Design of slope retaining structures based on the observational method

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SYNOPSIS

Slope stabilization and construction activities in slide-prone slopes commonly include a higher calculated risk than other branches of civil engineering. In most practical cases it is hardly possible (because too expensive) to design with high safety factors. It would be economically unjustifiable to construct most expensive protective structures by assuming throughout the most unfavourable parameters (e.g. along kilometres of roads). Consequently, sensitive and important buildings (e.g. appartment blocks, bridges) structurally should be separated from slope retaining measures, and both should be checked regularly by long-term monitoring.

According to uncertainties which result from locally scattering ground properties, from changing water pressures etc., the semi-empirical design method has proved most successful from geotechnical and economical point of view. This "philosophy" is based on the observational method in connection with a careful monitoring before, during and after construction. It takes into consideration the eventuality of local (small) failures and the possibility (or requirement resp.) of several steps of stabilizing measures - thus providing a very cost-effective solution.

1. RISK ASSESSMENT

In montainous regions soil and rock parameters frequently exhibit scatter, even within a small area, to such an extent that geotechnical design procedures seem to provide not more than limit values and serve for reference only. Due to the steeply inclined slopes, there is also the uncertainty of the seepage flow. Therefore the safety of such slopes cannot be proved in the usual manner only by theoretical methods. The results of evaluating lateral pressures and slope stability are less influenced by the method of calculation than by the assumptions of soil or rock properties and seepage flow conditions. Consequently, a sophistication of design methods is not warranted, but parametric studies with certain boundaries are very important. The optimal solution for retaining measures can frequently be achieved only step by step in connection with in situ measurements. It would be economically unjustifiable to construct most expensive protective structures, whilst throughout assuming the most unfavourable parameters.

"Calculated risks" are to be accepted in the design of roads and expressways through valleys in mountaineous areas where hillsides with a slide potential extend over a distance of several kilometres. In order to reduce construction costs as well as to save time, the possibility of supplementary measures (mainly anchors) should be considered; this is, however, still less costly than the "absolutely safe" structure 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 should be preferred, adaptable step by step, both technologically as well as economically, to the locally prevailing soil/rock pressures, slope movements and subsoil conditions. This practical approach is based on continuous measurements and observations of the retaining structure, the ground surface and the subsoil (e.g. with extensometers and inclinometers) during the entire construction period. Subsequent random monitoring is recommended. Calculations and theoretical considerations are only the basis for the first design and for interpreting the measuring results. This "semi-empirical" design method has proved successful under most difficult conditions - also in seismic areas - for more than 20 years (Brandl 1979). It can be also considered an observational method as already proposed in principle by K.Terzaghi.

2. SHEAR PARAMETERS

The main problems of risk assessment of slopes are the scatter of soil or rock parameters, and the locally and temporarily varying effects of seepage water.

Shear strength is the most important ground parameter used in stability analyses, which are necessary for risk assessment and/or design of restraining structures. The investigation of small soil or rock samples in the laboratory provides only approximate information; usually small specimens represent stronger rock material than the rock mass in situ. On the other hand, the exploration costs increase with the size, that is, the diameter of the sample. In situ tests are economically justifiable only in case of large and important structures, and the application of local results to a complex rock mass may include certain risks. Therefore a sufficient number of laboratory specimens should be investigated, focusing on the effect of planes or zones of weakness in the rock or subsoil. In case of heterogeneously layered texture, the shear strength depends essentially on the shear orientation. Rock material, gouge or mylonitic zones as well as many (overconsolidated) clays show a typical post-failure behaviour which should be taken into account when designig retaining structures.

Weathered and decomposed products on the joint surfaces represent materials which frequently develop slickensides with very low angles of internal friction, forming potential failure lines.

For such cases extensive laboratory shear tests on specimens from the weak zones should be performed. In slopes prone to slippage, or with a high clay mineral content (especially montmorillonite) in bedding, joints or faults etc. small movements may initiate a gradual decrease of the angle of internal friction. Limit values of residual shear strength have been measured down to $\phi_r = 4°$.

Though the knowledge of mineral composition provides some hints on the slippage-proneness of slopes, direct testing in the triaxial apparatus or shear box is still the most reliable method to determine shear parameters. These tests allow the investigation of a great number of samples, thus enabling a statistical analysis to form the basis of the final design parameter assumption.

The shear strength of filled joints is not only influenced by the properties of the fillings but also by their thickness. Clay seams of even a fraction of 1 mm reduce the strength significantly. Therefore shear

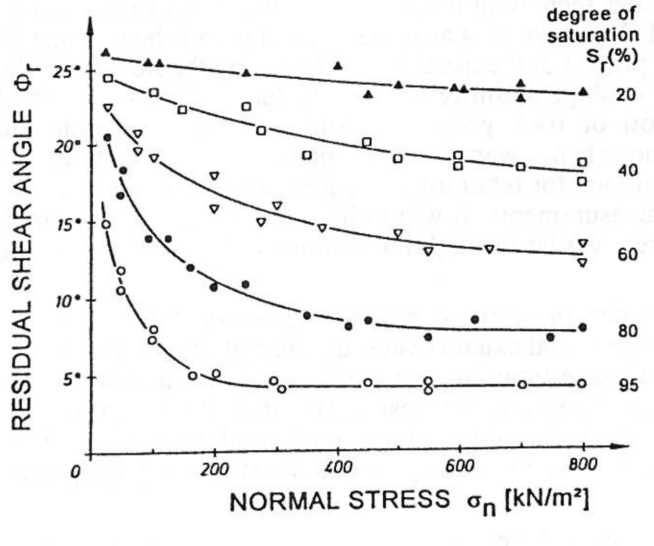


Fig. 1
Residual shear angle, ϕ_r, versus normal stress, σ_n; degree of saturation, S_r, as parameter.
Results of direct shear tests with silty clay

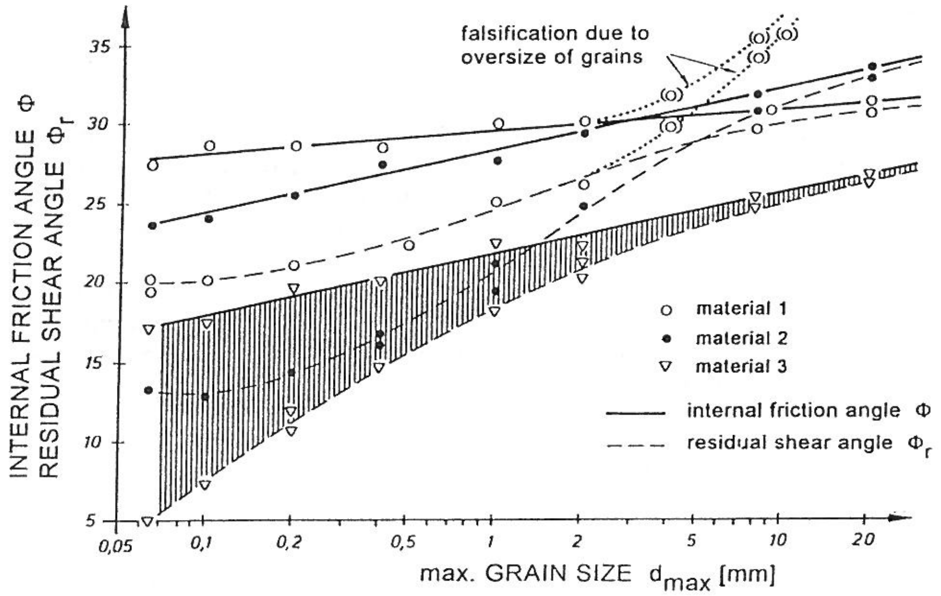


Fig. 2
Angle of internal friction, ϕ, and residual shear angle, ϕ_r, versus maximum grains size, d_{max}.
Three examples of test series on different soils.

parameters of the fill material should commonly be considered as decisive in designing practice - independently of the actual thickness of a fault.

When determining the residual shear strength in laboratory or field tests, attention should be paid to the following facts:

- The shear strain should be increased up to the limit value of ϕ_r. This can be performed more easily by the circular ring-shear apparatus or in shear boxes (e.g. to and fro shear directions) than by the triaxial apparatus.
- If the normal stress at the beginning of the shear test is too small, the measured value of ϕ_r is not the theoretical limit value (Fig. 1). 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-lying slide planes are more critical than those near to the surface.
- A decreasing degree of saturation reduces the tendency towards slickensides and decreasing ϕ_r (Fig.1).
- The size of the shear box or triaxial apparatus should be adapted to the maximum grain size, d_{max}, of the soil or of the joint filling, mylonitic material or decomposed rock - and vice versa respectively. Numerous shear test series have shown that commonly there exists a linear correlation between the logarithm of d_{max} and the internal friction ϕ (Fig. 2). An extrapolation provides safe values if the grain size distribution has no intermittent shape. The maximum grain size should not exceed 1/50 of the shear box side length and 1/10 of the box height. Otherwise too large values of ϕ and ϕ_r are measured. Consequently, an oversize of grains must be separated from the sample before the shear test is performed.That means that the material has to be prepared properly (remoulded); commonly, undisturbed samples cannot be used directly or serve for comparison only.

  The correlation between log d_{max} and ϕ_r is not linear as in case of ϕ, because ϕ_r results from the external friction along the shear plane and not from the internal friction between particles (e.g. Fig. 2).

In most cases of engineering practice it would be too expensive or even impossible to design retaining structures on the basis of the residual shear strength of the soil or rock. But on the other hand it cannot be justifiable to use the peak value, ϕ, if ϕ_r is extremely small. Thus each slope supporting or stabilizing measure must be considered unique, and especially in mountainous regions generally valid rules for design can hardly be given.

Laboratory tests and practical experience show that cohesion, c, and angle of internal friction, ϕ, are not always constant but may alter (with time, due to creeping, caused by external influences, etc.). Frequently cohesion must be considered an unreliable parameter and therefore it should be taken into account only cautiously when evaluating slope stability or when calculating the forces acting on retaining walls.

The knowledge of the residual shear strength is also of importance when assessing the urgency of supporting measures. If a slope starts moving critically (and also the retaining structure), stabilizing works might become necessary immediately in order to avoid a progressive decrease of shear strength and a consequent overall failure (Brandl 1979).

3. DESIGN ASPECTS FOR SLOPE SUPPORT

3.1 General

The semi-empirical design for slope stabilization, retaining structures or for buildings in unstable slopes is based on the observational method. This "philosophy" requires the possibility of subsequent strenghtening measures if monitoring indicates a gradual approach to a critical limit stage (e.g. increasing lateral pressures or slope movements).

Furthermore, sensitive or important buildings should be protected hillside by retaining structures which have to act as a first barrier against excessive slope pressures (= "primary" retaining system). Retaining measures and sensitive buildings statically should be separated. Moreover the latter should exhibit a foundation with a high resisting moment and - in especially critical cases - sufficient strengthening possibilites (spaces for anchoring etc.). In total, of course, all precautionary measures against slide damage are considered a geotechnical unit (referring to overal slope stability etc.).

Slide-protective retaining structures should be designed as flexible systems and in such a way, that additional (precautious) retaining measures could be performed at any time. Thus, excessive sliding pressures are taken over by the primary retaining structure without threatening the sensitive building itself (e.g. slope bridges, television towers, hotels, appartment blocks, etc.) The scheme of this design philosophy is illustrated in Fig. 3 for bridges or masts in slide-prone slopes. In case of wider buildings the retaining structure will be straight or polygonal in the ground plan.
The optimum solution and the most suitable time of supplemental measures for supporting unstable slopes or strengthening retaining structures can be fixed only in connection with the observational method.

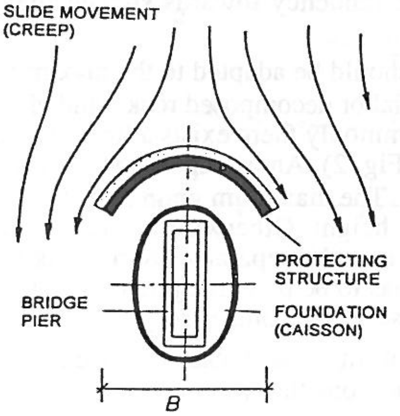


Fig. 3
Retaining structure to protect a structurally sensitive building (e.g. bridge pier) from sliding or creeping pressures in an unstable slope - schematical

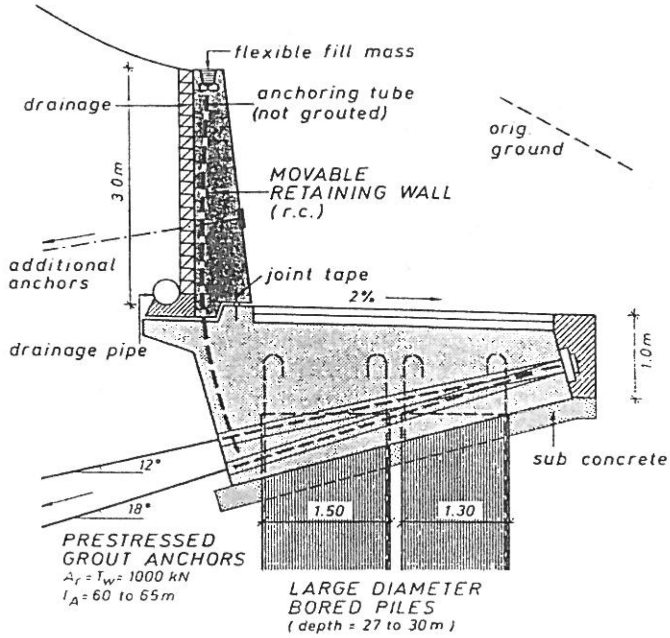


Fig. 4
Top of a twin-pile wall in an ustable slope (deep seated failure planes and creep zone near the surface); heavily weathered and decomposed schists with high clay content. Capping beam of the pile wall with movable reinforced concrete wall for lateral pressure reduction; precautionary features include the installation of additional prestressed anchors. Capping beam used as permanent access way for slope monitoring.

Fig.4 illustrates the structural details of a retaining structure in a slope with deep reaching multiple slide surfaces and a creeping zone within the topmost 2 to 4 metres. The subsoil consists of silty to clayey schistous mylonites with a very low residual shear strength.
The head of the anchored pile wall consists of twin piles with 2.8 m spacing and a continuous capping beam in the longitudinal direction. Upon this beam another retaining wall is placed, being connected with the capping beam by a hinged bearing and making sufficient allowance for the possible need for additional anchors. The flexible top wall avoids the development of full creep pressure. This measure has proved successful for more than 20 years as confirmed by long-term monitoring.

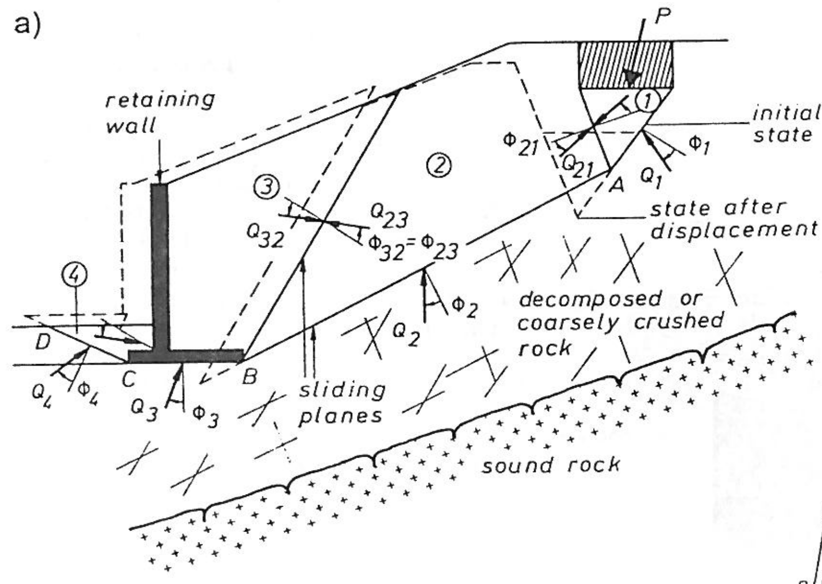
The following sub-chapters 3.2 to 3.4 illustrate the uncertainties in theory and practice when designing and executing slope supporting measures, thus emphasizing the importance of the observational method.

3.2 Retaining walls

The advantage of the observational method or semi-empirical design respectively becomes evident not only in case of scattering soil or rock parameters and groundwater conditions. Differences in theoretical assumptions, possible failure modes etc. may also lead to widely diverging results of geotechnical calculations. This can be easily demonstrated when designing a "simple" retaining wall on toe of a slope:

The classical calculation uses the Coulomb earth pressure theory. But if the slope exhibits a tendencvy to creep or to a gradual loss of internal friction, a creeping pressure should be taken into account which may exceed the active pressure by far (Brandl, 1987, 1992, 1993). Consequently, long term monitoring and the possiblity of strengthening measures is of greatest importance in such cases.

If (quasi) monolithic blocks of the subsoil or rock undergo a translation, a kinimatic calculation method should be preferred as indicated in Fig. 5. The "rigid" soil or rock bodies themselves may consist of parted blocks which move against each other along inner slide planes. In such cases, the first calculation step is to assume a kinematically possible failure mechanism, and to determine the relative and absolute dislacements between the single soil or rock bodies by means of a displacement plan. The lateral pressure on the retaining wall is conventionally derived from the ajacent sliding body (3 in Figure 5). The resistant body in front of the wall toe (4 in Fig. 5) should be neglected for safety reasons.



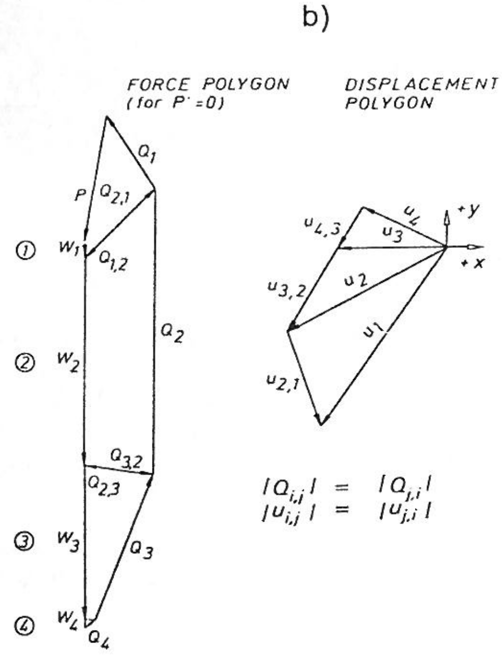


Fig. 5
Kinematic calculation method of conventional retaining walls sustaining rigid rock or soil bodies which move as (quasi) monolithic blocks:

(a) System and notations: P, external load; Q_i, resultant reaction force against failure planes; ϕ_i, angle of internal friction along the failure planes; W_i, weight of sliding blocks
(b) Force polygon (for P' = 0) and displacement polygon: u_{ij} relative displacement between adjacent sliding blocks i and j

3.3 Slope dowelling

Dowelling is another method of slope stabilization which has proved very successful in parctice but should be always combined with the observational method, as the commonly used theories lead to fairly scattering results.

"Dowels" may be micropiles or remaining grout pipes on the one hand, up to large-diameter bored piles or (elliptic) piers with diameters of several metres on the other. In weathered, decomposed or jointed rock micropiles and piers have proved especially suitable. The first may be installed even under extremely steep surface conditions, and piers provide a very great shear resistance. The dowels are placed in a regular or irregular pattern, as single elements or as continuous walls, with or without capping beams etc. - depending on the local requirements. Bored piles of a standard diameter (0.6 m to 1.5 m) are commonly used to stabilize creeping slopes or intensively weathered rock.

The bearing-deformation characteristics of dowels largely depend on their slenderness (Fig.6): Microdowels withstand shear movements only along a short section (about 1 m or less), and the upper part of the dowel does not take over substantial forces. Large-diameter dowels, however, overturn. In creeping slopes, the economically optimal pile diameter is about one-tenth of the slide plane depth

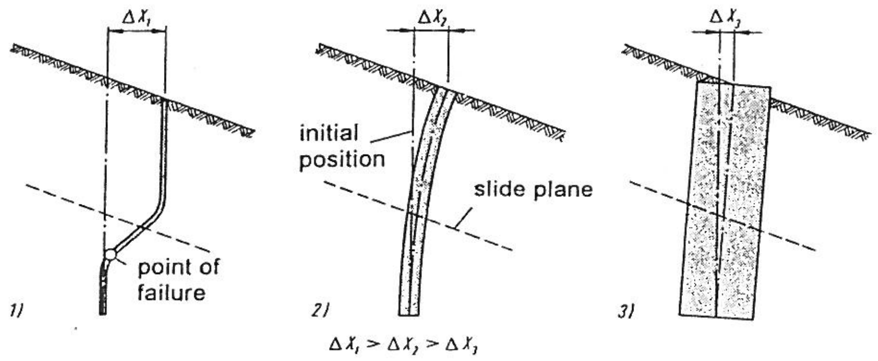


Fig. 6
Deformation of "dowels" in unstable slopes - schematical.

1 = microdowels ($d \leq 20$ cm),
2 = "commonly used" dowels
3 = macrodowels ($d > 1,5$ m or elliptical caissons, piers resp.)

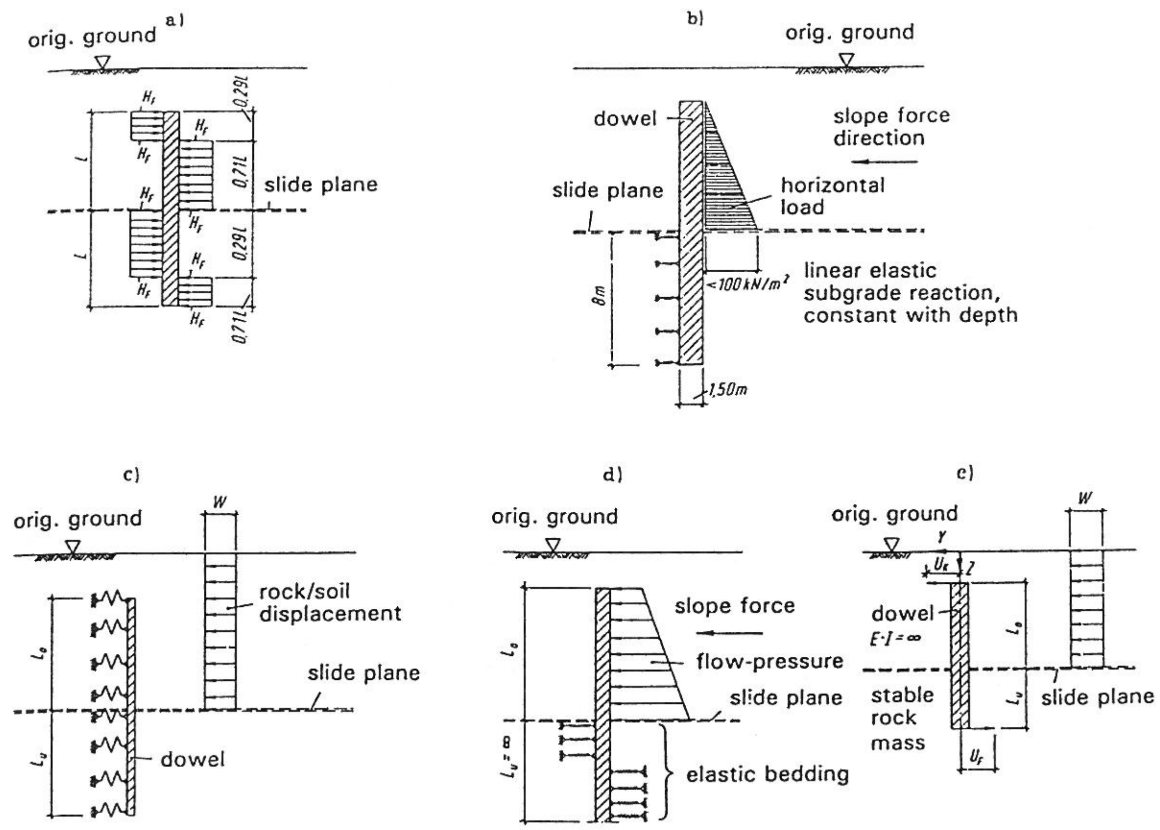


Fig. 7
Siveral hypotheses for dowelling a slide plane (Brandl 1992): (a) Rigid dowels, loaded by a creeping or stagnation pressure; (b) Conventional method with lateral "earth" pressure and resistance according to the subgrade reaction model; (c) Head deformation and maximum internal forces of the dowel derived from the differential equation for the flexural (bending) member; (d) Differential equation of the flexural (bending) member for dowels of a limited length, block sliding and variable moduli of subgrade reaction; (e) After Sommer and Buczek (1987): Rigid dowels, block sliding, hyperbolic load distribution (see Figure 8)

beneath the surface. This rough estimation assumes a failure plane fairly parallel to the surface and facilitates a first design. If only small deformations are allowable, the diameter and the embedding length of the dowels should be increased.

For piles with a diameter of d = 0.6-1.5 m, placed in a regular pattern, it has proved successful to connect them by capping beams. Compression beams arranged like trusses provide a fairly uniform load transfer from the unstable slope to the retaining structure.

The design loads on dowels depend on various factors which partially influence each other and include

- subsoil characteristics;
- slope stability (including possible seepage pressures);
- geometry and rate of movement of the sliding mass;
- relative movements between dowel and subsoil;
- diameter and length of the dowels;
- ratio of stiffness between dowel and subsoil;
- allowable risks and "residual movements" of a creeping slope after dowelling.

An overview of some calculation methods is given in Figs.7,8. Due to the different theoretical assumptions, the results may vary within wide range. Consequently the design should comprise at least two hypotheses to check possible boundaries. Parametric studies with different soil or rock parameters and static systems are also recommended, especially with regard to the group effect of the dowels. This group effect is influenced by several factors:

- dimensions of the dowel group;
- pile pacing;
- support of pile head and toe (free, hinged, fixed);
- stiffness and skin friction of the piles;
- driving and resisting soil or rock forces;
- rock parameters and discontinuities.

Local failures in the dowelling group may initiate a gradual instability of the whole retaining system similar to a long-term zip effect. Such progressive failures are known only from micropiles or grouted pipes in connection with a small residual strength of the soil or rock fault. In such cases the safety factor preferably should be increased by using larger pile diameters rather than increasing the dowel number.

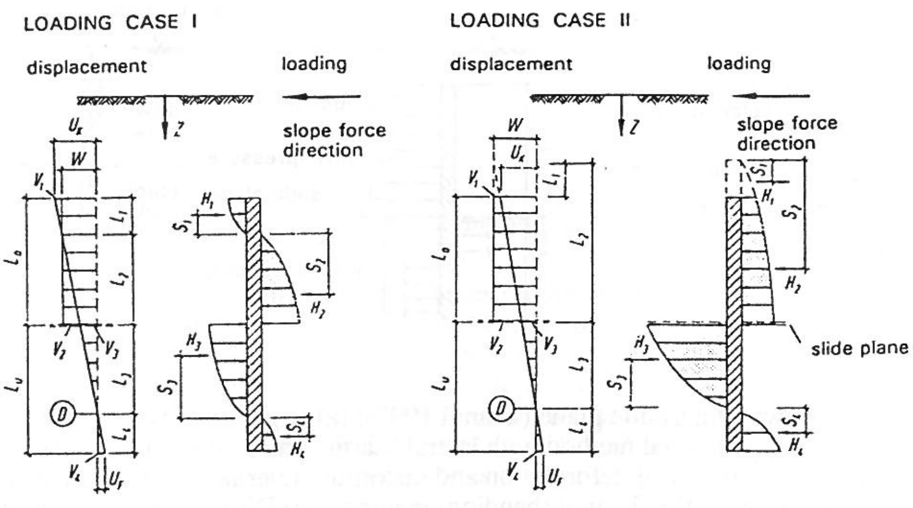


Fig. 8
Calculation of dowels - detail to Figure 7. Dowel movements (tilting), rock/soil movement (block sliding), forces on dowel and resultants; D, rotary point. Loading case I: dowel with fixed head and toe; Loading case II: dowel only with fixed toe (embedded in stable ground)

3.4 Prestressed anchors

Prestressed anchors have proved especially suitable for semi-empirical designing and the observational method. They easily can be installed as precautionary or additional elements to increase slope stability. Commonly, long prestressed anchors ($l_A \geq 15$ m) are superior to soil or rock nailing with short reinforcing units. But combinations of short nails and long anchors are thouroughly possible (Brandl 1987, 1992).

In Austria, about 2000 km of prestressed anchors with a length of l_A = 15 m to 120 m and a working load of T_W = 0,3 MN to 2 MN have been installed during the past 25 years. This great quantity was necessary to stabilize numerous slopes in mountainous regions. In many cases prestressed anchors were used in combination with other retaining measures.

Permanent anchors require a proper corrosion protection and should not be fixed over the total length (l_A). An extensible "free length" (l_A - l_o) enables possible stress rearrangements without critical constraints. The bond length (l_o) commonly varies between 6 to 12 m; longer l_o-values hardly increase the bearing capacity of prestressed anchors because the tensile forces are concentrated rather at the beginning of the fixed length.

In fine-grained soils and in strongly decomposed rock, especially in mylonitic zones with a silty-clayey grain size distribution such anchor systems should be used which enable a post-grouting of the bond length. Furthermore the tendon installation and primary grouting should be performed as quickly as possible after drilling and cleaning the anchor-borehole. Otherwise the stress-strain-behaviour of the anchors worsens significantly.

The ground-related bearing capacity of prestressed ground anchors cannot be calculated simply from soil or rock mechanics theory. Figs. 9,10 show the great scatter of results even for clearly defined homogeneous soils. In practice, the load-deformation behaviour depends on numerous parameters

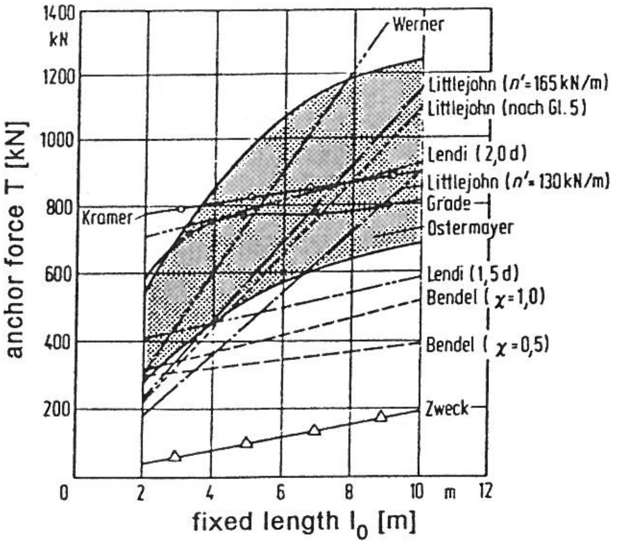


Fig. 9
Comparison of the theoretical bearing capacity of prestressed anchors (anchor force T_{max}), derived from different calculation methods.
Scattered area: measurements.
Non-cohesive soil: uniform sand with 15 % gravel, γ = 20 kN/m³, ϕ = 35°, D = 0,80

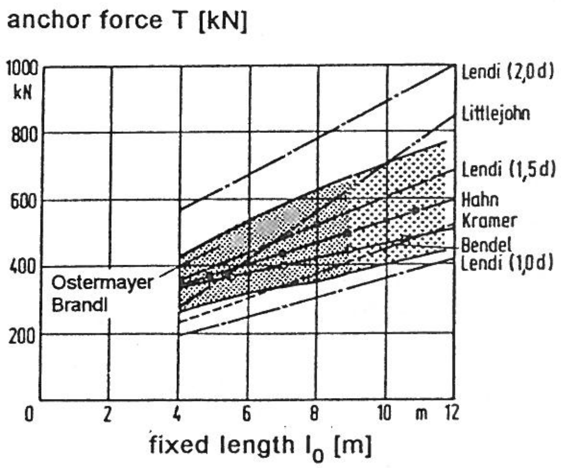


Fig. 10
Similar to Fig. 9, but cohesive soil with 90 % grains finer < 0,06 mm (about 40 % clay < 0,002 mm). γ = 20 kN/m³, ϕ_{calc} = 25 °, c_{calc} = 25 kN/m², c_u = 500 kN/m², I_c = 1,0

which influence each other and can hardly be quantified (e.g. ground properties, dilatation along the bond anchor length, details of anchor system, grouting pressure, etc.). Moreover, the way of installation, the time needed for installation and grouting and several other construction details play an important role. The author experienced sites where adjacent test anchors with the same dimensions in the same ground exhibited a scatter of the allowable working load between T_W = 200 to 1500 kN.

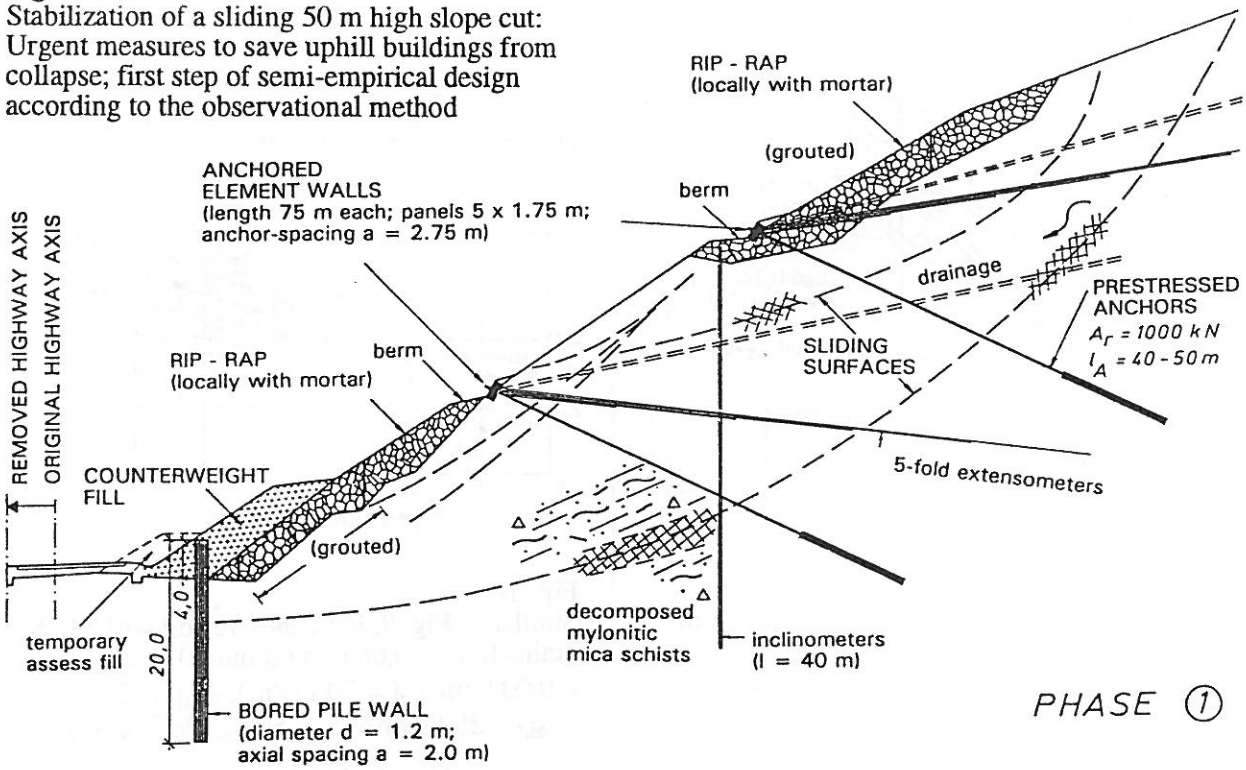
Consequently, the bearing capacity of prestressed anchors can be assessed only by experience in the design stage. Therefore in situ suitability tests are unavoidable before commencement of the works. Acceptance tests have to be performed with each anchor during the construction period, and monitoring is highly recommended to check the (long-term) behaviour of critical slopes and retaining structures.

4. CASE HISTORY

Fig. 11 and 12 illustrate that the optimal stabilization of a sliding slope generally comprises several measures. Moreover, they demonstrate substantial cost savings which result from semi-empirical designing in connection with the observational method. On the other hand, a relatively high calculated risk had to be taken over, because important buildings stood near the slope crest.

A deep slope cut had to be excavated for a highway construction in a slide prone area. Shortly before finishing the earthworks the slope began to move over a length of 150 m. The slides extended progressivly, forming multiple failure zones with deep cracks in the surface. As urgent measures drainage borings were drilled from the berms which already had been provided precautionarily. The open cracks in the surface were filled with quick lime and then covered after the first rainfall. The penetrating line led to a local increase of the internal friction along the slide surfaces. Simultaneously, grouted rip-raps were installed step by step on the berm and on the lower part of the slope cut.

Fig. 11
Stabilization of a sliding 50 m high slope cut: Urgent measures to save uphill buildings from collapse; first step of semi-empirical design according to the observational method



Moreover, the toe of the slope was stabilized by a counterweight fill, which required a removing of the highway axis off the slope by some metres.

The crest of the slope cut was also stabilized by rip-raps (locally grouted), in order to prevent a retrogressive hillside extension of slides. The stability against deep reaching slope failure was achieved by installing prestressed anchors along the berms, and large diameter bored piles on toe of the slope cut. These measures are shown in Fig. 11. Their efficiency was controlled by extensometers, inclinometers, anchor force checking and by geodetic survey.

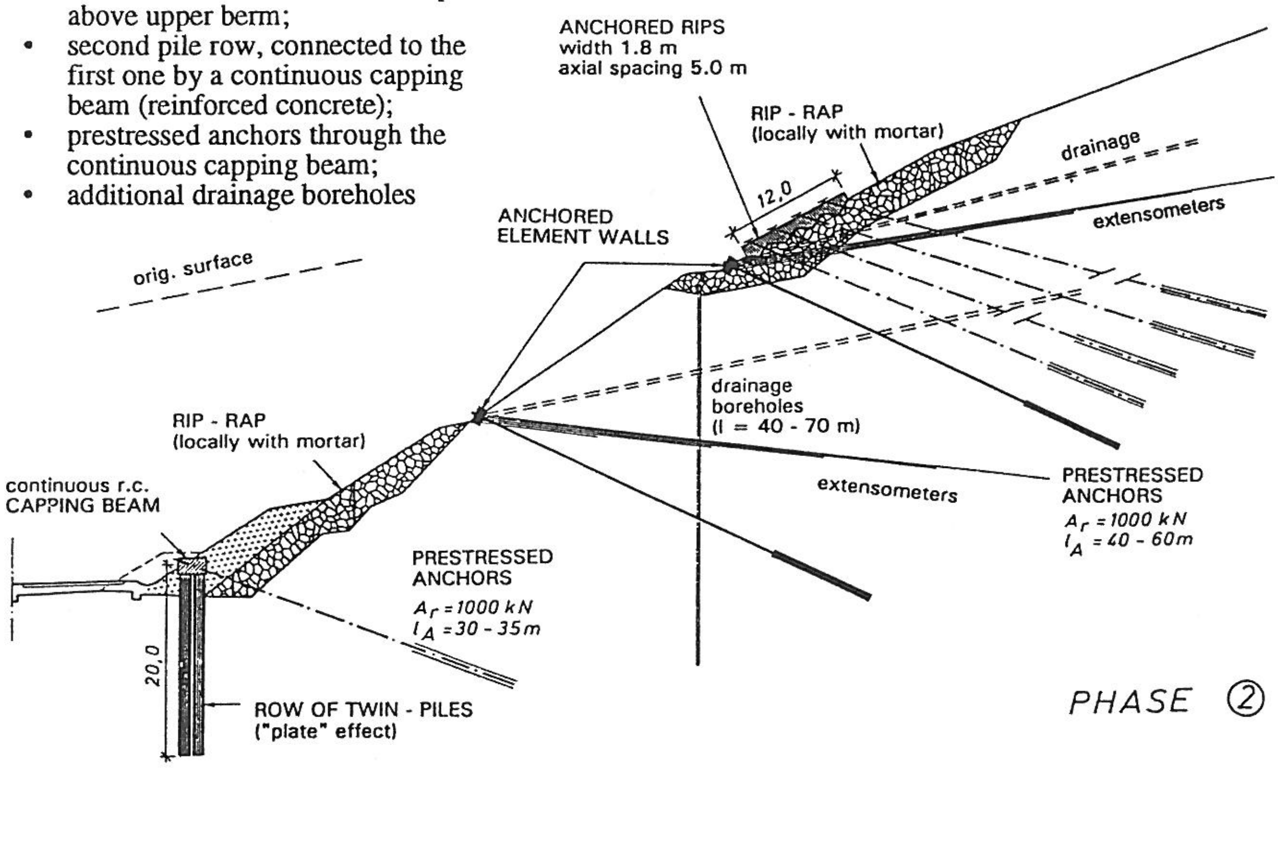
The ground consisted of decomposed schistous rock which actually behaved more like a "soil". Especially within mylonitic zones (with silty-clayey grain size distribution) the residual shear strength could decrease significantly (locally to about $\phi_r = 8°$). Therefore a second stabilizing project was designed as a precaution, if the measures according to Fig.11 turned out to be not sufficient. According to the philosophy of semi-empirical design or observational method respectively, several steps of stabilization methods had to be taken into consideration.

If the angle of internal friction, ϕ, had decreased progressively to the minimum residual value, ϕ_r, the stabilizing measures of the first step (Fig.11) would have been insufficient. The strengthening measures of the second step are illustrated in Fig.12: Installation of reinforced concrete rips with prestressed anchors from the upper berm, and a second pile row on toe of the slope cut. The second pile row would be connected with the first one by tangential piling and a continuous capping beam of reinforced concrete. This structure exhibits a great resisting moment and additionally would be tied back with prestressed anchors.

Long-term monitoring confirmed that the first step of stabilizing measures has been sufficient. In case of need, the second step could be executed at any time without disturbing the highway traffic at the toe of the slope cut.

Fig. 12
Second step or further steps respectively to Fig. 11.
Precautionary design of supplemental measures in case of increasing slope pressures.

- anchored reinforced concrete rips above upper berm;
- second pile row, connected to the first one by a continuous capping beam (reinforced concrete);
- prestressed anchors through the continuous capping beam;
- additional drainage boreholes



5. CONCLUSIONS

The risk assessment of unstable slopes and stabilizing or restraining measures always should incorporate a detailed investigation of the residual shear strength and of seasonally changing seepage and groundwater conditions. Commonly, the analyses are based on a value of $\phi > \phi_{calc} > \phi_r$. The calculation method itself mostly is of less importance than the variation of ground parameters and water pressures. Accordingly, parametric studies are of essential importance. Moreover the calculation should comprise at least two different hypotheses to check possible boundaries. To sum up, each slope stabilization and each retaining structure must be considered unique, and especially in unstable mountainous regions generally valid design rules can hardly be given. Neither should one just stick to a fix safety factor from standards which in reality cannot be proven in a conventional manner. Experience and engineering judgement play a dominating role then. The observational method provides an ecomonical solution if a calculated risk is taken over by all involved (owner, designer, consultant etc.).

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