Inter-active design in Geo-engineering practice

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ABSTRACT
Inter-active design or semi-empirical design is an adaptable design method based on site observation, monitoring and the possibility to strengthen the project even after construction. It involves parametric studies, contingency plans and a detailed in-situ proof of ground-structure interactions based on monitoring. The paper describes the basic principles in connection with case histories.

Keywords : Inter-active design, semi-empirical design, observational method, geotechnical risks

1 INTRODUCTION

"Inter-active design" can be considered rather the opposite of "fully engineered design". Inter-active design comprises a flexible procedure including the adaptability to changing/differing conditions (ground or structural parameters, construction sequence or time, etc.). Fully engineered design stands for design and construction within a rather rigid procedure based on "exactly" fixed requirements, "precise" prognoses, etc. It is expected to need no further modification after the detailed design, assuming that the ground parameters and ground-soil interactions locally found on the construction site are identical with design assumptions.

However, geotechnical engineering commonly involves higher risks than other branches of civil engineering. Therefore, inter-active designs have clear advantages over fully engineered designs if the particular prerequisites are taken into consideration. For emergency cases inter-active designs are frequently the only option (e.g. Brandl, 1998).

Buildings in unstable slopes, especially bridges and retaining structures, or embankments in creeping areas, earthquake zones are typical examples where inter-active designs have proved suitable for decades already. Therefore, the paper focuses ont his field of geotechnical engineering, whereby the principles of inter-active design (see chapter 5) are generally valid, i.e. in the entire field of geotechnics: Predominantly, for instance, open face tunnelling, deep excavation pits, land reclaiming, embankments on very soft soil or tailings dams (Fig. 1).

Commonly, technical "risk" is defined as amount of damage multiplied by the probability of damage occurrence. In connection with inter-active design "safety" may be also defined as that situation where the risk does not exceed a certain "borderline risk" (Fig. 2). The relevant borderline is based on multidisciplinary cooperation and depends on local conditions.

2 GENERAL

Within densely populated countries, good ground for new buildings becomes increasingly rare. Nevertheless, the requirements of local and international transportation infrastructure force geotechnical engineers to present solutions which often have to reach the borders of feasibility. Building in unstable area includes a significantly higher calculated risk than is experienced by the other branches of civil engineering. In most cases, sophisticated theoretical models and calculations simply feign an accuracy which in practice does not exist. Statistical investigations, in the end, do not really solve the

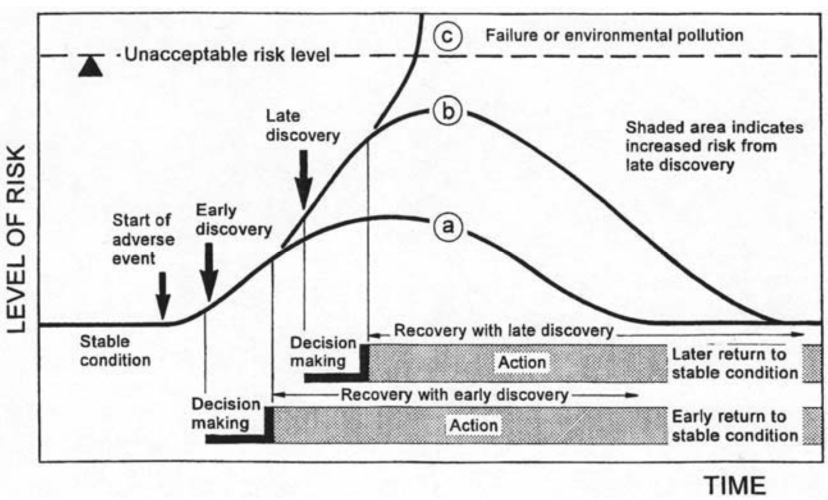

Figure 1. The "discovery-recovery" model for a risk analysis according to the inter-active design (semi-empirical design method with calculated risk) based on the observational method.

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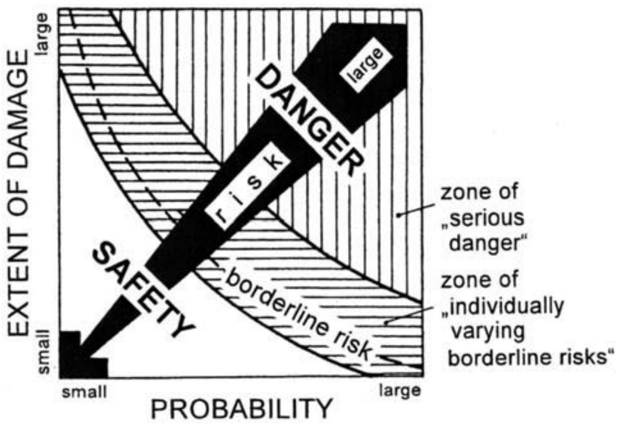

Figure 2. The borderline risk and the zone of individual risk assessment.

problem either. This refers to the ground parameters as well as to the climatic data. But, parametric studies are essential for a reliable risk assessment and to follow the concept of most probable and most unfavourable conditions. This involves design issues which can be adapted during construction or even in the long-term according to the observational method. Unstable terrain requires a „semi-empirical" design method, hence inter-active design, based on comprehensive monitoring - and pre-planned safety measures which allow for future strengthening if the results of long-term measurements require such, hence contingency plans.

3 INFLUENCE OF GROUND PARAMETERS

Most landslides occur in connection water. This may be direct due to joint or pore water pressures, seepage pressures, or indirect by reduction of the shear parameters of the ground.

Figure 3 shows a histogram which clearly indicates the important effect of weather on the number and magnitude of landslides in a certain region: Heavy, long lasting rainfalls in spring 1975 caused numerous, catastrophic slides in some regions of Austria as never experienced during the last 150 years. They were favoured by a preceding very wet autumn and heavy snowfalls during winter which left the ground soaked like a sponge and discontinuities filled with water already before heavy spring rains began.

Between 1999 and 2010 the number of landslides was again less than 50 slides per year. Thus, the question rises if a "safe" design should be based on such singular ground/slope water

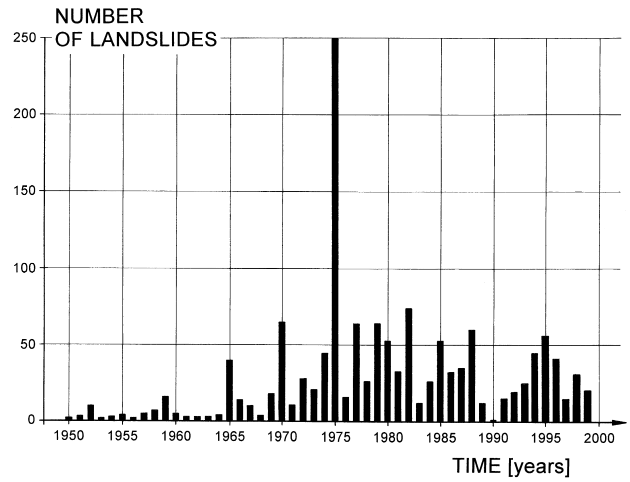

Figure 3. Landslides in Lower Austria between 1950 and 1999. Singular weather conditions in 1975 caused excessive mass movements.

situations as occurred in the example of Fig. 3 in the year 1975. In most cases this would be technologically and economically impossible. If, additionally, the worst soil or rock parameters were selected, most infrastructure arteries in slide prone areas and mountainous regions could be "calculated to death", i.e. theoretical safety factors would drop clearly below F = 1.0.

Such weather conditions cause severe site difficulties if they coincide with construction work. But, on the other hand, just then the weak points and critical zones are clearly recognisable, and the stabilizing measures can be optimally adapted already during the construction period.

Risk assessment and stability analyses of slopes should – above all – always involve the determination not only of the conventional shear parameters, but also of the residual shear strength. Only by taking into account the residual shear strength as well, a serious risk assessment is possible. In the case of sheared rock the test should focus on fine-grained joint fillings; heavily decomposed rock may be treated like wide-grained soil.

The internal friction and the residual shear angle of soil or rock joint fillings depend also on the level of effective normal stress. Therefore, if the normal stress at the beginning of the shear test is too small, the measured value of Φ_r is not the theoretical minimum (Fig. 4). As Φ_r mostly decreases with increasing normal stress, the overburden should be taken into account when assessing the possible residual shear strength in the field. Deep-seated slide planes are more critical than those near to the surface.

An increasing degree of water saturation favours the tendency towards slickensides and decreasing Φ_r (Fig. 4). Therefore, shear or triaxial tests should be performed on saturated specimens to obtain the minimum value of Φ_r for lower border analyses.

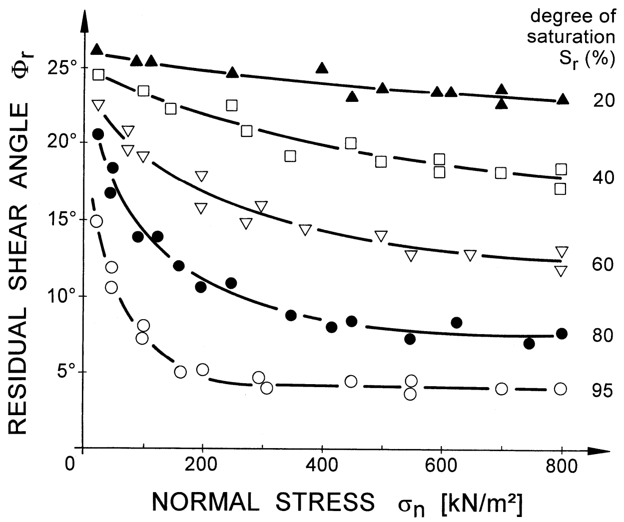

Figure 4. Residual shear angle, Φ_r, versus effective normal stress, σ_n; degree of saturation, S_r, as parameter. Results of direct shear tests with silty-clayey mylonite.

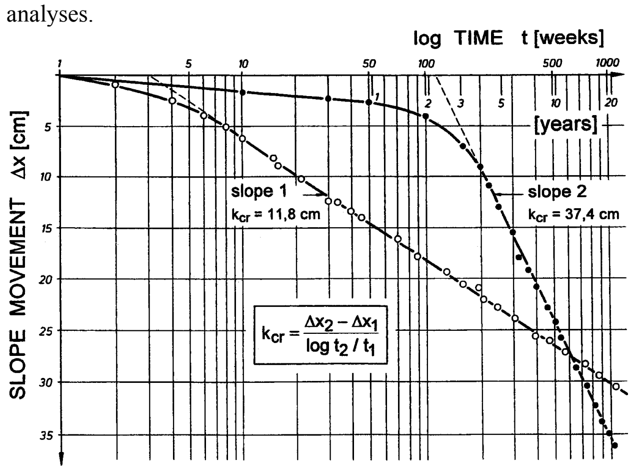

Figure 5. Steady creeping behayiour of two unstable slopes in weathered schists. Definition of creeping factor.

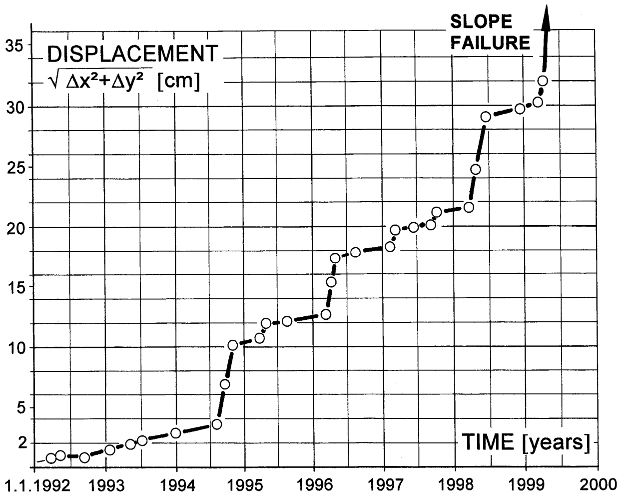

Figure 6. Irregular creeping of an unstable slope with a low residual shear strength. Increasing risk of sudden slope failure with increasing displacement.

An increasing degree of water saturation favours the tendency towards slickensides and decreasing Φ_r (Fig. 4). Therefore, shear or triaxial tests should be performed on saturated specimens to obtain the minimum value of Φ_r for lower border analyses.

Creeping rock or soil slopes close to the limit equilibrium ($F = 1$) and exhibiting a low residual shear strength tend towards progressive failure with a gradual transition from creeping to (sudden) slip failure. The risk of tertiary creep increases with decreasing Φ_r. Long-term monitoring is therefore essential for a reliable risk-assessment and to start stabilizing or retaining measures in time. If sufficient data exist, a creeping factor can be deduced and future extrapolation is possible (Fig. 5).

Sometimes long-term creeping of a slope occurs in rather irregular steps which make a reliable extrapolation and prognosis very difficult. In case of a very low Φ_r, the risk of a sudden slope failure increases significantly with shear deformation (Fig. 6). Moreover, in ground with a high tendency towards progressive failure (low Φ_r), the slip surfaces gradually run deeper and deeper and spread retrogressively. Thus, the sliding mass may increase significantly with time. Therefore, such slopes should be stabilized as early as possible; a delayed stabilization would become increasingly more expensive.

Time may also have significant influence on the shear resistance of soils or rock. Long-term strength deterioration of the ground (especially in flysh, mylonitic schists, over-consolidated clay) leads to a significant reduction of the factor of safety with time, hence causing landslides of slopes which originally were stable.

If the shear strength of the ground starts to decrease towards the residual value, rapid stabilization and retaining measures are essential in order to avoid failure. Fig. 7 shows such a case which is typical for the observational method:

An expressway, designed along a geological fault of a steep slope should rest on a 30m high embankment. After filling only 5m, sliding began which threatened two main railroads of Central Europe (on top and toe of the slope) and a Federal Highway. Rapid soil investigation disclosed locally clayey mylonites with a residual shear angle of only $\Phi_r = 4.5°$. Therefore, the fill was removed quickly, drainage borings and an anchor wall were installed to stabilize the lower part of the slope. The new expressway was then placed on multi-anchored crib walls.

4 STRUCTURES IN UNSTABLE SLOPES

According to the engineering philosophy of the semi-empirical design method with calculated risk ("inter-active design") several highways and railways in the Austrian Alps were constructed within the last 35 years along unstable slopes (also seismic areas) which years before had been considered unsuitable for such alignments: There are highway sections, many kilometres long, where about 75 % of the alignment is running on slope bridges and viaducts. Nevertheless, the visible construction costs, whereas the other 80 % are invisible, i.e. foundations, retaining structures, and prestressed anchors (up to a single length of 120 m).

In mountainous regions, the ground parameters frequently exhibit wide scattering (even within a small area) to such an extent that geotechnical design procedures seem to provide only border values and serve for reference only. The mean design value can only be a „most probable" value and has to be validated by the observational method. Due to the steeply inclined slopes, there is also the problem of seepage flow, and, moreover, seismic aspects have to be considered. The results of assessing slope stability or the calculated lateral pressure on retaining structures are less influenced by the method of calculation than by the assumption of relevant soil/rock properties, seepage flow conditions, and seismic parameters. This is the reason why, generally, sophisticated design methods are by far less informative than parametric studies involving geological variability, groundwater conditions, and specific construction measures.

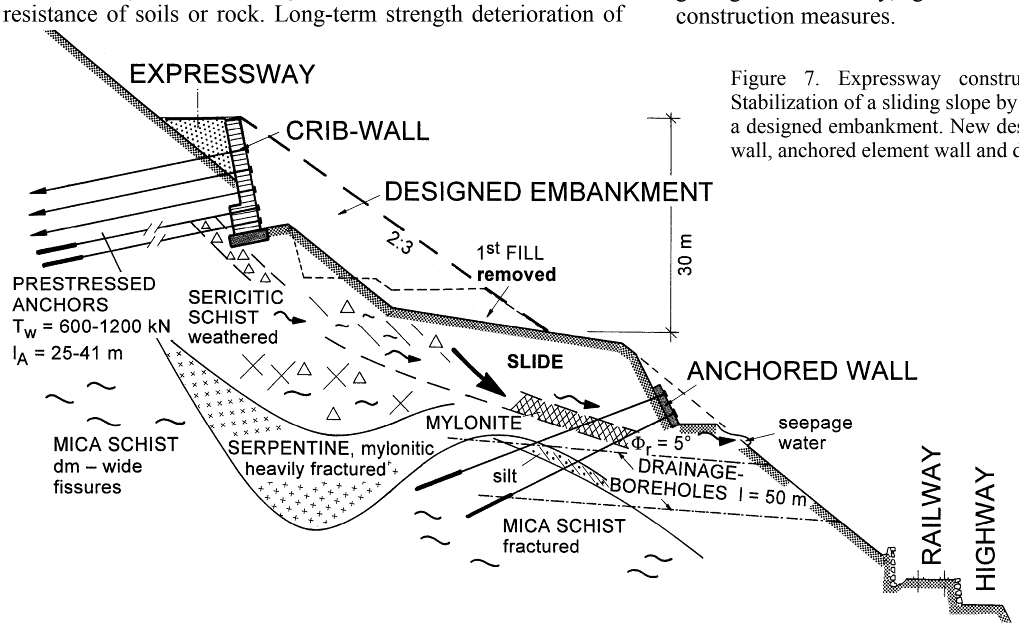

Figure 7. Expressway construction in a geological fault: Stabilization of a sliding slope by rapidly removing the first fill of a designed embankment. New design comprises an anchored crib wall, anchored element wall and drainage borings.

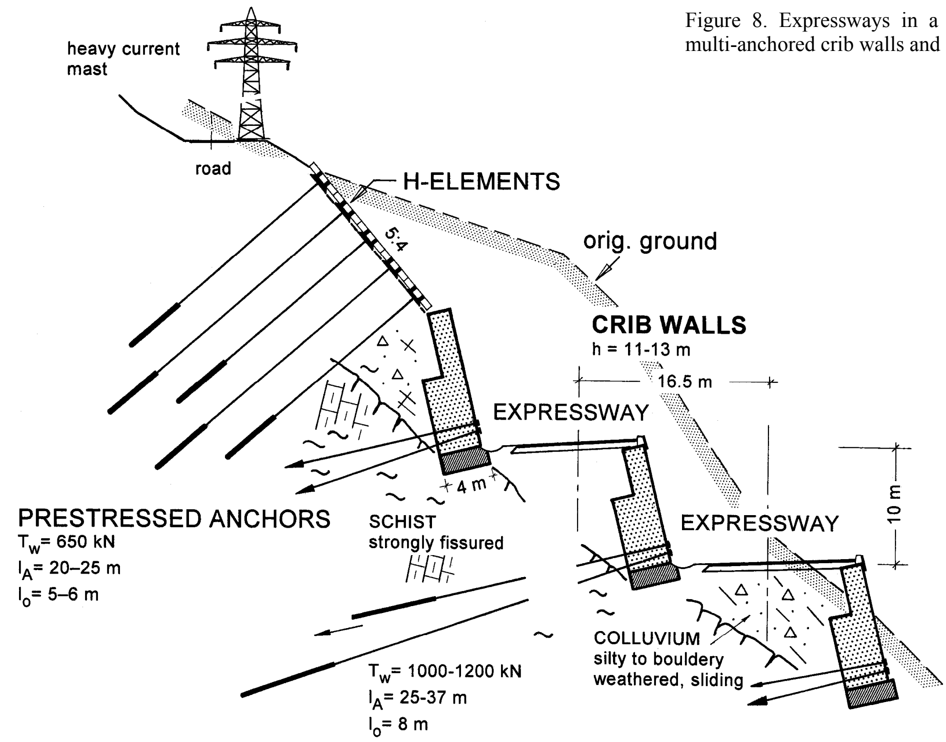

Figure 8. Expressways in a steep unstable slope: Stabilization with multi-anchored crib walls and anchored reinforced concrete elements.

The optimal solution for slide stabilization and retaining structures can frequently be achieved only step by step in connection with taking in-situ measurements. It would be economically unjustifiable to construct most expensive protective structures, by throughout assuming and superposing the most unfavourable parameters. In mountainous regions this would be even technologically impossible.

„Calculated risks" are to be accepted in the design of roads, expressways, and railways through valleys in mountainous areas where slopes with a slide potential extend over a distance of several kilometres. Risk assessment has to distinguish between the possibility of local slides and the stability against general, large-scale failure. In order to reduce construction costs as well as to save time, supplementary measures (mainly anchors) should be considered. This requires detailed contingency plans. Such measures are – even in connection with local remedial works - less costly than an "absolutely safe", fully engineered design which seeks to avoid the possibility of additional measures taken at a later time. Finally, one should bear in mind that an „absolute safety" cannot be provided under such extreme topographical and geotechnical conditions.

In such cases, flexible retaining structures have proved successful. They are adaptable step by step, both technologically as well as economically, to the locally prevailing slope pressures, slope movements, and ground conditions. This practical approach is based on continuous measurements and observations of the retaining structure, the underground and the subsoil/rock surface during the entire construction period (e.g. by geodetic survey, extensometers, and inclinometers, monitoring anchors, earth/rock pressure cells). After completion of construction, subsequent random monitoring is recommended. Calculations and theoretical

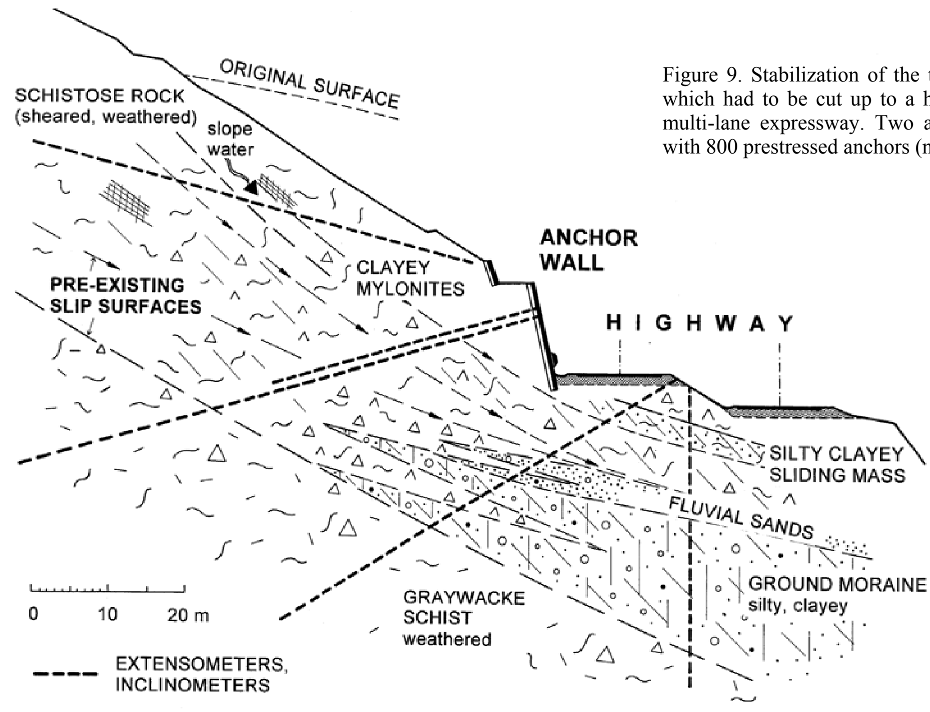

Figure 9. Stabilization of the toe zone of a 800m high unstable slope which had to be cut up to a height of 45 m for the construction of a multi-lane expressway. Two anchored element walls of 250m length with 800 prestressed anchors (not drawn).

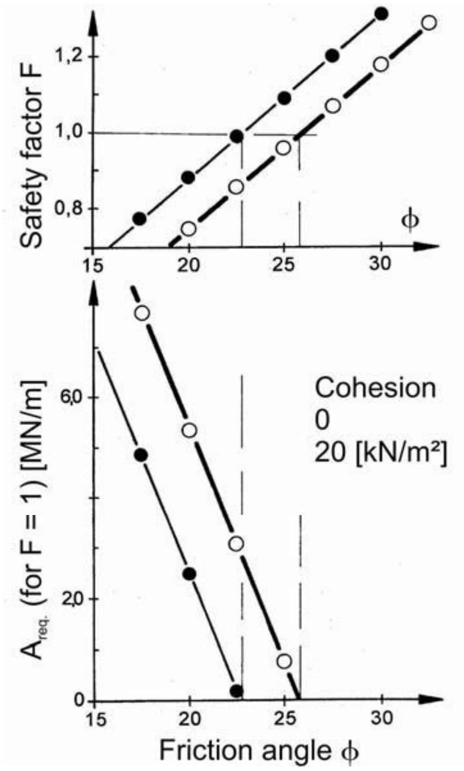

Figure 10. Influence of shear parameters on the safety factor against slope failure, F, and on the required anchor force T per meter run of the structure to achieve F = 1 for the anchored element wall in Figure 11.

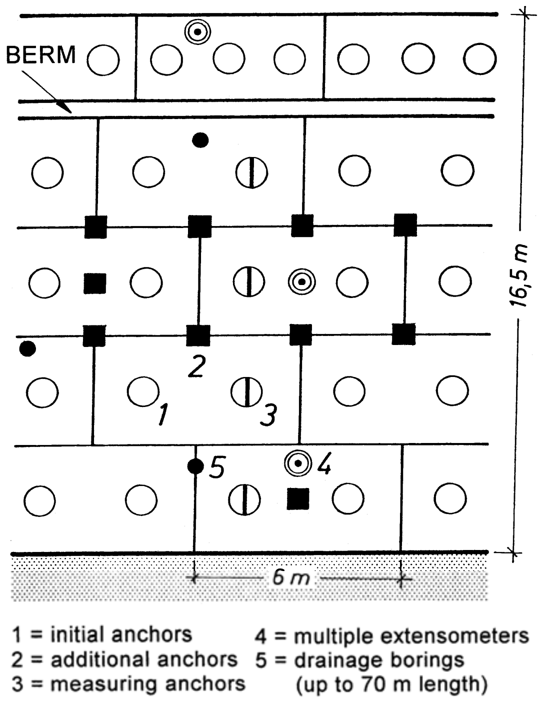

Figure 11. Partial view of Figure 9 illustrating the semi-empirical design and observational method.

considerations are only the basis for the first design and for interpreting the obtained measurement results. This „semi-empirical" design method has proved suitable under most difficult conditions for more than 35 years.

Figure 8 shows an example of flexible retaining structures which were installed to construct expressways in a steep slide-prone slope. The structures can withstand great differential movements and may be easily strengthened at all times. When cutting the slope, a mast of a nearby high voltage line began to move. This critical situation could be overcome by a rapid installation of prestressed anchors connected to a prefabricated reinforced concrete system (H-elements) that acts like a girder grille.

Figure 9 illustrates the use of the semi-empirical design method (inter-active design) for the stabilization of a 800 m high slope. Its toe zone had to be cut along a length of 350m and up to a height of 45m which required two anchored walls. Comprehensive ground investigation disclosed a wide scatter of soil and rock properties. Parametric studies showed that already minimal changes of the shear parameters resulted in significant changes of the required anchor forces (Fig. 10).

In fact, the internal friction varied by about $\Delta\Phi$ = 15°, and moreover, it could decrease to a very low residual value of Φ_r. Therefore, 800 prestressed anchors of l_A = 24 to 70m length were installed, in total $\sum l_A$ = 35000 m. The anchor forces along the 250 m wall varied between $\sum T_w$ = 2450 to 3700 kN/m, depending on the measurement results during construction (Fig. 11). This involved the installation of several additional and longer anchors in some sections due to extremely unfavourable weather conditions which re-activated pre-existing slip surfaces in the ground and caused a local drop of the shear strength. (The most critical construction phase was just in spring 1975 – Fig. 3). Therefore a "contingency project" was designed for the worst case that the installed anchor forces might not be sufficient in the long-term (Fig. 12). Until now, that is already 38 years after construction, no pile or other strengthening measure was necessary as monitoring has proved. This example underlines most impressively the advantage of the inter-active design or semi-empirical design respectively over the fully engineered design method – especially if there is a great difference between "Most Probable" (MP) and "Most Unfavourable" (MU) conditions.

Dowelling of unstable slopes has proved suitable if larger displacements which are necessary to activate sufficient ground resistance are allowable. If large diameter bored piles do not exhibit sufficient resisting moment, socket or caisson walls should be taken into consideration. They are installed by sinking shafts and filling them with reinforced concrete, and they have proved suitable as visible or completely sub-surface structures (Fig. 13).

5 BASIC PRINCIPES OF INTER-ACTIVE DESIGN

"Inter-active design" or "semi-empirical design with calculated risk" (Brandl, 1979) is based on the observational method and on contingency plans to quickly allow strengthening/recovery measures if results of monitoring require that. The basic principles of this methodology may be summarized as follows:

- Detailed ground investigations.
- Proper laboratory/field tests.
- Geotechnical calculations with parametric studies (incl. worst case scenarios).
- Plausible design assumptions and detailed design of contingency plans (possibilities of quick strengthening or stabilizing measures).
- Geotechnical prognoses.
- Experienced site supervision and comparison of design parameters with the ground parameters that were found in-situ during construction work.
- Monitoring (with early zero readings) and experienced interpretation of the data.
- Over-determination of monitoring data by independent, different measuring methods (e.g. deformation and forces/stresses) allows a clearly better interpretation than relying only on one system.
- Back-calculations.

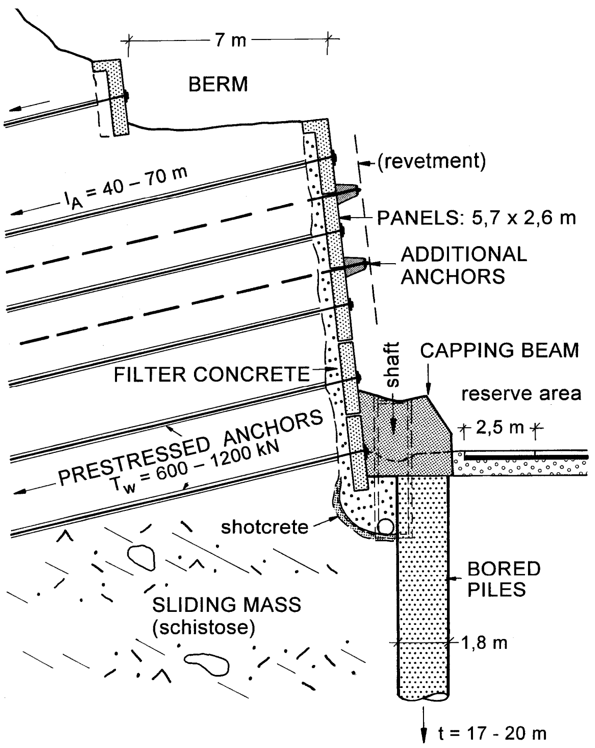

Figure 12. Detail to Figure 11 with additional anchors which were necessary during the construction period according to the observational method.
Contingency plan: Large diameter bored piles with capping beam for a possible next step of strengthening the retaining structure if long-term monitoring would require this.

- If necessary: iterative additional calculations based on available measurements.
- If necessary: (ad hoc) adaptation/strengthening of the project during or after the construction period.

There is always a multiple interaction between ground, slope stabilization measures, foundations, basement walls, hence sub-surface elements and the structure above surface. Therefore, inter-active design should involve also the possibility to react at the (super-)structure, i.e. re-adjustable bridge bearings in unstable slopes or weak ground, compensation measures for buildings close to underground excavations, etc. Geotechnical engineering and structural engineering should be considered as a closely interacting unit.

To sum up, the main advantages of inter-active design are:

- Detailed proof of ground – structure interaction, hence the safety factors.
- Data collection may be used for research, improvement of geotechnical/numerical models, calculation methods, and systems of measurement, etc.
- Optimization of construction methods, construction phases and maintenance.
- Cost saving.
- Frequently reduction of construction time.
- Pre-warning in case of locally unforeseen or changing ground properties.
- Possibility to compare the design assumptions with the measurement results.
- Training of the geotechnical way of thinking of involved persons.
- Important tool for engineering judgement.

6 FINAL REMARKS

In many cases of ground engineering under difficult conditions the “philosophy” of inter-active design provides the only technical solution – not to mention the cost savings. A “fully engineered” design, i.e. a design that requires no further modification following detailed design is hardly possible. The potential to make modifications during construction and to strengthen the structure at any time, also after construction, is a fundamental requirement of the inter-active design or the semi-empirical design method with calculated risk based on the observational method. It involves the concepts of the most probable and most unfavourable conditions, hence a creative process and not over-complication, but “high-quality simplicity”: High-quality simplicity does not forget the reasoning behind “simple” practices, because over-simplification, sometimes through so-called high-tech mechanistic calculations, can cloud one’s engineering judgement.

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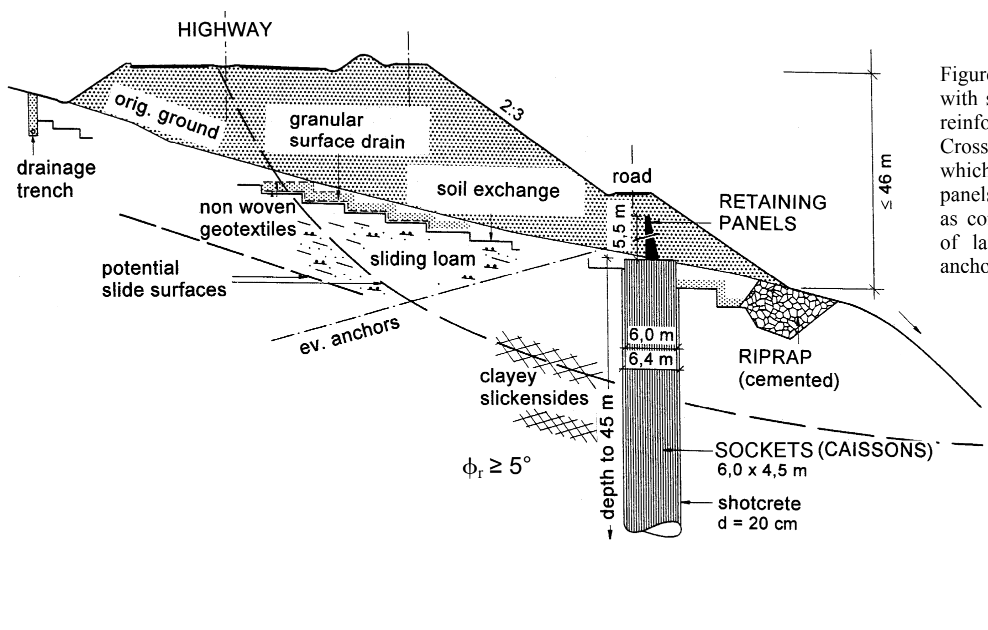

Figure 13. Highway in unstable slope with slope dowelling and geosynthetic-reinforced embankment.
Cross section through a socket wall which exhibits reinforced concrete panels on top of the sockets (caissons) as contingency plan for the possibility of later tying back with prestressed anchors.