling

The hazard warning level described in the previous section can potentially be used for monitoring the stability of tunnels. If the strains occurring around tunnels tend to cross the warning levels, then back analysis must be carried out to re-evaluate the design parameters which were used in the original design. In the geotechnical engineering field, material constants such as Young's modulus, Poisson's ratio, cohesion and internal friction angle are usually determined by back analysis from the measured values of displacements, strains, stresses and pressures. The initial states of stresses are also obtained by back analysis.

It should be noted that these back analyses are considered as parameter identifications, so that they are adequate only when the mechanical models are well defined and fixed. However, the mechanical characteristics of geomaterials such as soils and rocks are so complex that it is extremely hard to define the mechanical model to represent their behavior. In fact, much research is currently being carried out, but there still remain many problems to be solved in the modeling of geomaterials.

The author has already emphasized that in the back analysis of geotechnical engineering problems, the mechanical model should not be assumed, but should be determined by back analysis (Sakurai and Akutagawa 1995). This means that a back analysis in geotechnical engineering practice should be capable of identifying not only the mechanical constants, but also the mechanical model itself.

In forward analysis, a mechanical model is usually assumed or given, such that the ground is represented by a certain model such as elastic, elasto-plastic, visco-elastic-plastic, discrete block models, etc. The values of the mechanical constants of the models can then be determined. Once all the mechanical constants are known, we can calculate displacements, strains and stresses. These results give exact values so that the uniqueness of the solution is confirmed between the input data and output results, at least for an assumed mechanical model. It is extremely important for the forward analysis to assume the most appropriate mechanical model by considering the results of explorations in both laboratory and in-situ.

In back analysis, on the other hand, we first obtain displacements, strains, stresses and pressures as a result of field measurements, and the mechanical constants are then determined by back analysis by assuming a mechanical model. It is no wonder that the values of mechanical constants determined by back analysis depend entirely on which model we assume in back analysis. For instance, if we assume an elastic model, then Young's modulus can be obtained, but if a rigid-plastic model is assumed, then Young's modulus cannot be obtained, though the identical values of measured results are used. This means that the results of back analysis are basically a matter of assumption of the mechanical models representing the behavior of geomaterials. In other words, in back analysis the uniqueness of the solution in general cannot be confirmed between the input data and output results (see Fig. 6).

We can now conclude that back analysis is not simply a reverse calculation of the forward analysis. Its concept should be different from forward analysis in such a way that back analysis can provide modelling as well as identify the parameters of the model.

4. Difference Between Parameter Identification and Back Analyses

In a tunnel design, many uncertainties are involved in evaluating the geological and geomechanical characteristics of soils and rocks, as well as initial state of stresses. In

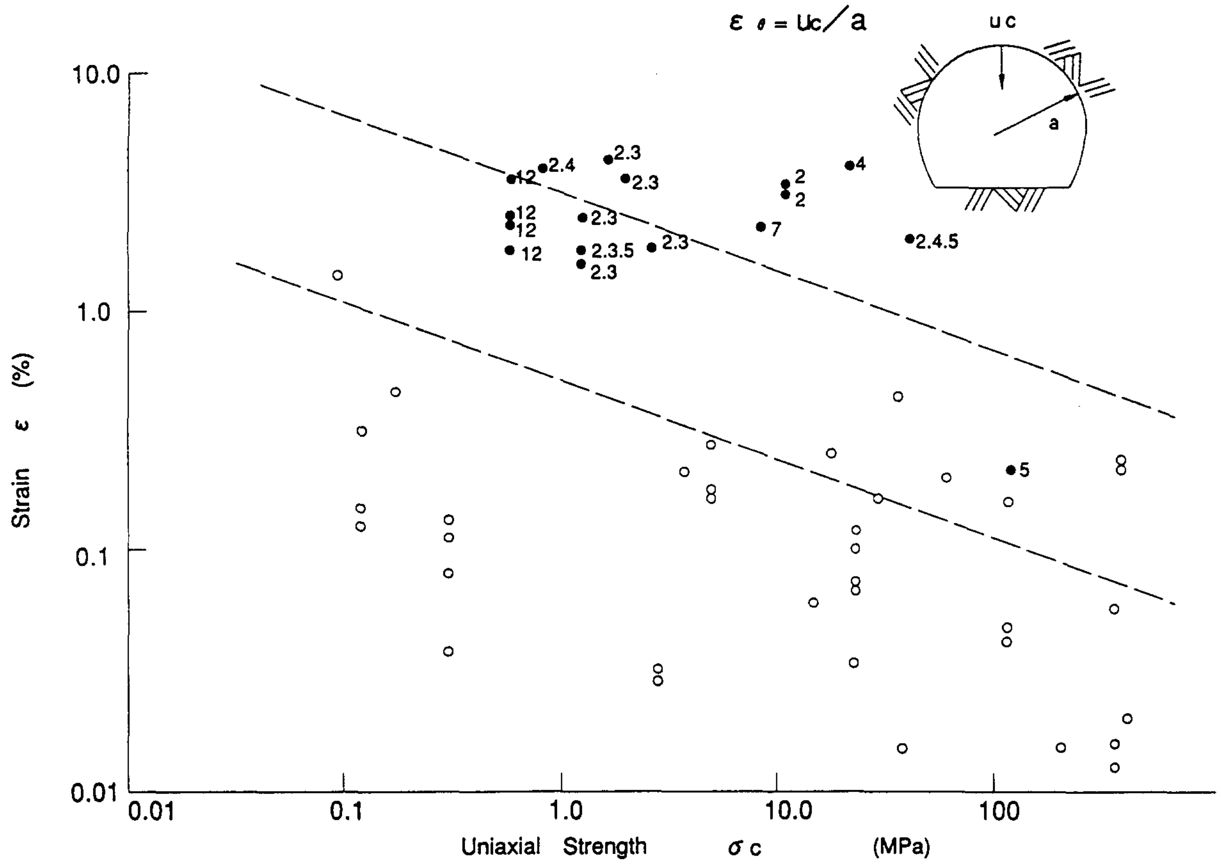


Figure 3. Relationship between the measured strain (obtained from crown settlement) and hazard warning levels.

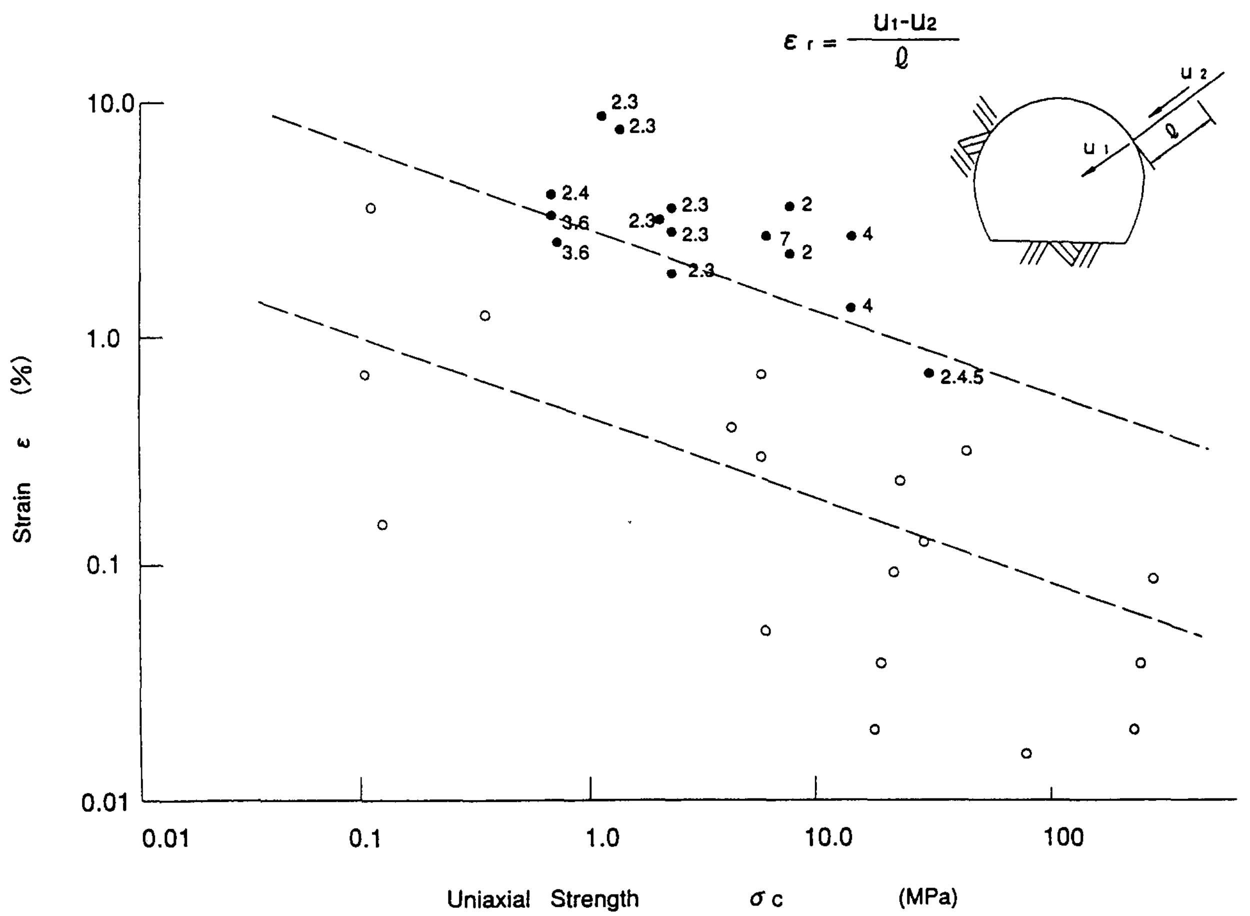
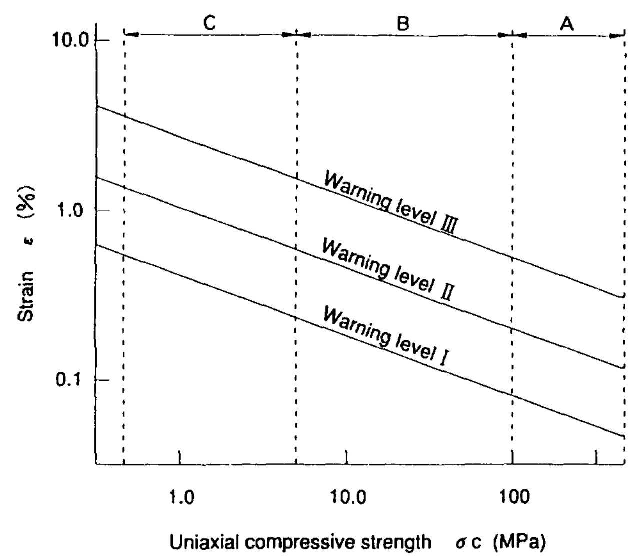


Figure 4. Relationship between the measured strain (obtained from extensometer measurement results) and hazard warning levels.

other words, structures like tunnels are designed under conditions where loads and mechanical properties are not well known and they cannot be controlled. This is entirely different from the situation of bridge-like structures, where loads and mechanical properties of materials are well known since the materials used are all artificial ones like steel and concrete, such that their material properties can be easily controlled. It is possible, of course, to control the strength of soils and rocks, if the strength is not sufficient enough to stabilize tunnels, by installing rock bolts, shotcrete, as well as through injections. However, the mechanical behavior of materials strengthened by rockbolts and shotcrete becomes more complex, so that further assumptions are needed.

Nevertheless, once modeling of the materials is achieved, the material properties such as Young's modulus, Poisson's ratio, cohesion, internal friction angle, joint stiffness, etc., are determined by considering the results of both laboratory and in-situ experiments. The mechanical behavior of tunnels can then be predicted by a computational method such as FEM or BEM, by using these material properties as input data for the computation.

However, as mentioned earlier, the real behavior of the tunnels quite often differs from that predicted by the computational methods. We therefore adopt observational methods to improve agreement between the real and predicted behaviours of tunnels, by modifying the input data that have been used in the computations. This computational procedure is called "parameter Identification", which should be distinguished from "back analysis." In parameter identification, the input data used in the computations are checked after the field measurement results have been analyzed, and can be modified if needed, but the model remains the same the whole time. In back analysis, the modelling should also be checked with field measurements, as well as the



	A	B	C
Ⅰ	0.3~0.5	0.5~1	1~3
Ⅱ	1~1.5	1.5~4	4~9
Ⅲ	3~4	4~11	11~27

(Unit : cm)
(Radius of tunnel: 5.00m)

Figure 5. Hazard warning levels for assessing the stability of tunnels.

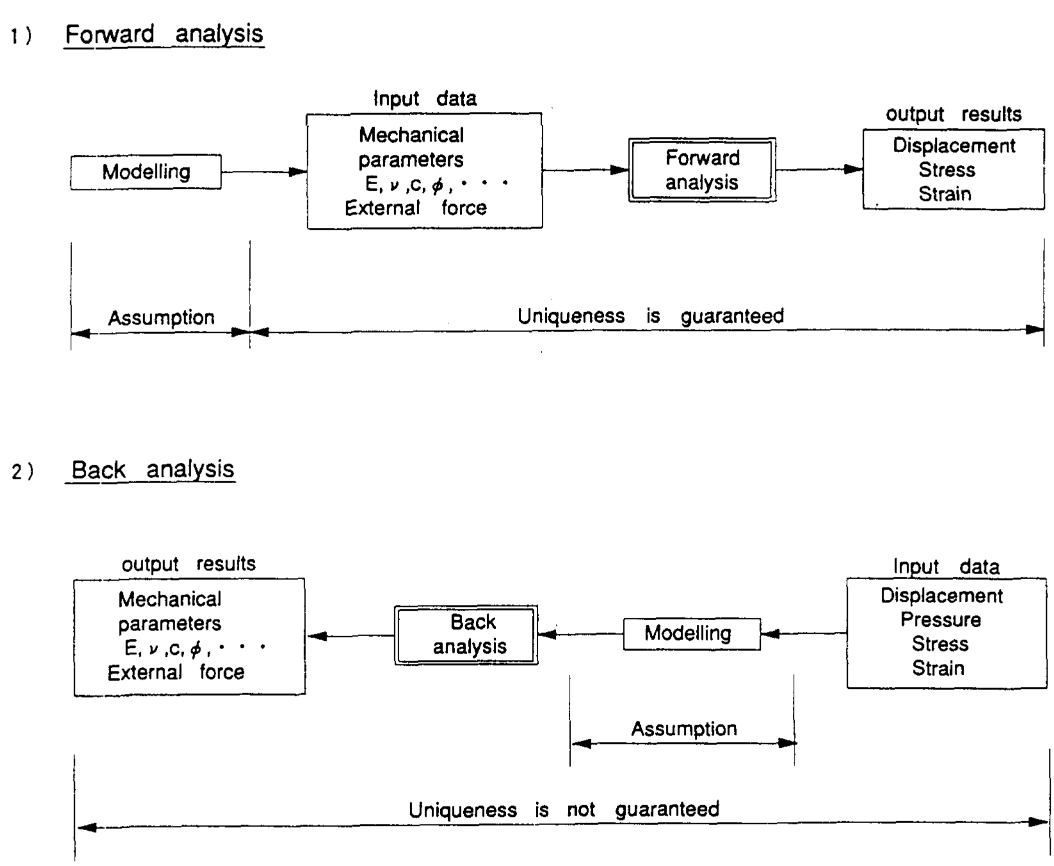


Figure 6. Comparison between the procedures of forward analysis and back analysis.

material properties. Nevertheless, it is common that in the observational methods the input data used in the computation are usually checked during the excavations, but with the modelling being fixed.

As described earlier, it is extremely important in any geotechnical engineering problem that the models should not be assumed, but rather should be determined by a back analysis. If a model is fixed all the time during observational procedures, the results are not only inadequate, but also misleading in their interpretation, in that they provide wrong information in the decision making for modifying design and construction methods. The example discussed below demonstrates how misleading it can be if we assume a model of materials in the observational methods in tunnel practices.

A double-track railway tunnel of shallow depth was constructed underneath a densely populated urban area. The ground in which the tunnel was located consisted of fine grain sand deposits. Both the tunnel diameter and the height of overburden are approximately 10 m. Both extensometers and inclinometers were installed from the ground surface before tunnel excavation so that the total displacements due to excavation could be measured. The ground surface settlements were also measured by surveying.

The measured displacements were interpreted by parameter identification to check the material properties used as the input data in the design analysis.

In this parameter identification, the model of the ground was assumed to consist of homogeneous and isotropic material, so that Young's modulus and Poisson's ratio were the output results of the parameter identification. The identified material properties and initial stresses are used to compute the displacements around the tunnel, and the maximum shear strain distribution is then calculated, as shown in Figure 7. This maximum shear strain is compared with the allowable value, perhaps the critical shear strain, to assess the stability of tunnels.

It is obvious from Figure 7 that no loosening zone occurred in the ground above the tunnel arch. This is no surprise, however, as the model assumed the ground to be homogenous and isotropic. Therefore, the existence of loosening has not been taken into account in this calculation.

In back analysis, on the other hand, we must identify the model as well. In this example problem the non-elastic strain approach is used, in which no assumption are made in modeling; rather, a computer can make a model and tell us if some loosening zone has occurred. The details of this back analysis procedure have been presented elsewhere (Sakurai et al. 1993); only the result of the maximum shear strain distribution is shown in Figure 8. It is of interest to compare the results shown in Figure 7 and Figure 8. These two maximum shear strain distibutions were obtained from identical data, but both are completely different from one another. They depend entirely on which model we used.

The results shown in Figure 8 were obtained without assuming any model, but the modeling was done by a computer. Therefore, the results provided must be closer to the real situation. This means some loosening zone in the ground above the tunnel arch may exist. However, we can say from Figure 7 that no loosening zone is likely to occur in the ground above the tunnel arch. This is a rather dangerous conclusion in assessing the tunnel stability, although Young's modulus and Poisson's ratio can be identified. It should be understood that the back analysis is more important than the parameter identification in tunnelling practices. Moreover, although parameter identification can provide material properties, it sometimes can provide misleading information.

It is also worth mentioning that in this example case study, rock bolts, shotcrete and steel ribs were installed as support measures. In the design analysis, these support structures were considered as stiff elements installed inward from the tunnel surface for rock bolts, and placed on the tunnel surface for shotcrete. However, in back analysis the best agreement between measured and computed results were obtained for the case of no stiffness of support structures. This fact is demonstrated in Figure 9. In this figure, the error function defined in Eq. (5) is plotted as a function of the ratio of Young's modulus of support structure and that of the ground, E_l/E_g (E_l : Young's modulus of support structure; E_g :Young's modulus of the ground).

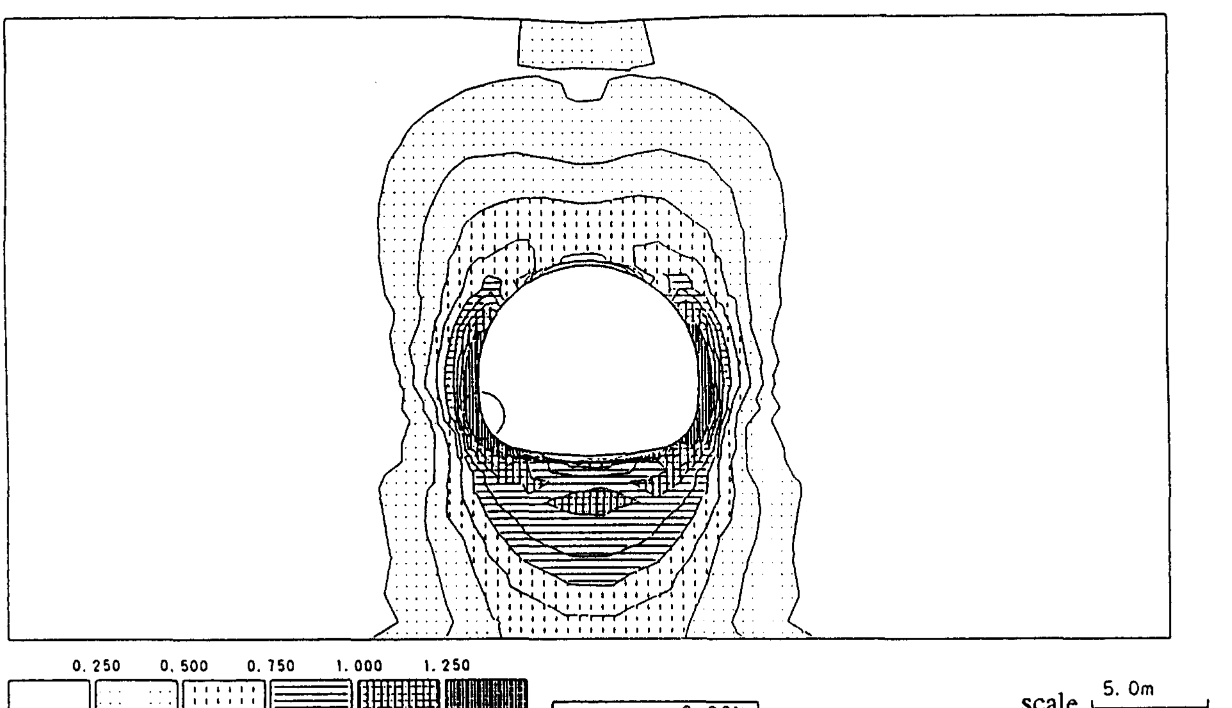


Figure 7. Maximum shear strain distribution (isotropic elastic model).

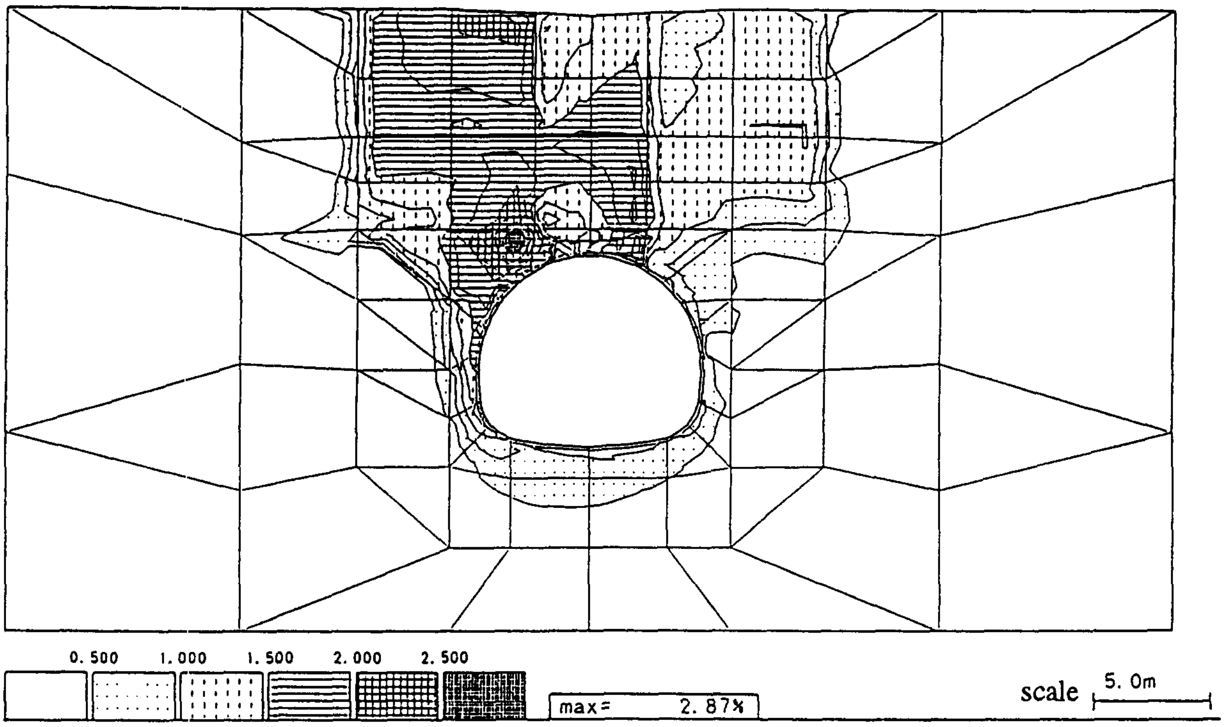


Figure 8. Maximum shear strain distribution (taking into account non-elastic strain).

$$\varepsilon = \frac{\sum_{i=1}^{N} (u_i^m - u_i^c)^2}{\sum_{i=1}^{N} u_i^m} \tag{5}$$

where u_i^m and u_i^c are the measured and computed displacements at the measuring point i, respectively. N is the total number of measurements.

It is seen from this figure that the best agreement between measured and computed displacements is obtained when $E_l/E_g = 1.0$. This means that the best agreement is achieved in the case of the tunnel being unlined. This is a surprising result in that the tunnel behaves in reality just like an unlined tunnel, though support structures such as shotcrete, steel ribs and rock bolts were installed. However, it should bc emphasized that Young's modulus of the ground increases obviously with installation of stiff support structures.

This case study demonstrates the difficulty of modelling the support structures such as shotcrete, steel ribs and rock bolts, and that misleading conclusions can easily be derived if an improper model is adopted.

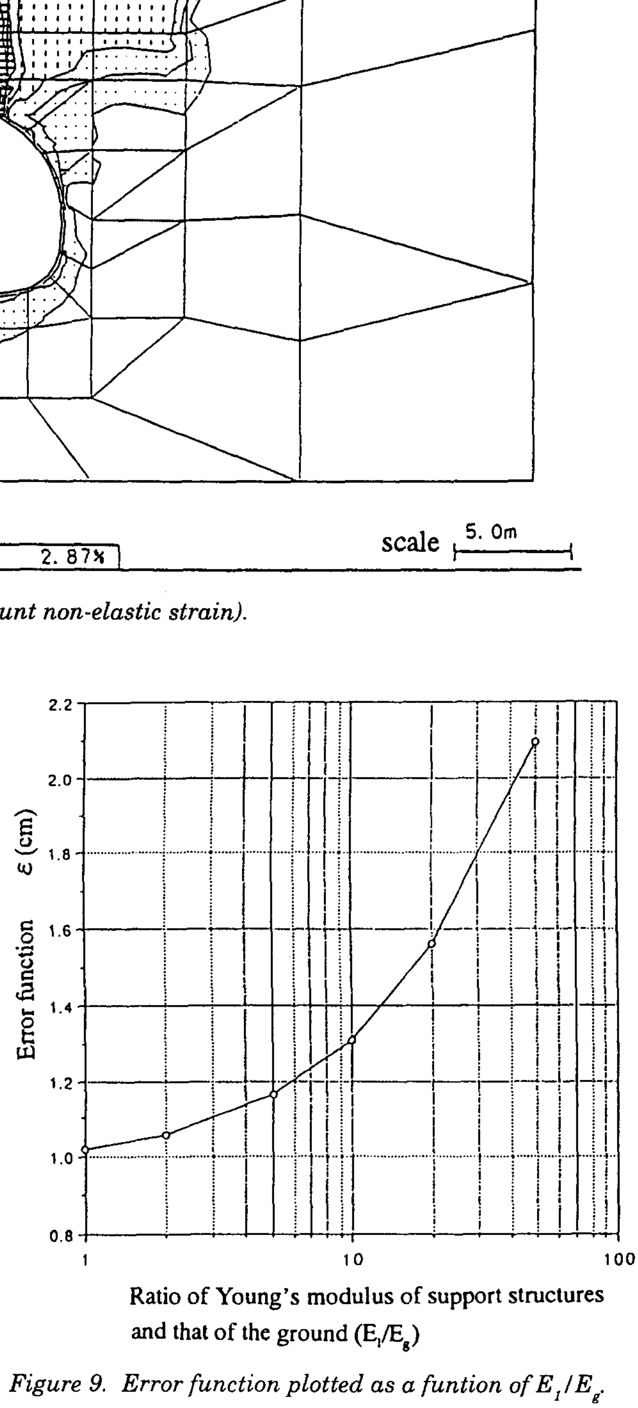


Figure 9. Error function plotted as a funtion of E_l/E_g.

5. Conclusions

1) The observational method is a very promising means of achieving the rational design of tunnels. With this method, however, a crucial problem is how to interpret the results of observation and field measurements taken during/after the excavation.
2) For monitoring the stability of tunnels, the hazard warning levels are extremely important, and they must be determined prior to construction, so that the measured values can be assessed immediately after taking them.
3) In order to determine the hazard warning levels, the critical strain is useful. The critical strain of in-situ rock masses can easily be obtained by laboratory experiments carried out on a small specimen. The critical strain also has an advantage in that it is not much influenced by various environmental factors like water content, temperature, etc. The stability of tunnels is then assessed by comparing strain obtained from measured displacements (crown settlement and extensometer measurements) with the hazard warning levels evaluated from the critical strain.
4) Back analyses are very powerful tools for interpreting the results of field measurements. In back analysis, the model of soils and rocks should not be assumed, but should automatically be obtained by a computer. Therefore, in back analysis not only material proper-

ties but also a mechanical model of soils and rocks should be determined. It should be emphasized that back analysis is entirely different from parameter identification, in which only the material properties are determined from the measurement results, while the model remains the same all the time.

5) An example case study was discussed to demonstrate the difference between back analysis and parameter identification, and to show that parameter identification provides not only less accurate information, but also misleading information about the failure mechanism of tunnels.

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