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Design of reinforced excavations and natural slopes using new European Codes

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ABSTRACT: In 1994 the European Committee for Standardization (CEN) edited the new European Standard ENV 1997-1 "Eurocode 7: Geotechnical design - Part 1: General rules". For future design of reinforced earth structures the EC 7 will be very important. The essential difference to the conventional design methods is the application of several partial safety factors instead of only one overall safety factor. Design examples of nailed walls are presented showing the application of the various partial safety factors for loads and acting forces as well as for the properties of the materials, including soil friction and cohesion, and the proprties of steel.

1. INTRODUCTION

Over 15 years ago, work started to create common rules for geotechnical design in the European countries. Since the end of the 1980s, this work has been carried out under the direction of the European Committee for Standardization (CEN). The result is published now as ENpr 1997-1 "Eurocode 7: Geotechnical design - Part 1: General rules".

This lecture introduces the general features of the Eurocodes, and deals with the application of the geotechnical design rules of Eurocode 7 - Part 1 (EC 7-1) to insitu reinforced earth structures in particular.

2. THE NEW EUROPEAN CODES IN GENERAL

2.1 *History*

In 1980, the Commission of the European Communities and the International Society of Soil Mechanics and Foundation Engineering (ISSMFE) made an agreement that the ISSMFE should produce a draft for the European Code 7 - Part 1. A group of geotechnical experts from several European countries presented the draft in 1988. The Commission of the European Community then set up a project group which revised the draft and adapted it to the other nearly completed Eurocodes. In 1990, responsibility for the issue of the Eurocodes was transferred to CEN (Comité Européen des Normalisation) in Paris. Part 1 of Eurocode 7 was published as ENpr 1997-1 in English in November, 1994. The letters "pr" denote this stage as a European prestandard.

2.2 *Scope of the present European Codes in civil engineering*

There are nine Eurocodes in total, each containing a set of common unified design rules for some aspect of building and civil engineering works. Each Eurocode is subdivided into various parts. The ENpr 1991-1, or Eurocode 1: *Basis of design and actions on structures* - Part 1: *Basis of design*, contains the principles of design and defines the design values of actions. In addition to Eurocode 1, a further four codes which are of interest to the geotechnical engineer are mentioned here. They are titled as follows:

- EC 2 Design of concrete structures
- EC 3 Design of steel structures
- EC 7 Geotechnical design
- EC 8 Design provisions for earthquake resistance of structures.

A further set of European standards for the execution of special geotechnical work should be mentioned. These include ENpr 1536 *Bored piles*, and ENpr 1537 *Ground anchors*. The latter codes represent the best European knowledge on the execution of ground engineering works at present. However, one has to distinguish codes on design (ENpr 1991 to 1999) from codes on engineering products (ENpr 1536 to 1538). This lecture will only deal with the geotechnical design codes.

2.3 *Objectives of European Codes*

From the outset, the political objective of the Eurocodes, not forgetting the European standards for execution, was to harmonize the very different technical rules of the 15 EU-countries in order to create a real European market for the design and execution of civil engineering works.

It is important to note that, at present, the safety level of building and civil engineering works is extremely different in Europe (not to mention other parts of the world). There are many countries where national standards prescribe specific global safety factors, the magnitude of which can differ from country to country in a range of ±50% for the same check. Furthermore, there are some countries in Europe where the designing engineer fixes the safety factor at his own risk, and there are countries where the insurance company determines safety requirements. Lastly, one has to mention the fact that, due to different safety *definitions*, the safety level is not homogeneous, neither in ground engineering nor in comparision with construction engineering.

It follows from the above discussion that one of the principal objectives of the Eurocodes is to harmonize the understanding of safety requirements in two ways: specifically and internationally. Therefore, as a common basis for all EC-countries, the following two fundamentals feature in the design of building and civil engineering works:
- Limit state design format, and
- Partial factors of safety.

Thus, the above mentioned nine Eurocodes represent a set of fairly well harmonized safety standards for both of the above, covering fifteen European countries and a wide range of materials (e.g., concrete, steel, timber, and importantly soil.

In the following sections the lecture will be focussed on Eurocode 7, Part 1, which concentrates on the functional requirements for geotechnical design. Consequently, one will not find specific calculation procedures for soil nailing in this code. In any case, the design of nailed walls or slopes has to fulfill the above mentioned fundamentals: limit state and partial safety factors.

3. CODES, LICENCES AND RECOMMENDATIONS FOR SOIL NAILING

3.1 *History of soil nailing as a new construction method*

Without doubt, the idea of reinforcing new cut slopes was inspired by the new Austrian tunnelling method (NAT), which had been primarily a rock tunnelling system using a combination of shotcrete and fully bonded steel inclusions to provide early, efficient excavation stability (Bruce & Jewell, 1987). In 1972 the first case of a reinforced cutting was realized in France near Versailles, where a 70° cut slope in cemented sand was reinforced temporarily using grouted steel bars (Rabejac & Toudic, 1974). However, apart from some practical cases, the new method had not been scientifically studied in France in the seventies.

At the beginning of the seventies some applications of the NAT in Germany had been proved successful in less competent materials, such as silts, sands and gravels. Thus, in 1975 the idea of insitu reinforced soil was taken up in Germany, when the German research and development project "Bodenvernagelung" started to study and develop the new technique for stabilizing slopes and excavation cuts.

In the meantime the technical world has nearly forgotten that the technical term *soil nailing* derives from the two German words "Boden" (meaning *soil*) and "Vernagelung" (meaning *nailing*). After several publications on the experimental and theoretical results of the new technique in German and English language (Stocker 1976, Gässler 1977, Stocker & Gässler, 1979), the German contribution on *soil nailing* (Stocker, Körber, Gässler, Gudehus,

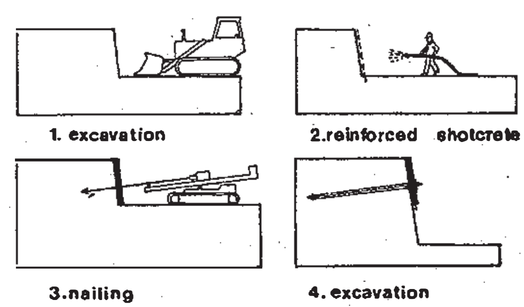


Fig. 1 Construction method of soil nailing (adapted from *soil nailing*, Stocker, Körber, Gässler & Gudehus, Paris, 1979

1979) introduced the new technical term to an international audience at the *International Conference on Soil Reinforcement* in Paris, 1979. Fig. 1 shows the very first sketch of the construction method of soil nailing in its original form from the above mentioned publication.

It should also be mentioned that in the USA a simul-taneous, but independent development of insitu reinforced earth took place in the years from 1976 to 1981, which was called "lateral earth support system" (Shen et al., 1981a). However, today *soil nailing* is also the most common term employed in America.

In France the national project "Clouterre" (clou = *nail*, terre = *soil*) was started in 1986, about ten years after the German research project "Bodenvemagelung" had been initiated. The first results of *Clouterre* were reported to an international forum in 1989 (Plumelle, 1989).

3.2 *The German research and developement project soil nailing (1975-1980)*

As mentioned above, the German research and development project "Bodenvemagelung" started in 1975. The special contractor Bauer GmbH & Co KG and the Institute of Soil Mechanics and Rock Mechanics of the University Karlsruhe in joint venture established a five year research and development project. This research programme covered the following items:

(a) Theoretical stability analyses to study the relevant failure mechanisms of nailed cuttings.

(b) Model tests to study the behaviour of nailed cuttings or nailed walls at limit equilibrium.

(c) Execution of seven full scale nailed walls to develop drilling and construction techniques in cohesive and non-cohesive soils.

(d) Instrumentation of these test walls to observe behaviour during construction, under safe loads and at limit equilibrium.

Concerning item (a), the results of the theoretical stability analyses were presented briefly in the above mentioned contribution in Paris, 1979, and in detail in a German doctoral thesis (Gässler, 1987). The reader is also referred to a comprehensive paper on theory and practical design of soil nailing, presented by the author at IS Kyushu 88 (Gässler, 1988).

The author's philosophy, followed in all his publications from the very beginning, was to design nailed cuttings against the occurrence of a limit state. In the Paris, 1979, paper circular surfaces, two-part wedge mechanisms and the Coulomb wedge were anticipated as potential failure mechanisms for nailed walls (Fig. 2), and a procedure was

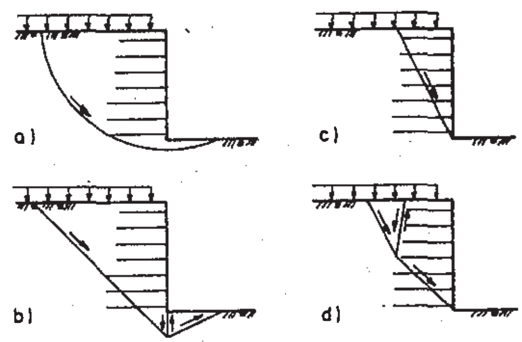


Fig. 2 Potential failure mechanisms of nailed walls (adapted from *Soil nailing*, Stocker, Körber, Gässler & Gudehus, 1979)

presented for the determination of the most critical failure surfaces of a two-part wedge mechanism (Fig. 3).

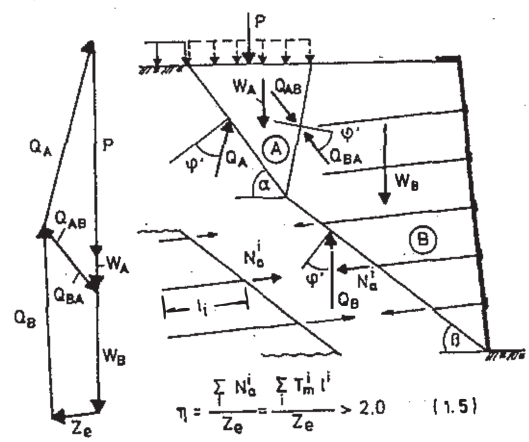


Fig. 3 Two-part wedge analysis for a nailed wall (adapted from *soil nailing*, Stocker, Körber, Gässler & Gudehus, 1979)

Since in the seventies and eighties overall factors of safety were common in geotechnical engineering in nearly all countries, the author used the overall factor η applied to the nail forces at that time (see Fig. 3). This factor $\eta = 2.0$ was estimated to provide for a safe distance from the occurence of failure.

Referring to (b), model tests (scale 1:25) on nailed walls were first presented in Paris, 1979 (Stocker et al.), and then in 1983 (Gässler & Gudehus, 1983). In all tests the observed failure mode was in agreement with theoretical predictions. This has been documented by photographs and by analyses published in the doctoral thesis mentioned above.

Referring to (c) and (d), one can state that all seven fully instrumented test walls up to 6.9 m height yielded many results of theoretical and practical importance. As it is not the object of this paper to deal with the experimental results in detail, the reader is referred to Gässler (1992a & 1993)

Additionally, in the beginning of the eighties the author studied the statistic-probabilistical safety theory after Hasofer & Lind (1974) to deduce partial safety factors for the design of nailed walls following the Eurocode 7, which was in drafting stage at that time. Thus in 1983, for the first time, the set of partial safety factors in Table 1, applied to characteristic values (see ch. 4.3), was proposed to assure a certain safety level (probability of failure $p_f \approx 10^{-6}$) for nailed slopes and walls (Gässler, 1982; Gässler & Gudehus, 1983).

Table 1: Partial safety factors proposed by Gässler (1982), and Gässler & Gudehus (1983)

	Partial safety factors
Nature of forces	
Permanent forces:	
Soil unit weight	$\gamma_\gamma = 1.0$
Variable forces:	
Live loads	$\gamma_Q = 1.1$
Material properties:	
Soil friction	$\gamma_\varphi = 1.1$
Effective cohesion	$\gamma_c = 1.4$
Soil-nail-interaction	$\gamma_N = 1.3$

3.3 *German licences for soil nailing (1984)*

Presenting a new technology soil nailing was not acknowledged as a so-called "common construction method" at its very beginning in Germany. Therefore, until 1983, special permission for each soil nailing project was required by the construction administration of the communities. In all cases this permission was given, as the reliability of design and performance could be proved by the model and field tests of the research programme *soil nailing* and, last but not least, by various well instrumented practical projects.

As the special contractor Bauer had pioneered soil nailing in Germany, the first general licence for soil nailing was issued to the contractor Bauer by the German Federal Institute for Construction Techniques, Berlin, in 1984. Naturally, the design and calculation methods obtained by the research project *soil nailing* were applied in this general licence of *Soil Nailing System "Bauer"*, which is still valid at present. The following items of the *Bauer-licence* shall be given:

(1) General requirements of application
(2) Field of application
(3) Soil exploration
(4) Requirements for the construction of temporary nails and permanent nails and for the facings
(5) Requirements for predrilling the boreholes and installing the nails
(6) Check of internal and external stability
(7) Pull-out tests of nails
(8) Quality control (construction supervision)
(9) Monitoring under service

In contrast to anchorage licences dealing with the construction and installation of single anchors, the licence for soil nailing covers both the nail individually and the nail-reinforced soil system as a whole, including the design and calculation procedure. This is due to the particular history of development of soil nailing: not only the nail as such, but the construction method was entirely new.

Returning to item (6) *check of internal and external stability*, it is interesting to mention that all checks for external stability (e.g., circular surfaces), and internal stability (e,g., two-part wedge mechanism, see Fig. 4) of the system as a whole, as well as the checks for the nail as an element against bond failure with the ground (external stability of a nail) or rupture of the steel (internal stability of a nail) are to be undertaken in limit state using overall safety factors.

The licence of *Soil Nailing System "Bauer"* was the first "code of practice" in the history of soil nailing. However, as this licence has never been

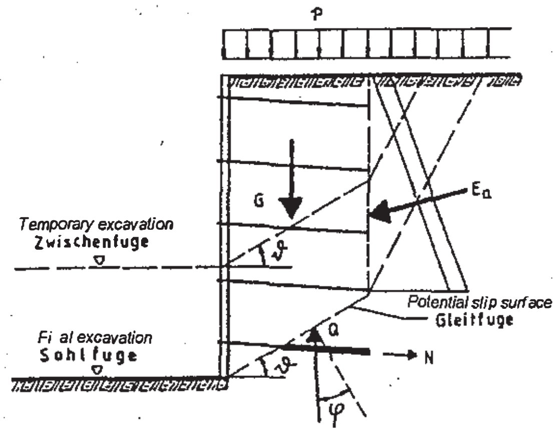


Fig. 4 Two-part wedge analysis for internal stability of a near vertical nailed wall (adapted from the German general licence of *Soil Nailing System "Bauer"*, 1984)

translated into English, no one outside Germany, Austria and Switzerland has taken any notice of it. In the meantime licences for several other German special contractors followed, so that soil nailing became definitely popular as an advantageous and reliable construction method in Germany (Gässler, 1989 & 1995). These licences helped to ensure a high standard of execution of soil nailing.

It should be remarked that as soil nailing has been a steadily developing method in both construction and design in the eighties, it would not have been advisable to produce a German Standard (DIN) for soil nailing. This would have certainly restricted its progress in both practice and theory.

3.4 *French soil nailing recommendations (1991)*

The Soil Nailing Recommendations (original French title: *Recommandation Clouterre 1991*) represent the five years of research, studies and tests (including three field tests on walls, 6 m and 7 m high) of the French national project *Clouterre*, which was conducted from 1986 to 1990. It summarizes the whole design and construction process, from geotechnical investigation to field quality control. In the *Recommendations 91* the reader will find many references to French publications and documents, but relatively few references to research and practical experience outside France. Many important results (e.g. the limit state as optimal design approach, the safety relevant failure mechanisms, or the observed displacements on the top of nail facings) are presented without any background information on results yielded by the research programme *Bodenvernagelung* ten years before.

Nevertheless, the French *Soil Nailing Recommendations 91* represent a remarkable compilation from the studies on nailed walls and slopes carried out by a large group of contracting authorities, prime contractors, research centres and laboratories, consulting firms, and construction companies. It is a very extensive work, from which only the main chapters can be cited here as follows:

(1) The technique used for soil nailed structures: Description and developements
(2) Soil nailing in retaining structures: Mechanisms and behaviour
(3) Conception and design
(4) Investigation and tests
(5) Wall structures construction
(6) Durability of structures
(7) Specifications and inspections

Table 2: Partial safety factors applied to characteristic values, recommended in the *Recommandation Clouterre 1991* (excerpt)

	Partial safety factors
Nature of forces	
Permanent forces:	
Soil unit weight	
de-stabilizing force	$\Gamma_{s1} = 1.05$
stabilizing force	$\Gamma'_{s1} = 0.95$
Other *permanent* forces	
unfavourable forces	$\Gamma_{s1} = 1.20$
favourable forces	$\Gamma'_{s1} = 0.90$
Variable forces:	
Live loads	$\Gamma_{Q} = 1.33$
Material properties:	
Soil:	
tangent of the friction angle	$\Gamma_{m,\varphi}{}^{1} = 1.20$
effective cohesion	$\Gamma_{m,c} = 1.50$
undrained cohesion	$\Gamma_{m,cu} = 1.30$
Mild steel	$\Gamma_{m,\sigma c} = 1.15$
Soil-nail-interaction (tests)	$\Gamma_{m,qs} = 1.40$

With respect to the subject of this lecture, chapter (3) is the most interesting. For the justification of the stability of the soil nailed wall at ultimate limit state, the reader will find there a set of partial safety factors for loads and for material properties, i.e., soil, steel and soil-nail interaction. Table 2 gives an excerpt of the main values from the *Recommandation Clouterre 1991.*

Whereas in early French publications on soil nailing (e. g., Schlosser, 1982) the limit state design with one overall factor. had been proposed, the French *Soil Nailing Recommendations 91* follow the safety concept of Eurocode 7 in principle. However, there is no derivation or any explanation of where the values of the various partial safety factors were taken from.

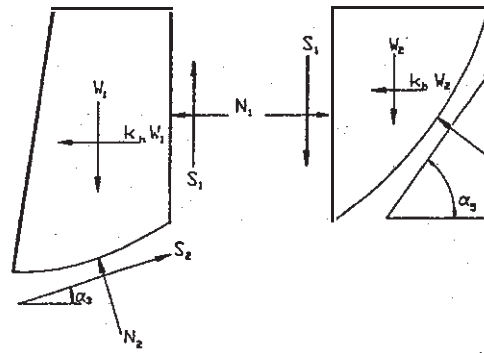


Fig. 5: Design method for nailed walls in the USA after Bang et al. (1992)

3.5 *American research and developement projects*

At the University of California at Davis, a limit state design method was developed in the late seventies, absolutely independently from European activities in soil nailing (Shen et al. 1981b).

The potential failure line is assumed to be a parabola, passing either only through the nailed zone, or through both the nailed zone and the natural soil in the rear. In the latter case, two blocks are separated by a vertical line through the lower end of the nails, where the intersection forces have to be estimated. The global factor FS of safety is calculated by comparing the component of the total resisting force along the direction of driving force with the magnitude of total driving force (Bang, Kroetch & Shen, 1992). Using the same global factor for both nails and soil, the minimum safety factor determines the most critical parabola (Fig.5). For permanent structures, a minimum global factor of 1.5 is proposed.

However, in the author's opinion, it is not expedient in homogeneous soils to seperate the potentially sliding earth block into two blocks, if there is no necessity for it. Therefore, the German design method on the basis of *one* rigid body in rotation (circular slip surface) avoids intersection lines with intersection forces that cannot be determined without questionable assumptions (Gässler, 1988).

Another quite different design method after Juran (1990) has to be mentioned which divides the nail-reinforced block into slices *parallel* to the nails. The potential failure line is assumed to be a logarithmic spiral.

Despite of the fact that this lecture deals with design of nailed slopes, a recent practical recommendation for monitoring performance of nailed structures should be mentioned. It is the *FHWA Soil Nailing Field Inspector's Manual*, edited by the Federal Highway Administration of the U.S. Department of Transportation in 1994. In the author's opinion, it represents the best manual of its kind for soil nailing practice in the world at present.

3.6 *British Standard BS 8006 (1996)*

This new British Standard containes guidelines and recommendations for the application of soil reinforcement techniques to soils, as fill or in situ. In the scope of this lecture it is not possible to report on this extensive standard in detail. However, two main principles of this standard should be noted:

- it is written in a limit state format, and
- guidelines are provided of safety margins in terms of partial load factors and material factors.

The major part of the BS 8006 (1996) deals with reinforced masses of fill using metallic strips or polymeric strips, sheets or grids. However, soil nailing and in situ reinforcing is considered as well, and background information is provided by a substantial bibliography to help the user gain a deeper understanding.

One should also mention the new DOT Advice Note, HA 68/94, which forms part of the Design Manual for Roads and Bridges in the U.K. The Document, entitled *Design methods for the reinforcement of highway slopes by reinforced soil and* *soil nailing techniques*, is based on two-part wedge analysis, where friction is introduced along the interwedge boundary (Love, 1995). This procedure is in conformity with BS 8006 and will be discussed later.

3.7 *Design methods in Japan*

The author regrets his lack of information on design procedures for soil nailing in Japan. What can be seen from international publications by Japanese authors is that a limit state format with an overall factor F_S seems to be more common (e.g., Maramatsu et al., 1992; Taga et al., 1992; Teramoto et al., 1992; Tsubouchi et al., 1992) than with partial safety factors.

4. THE AUTHOR'S DESIGN METHOD USING PARTIAL SAFETY FACTORS

4.1 *Method of kinematics of rigid earth blocks*

The kind of plastic limit state, which is reached by earth retaining structures after sufficiently large deformations, depends on the statical and kinematical boundary conditions. It was assumed for nailed walls and slopes that, in the nailed zone as well as in the unnailed zone of the soil, the plastic shear deformations are located in thin shear planes, whereas the greater part of the soil remains rigid in the limit state. Thus, the failure model of nailed walls was based on the kinematics of rigid earth blocks containing two principles (Gudehus, 1981):

(1) Principle of the kinematic compatibility of the failure mechanism

This means the displacements of rigid earth blocks have to be correctly described by a hodograph.

(2) Principle of the minimum of safety by means of the variation of slip planes.

This means, one has to vary the inclinations (and radii) of the slip planes of a potential failure mechanism (e.g., translation mechanism or rotation mechanism) until the most unstable configuration of the slip planes is found.

As a provisional safety definition

$$\eta_N = \frac{Z_a}{Z_g} \tag{1}$$

can be used. Herein Z_a denotes the available axial nail forces (determined from pull-out tests) and Z_g the axial nail forces in equilibrium or limit state. The principle of the minimum of safety states that failure will occur with that configuration of slip planes, which gives a value of 1 in the safety definition (1). This is analogous to the well known principle of Coulomb's earth pressure theory (c.f. Gudehus, 1981).

Using the regular cross section of a nailed wall, the safety of different potential failure mechanisms consisting of rigid blocks was investigated and compared for various boundary conditions. The procedure systematically develops in three steps:

1. Compiling of potential modes of failure mechanisms,
2. Variation of the slip planes to find the least safe configuration for each failure mode,
3. Determination of the absolutely least safe failure mechanism by means of comparing the least safe configurations of the different failure modes.

The investigated failure modes included:
- translation of a rigid block (simple wedge, TRA-I)
- translation of two rigid blocks (two-part wedge, TRA-II)
- rotation of one rigid block (slip circle, ROT-I)
- rotation of two rigid blocks (ROT-II).

In its simplest form the method of kinematics of rigid earth blocks can be best described by means of example as follows:

In Fig. 6a a cross section of a near vertical slope is shown. The soil has an angle of internal friction $\varphi' = 30°$, cohesion $c' = 7.5$ kN/m², and unit soil weight 20 kN/m³. The vertical nail spacing s_v is 1.1 m, and the horizontal spacing s_h is 1.25 m. As mean value, the *available* nail shear force $T_{m,a} = 30$ kN/m has been anticipated by in situ pull-out tests of several nails.

The assumed failure mechanism is a slip circle (or in other words: a rotation mechanism of one rigid block), determined by the chord inclination angle $\vartheta = 50°$ and the radius $r = 1.5h$. This slip circle is arbitrary and not yet the unsafest configuration. The external and internal forces acting on the sliding earth block are shown in Fig 6b.

The internal force N_i of a nail in one of the lower rows i, which is intersected by the slip circle, is determined by the nearly constant mean shear force T_m (N.B., this shear force acts *parallel* to the nail axis) and the section of the nail beyond the slip plane l_i:

$$N_i = T_m \cdot l_i \tag{2}$$

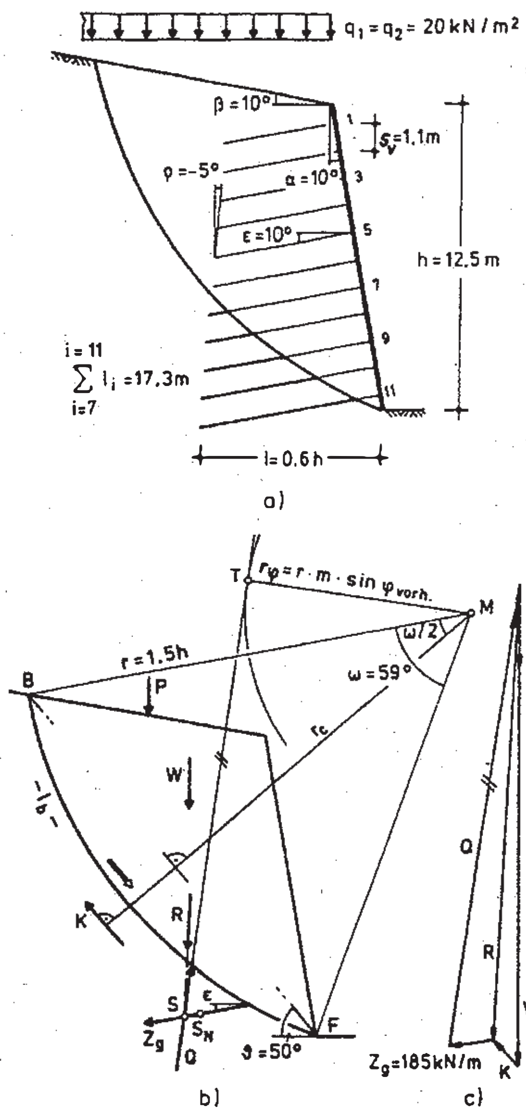


Fig. 6 Stability calculation of a nailed wall: a) cross section with slip circle, b) free body diagram, c) force polygon

The sum of the axial forces, referring to a unit width of the wall, can be expressed by:

$$Z_g = 1/s_h \cdot \sum_{i=j}^{i=n} N_i = T_{m,g}/s_h \cdot \sum_{i=j}^{i=n} l_i \qquad (3)$$

Herein j denotes the upmost row, and n the lowest row that is intersected by the slip plane.
Z_g is located in the centre of gravity S_N of the nailed zone beyond the slip circle. The forces R and Z_g are balanced in point S by the resultant slip circle force Q, the latter determined with the friction circle assumption after Krey (1932). Completing the force polygon in Fig. 6c yields the sum of the axial nail forces in the limit state Z_g = 185 kN/m. From Equ.(3) follows $T_{m,g} = 185 \cdot 1.25/17.3 = 13.4$ kN/m.

The normalized term, equivalent to $T_{m,g}$, is the specific nailing density μ_g, which is defined as follows:

$$\mu_g = T_{m,g} / (\gamma \cdot s_v \cdot s_h) \qquad (4)$$

where γ = unit weight of the soil [kN/m³]
s_v = vertikal nail spacing [m]
s_h = horizontal nail spacing [m]

As with $T_{m,g}$, the index g in μ_g denotes the equilibrium state. The specific nailing density was first introduced by Gässler & Gudehus (1981) for use in design charts (see ch. 6). Thus, one obtains from Equ.(4) $\mu_g = 13.4/(20 \cdot 1.1 \cdot 1.25) = 0.487$.

Instead of the safety factor η_N (see Equ.(1)), it is now more expedient to use the equivalent safety definitions:

$$\eta_T = \frac{T_{m,a}}{T_{m,g}} \quad \text{or} \quad \eta_\mu = \frac{\mu_a}{\mu_g} \qquad (5), (6)$$

which can be obtained from Equ.(1), (3), and (5).

Consistently one yields the same minimal safety factors η_N, η_T, or η_μ for the slip circle in Fig. 6:

$\eta_T = 30/13.4 = 2.24$, or $\eta_\mu = 1.091/0.487 = 2.24$

As second step, one has to vary the parameters ϑ and r of the slip circle, until the *maximum* of $T_{m,g}$, or μ_g, is found. By means of a computer program, the solution of the arithmetic variation is found:

$\mu_{g,max} = 0.636$ with $\vartheta^* = 49.6°$ and $r^* = 3.4 \cdot h$.

The equivalent *minimum* safety factor is then:

$$\eta_{\mu,min} = \mu_a/\mu_{g,max} = 1.091/0.636 = 1.72$$

It is interesting to mention here, that, for the given wall geometry and soil parameters, practically the same result will be obtained on the basis of the two-part wedge mechanism (TRA-II).

Emphasis is placed here on the very important hypothesis for the validity of limit state design, namely that the maximal resistance of the soil and of the nails in the various rows is simultaneously mobilized at failure. For vertical or near to the vertical slopes this hypothesis is only valid for the tensile resistance (in other words: the pulling-out resistance) of the nails, but *not* for shear forces due to bending. In sandy or clayey soils, the latter are mobilized too late. This was measured by instrumented nails during failure of several full scale test walls carried out in the research project *Bodenvernagelung* (Gässler 1987, 1990a & 1993). Maybe, at *post* failure the bending resistance of the nails can prevent nailed walls from undergoing total collapse. However, this is not relevant for the conditions immediately antecedent to failure. As a conclusion from all the measurements and analyses of several full scale tests, the shear forces of nails of diameters of about 20 to 28 mm are of second order. This is opposite to the French recommendations *Clouterre*, but in agreement with measurements in large shear boxes, carried out by Pedley, M.J., Jewell, R.A. & Milligan G.W.E. 1990.

Therefore, the calculation method shown in Fig. 6 is based only on the *axial* nail forces. The shear forces are not ignored in order to obtain the most simple calculation method; they are ignored for safety reasons. This results in a slightly conservative, but very expedient design procedure.

4.2 *General rules for finding the unsafest failure mechanism*

The third step should now be made to find the least safe failure mechanism in the calculation example of Fig. 6. In principle, there are four possible failure modes to check (see ch. 4.1). The least safe mechanism is the one that leads to the *absolute* maximum $T_{m,g,max}$, or, $\mu_{g,max}$. For various soil properties and boundary conditions the absolutely maximum specific nail densities from the four potential failure mechanisms have been analysed.

Considering the calculation example of Fig. 6, the result is that the ROT-I-mechanism (slip circle) and the TRA-II-mechanism are similarly unsafe, and that both ROT-I and TRA-II are less safe than TRA-I (simple wedge) (cf. Gässler, 1988).

In general, the results of numerous calculations performed by the author (Gässler, 1987) can be summarized into the following rules for practical use:

- In *cohesionless* soils the slip circle and two-part wedge are the two most critical mechanisms for vertical or near vertical nailed walls (the slight difference is here only of academic interest). For less steep walls and/ or longer nails in the upper rows, the slip circle is the least safe mechanism. Especially for high loads beyond the reinforced zone, the two-part wedge is the least safe mechanism.

- In soils with *little cohesion*, both the slip circle and the two-part wedge are, as far as safety is concerned, nearly equivalent for steeper walls.

- In soils with *medium or high cohesion* the slip circle distinctly is the least safe failure mode.

The rotation of two rigid blocks may occur in the case of extremely high surface load beyond the reinforced zone. This has been observed in a model test with sand (Gässler & Gudehus, 1983). Rotation of two rigid earth blocks has also been found as the least safe failure mode for seismic loading (Tufenkjian & Vucetic, 1992). The principal graphic solution for this type of a combined failure mechanism is given in the discussion leader's report *slopes and excavations* of the IS Kyushu '92 (Gässler, 1992b). More background information on this most interesting failure mode is given by Gässler (1987).

4.3 *Deduction of partial safety factors on the basis of the statistic-probabilistic safety theory*

Instead of only one global factor η_N (see Equ. (1)), or η_μ (see Equ. (6)), it is proposed to make use of various partial safety factors. For the derivation of partial safety factors a limit state equation is necessary. For this a limit state equation has been formulated based on the two-part wedge mechanism (Gässler & Gudehus, 1983).

In the limit state equations for every failure mode of nailed slopes, the magnitudes q (e. g., live surface load, dimension kN/m²), φ (friction angle, dimensionless), cohesion c (dimension kN/m²), and T_m (mean nail shear force, dimension kN/m) are scattering, i. e. they are basic variables in the sense of the new statistic-probabalistic safety theory in EC 7. Concerning the statistical distribution of the basic variables, assumptions have to be made, as the available quantity of statistic geotechnical data is very poor at present.

Nevertheless, partial safety factors can be developed on the basis of numerous so-called Level II approach calculations after Hasofer & Lind (1974). The Level II approach applied on soil nailing is appoximately described by Gässler and Gudehus (1983).

Readers interested in the principal procedures of such calculations, are referred to the structural diagram of the computer program that has been developed by the author to derive partial safety factors (Gässler, 1987). As result the following set of factors was achieved, providing a sufficiently high and homogeneous safety level for steep nailed walls (safety index $\beta \approx 4.7$, or in other words: probability of failure $p_f \approx 10^{-6}$):

$$\gamma_Q = \frac{q_d}{q_m} = 1.3 \qquad (7)$$

with q_m : mean value of surcharge q,

$$\gamma_\varphi = \frac{\varphi'_k}{\varphi'_d} = 1.1 \qquad (8)$$

with φ'_k: characteristic value of φ' (=10%-fractile of a log normal distribution, truncated at 20°),

$$\gamma_T = \frac{T_{m,k}}{T_{m,d}} = 1.3 \qquad (9)$$

with $T_{m,k}$: characteristic value of T_m (= 10%-fractile of a log normal distribution).

The index "d" denotes the so-called design point, i. e. the assumed limit state equlibrium with the most unfavourable combination of all basic variables. The values of the partial safety factors in (7), (8) and (9) have already been compiled in Table 1 (see ch. 3.2).

In the Level II approach calculations, it has been found by the author that the scattering of unit soil weight γ[kN/m³] is small compared to the shear parameters φ' or c, so that the safety level is not as much influenced by γ as by φ' or c. Therefore, very early, the author has proposed the partial safety factor $\gamma_G = 1.00$ for soil weight as a permanent load (Gässler & Gudehus, 1983). At this point, it does not matter wether the soil weight is an unfavourable or favourable load. As soil nailed walls are not embedded in the ground, soil weight mostly acts unfavourably. The convention to set $\gamma_G = 1.00$ is very expedient and eases conventional and computer calculation.

However, in the meantime, the partial safety values given in Equ. (7), (8) and (9) are obsolete. In future, the partial safety factors for loads will be prescribed by the Eurocode 1, and the partial safety factors for the material properties of the ground, say φ', c', T_m, will be recommended by Eurocode 7.

5. THE NEW EUROPEAN CODES (EC 1, EC 7)

It is not the object of this lecture to present the content of both codes in detail. The lecture only aims to give a brief overview and to set out what is important for the design of nailed slopes.

5.1 *Eurocode 1, Part 1 (ENV 1991-1:1994)*

The EC 1-1 describes the principles and requirements for safety, serviceability and durability of structures. It is based on the limit state concept used in conjunction with a partial factor method. For the design of new structures, the EC 1 is intended to be used for direct application, together with the design Eurocodes (EC 2 to EC 9; cf. ch. 2.2). The EC 1 is divided into a main text and a series of annexes. The main text includes the principles and most of the application rules necessary for direct application for designs in the field. The annexes are informative only. In order to give an idea of the content of the main text, the titles of the main sections are given as follows:

1 General
2 Reqirements
3 Limit states
4 Actions and environmental influences
5 Material properties
6 Geometrical data
7 Modelling for structural analysis and resistance
8 Design assisted by testing
9 Verification by the partial factor method.

The principles comprise (cf. EC 1-1 sec. 1.4):
(a) general statements and definitions for which there is no alternative
(b) requirements and analytical models for which no alternative is permitted unless specifically stated.

The application rules are generally recognized rules which follow the principles and satisfy their requirements. It is permissible to use alternative rules to the application rules given in the Eurocode.

In sec. 3 of EC 1-1 it is formulated, as a principle, how limit state design shall be carried out, namely by:
- setting up structural and load models for relevant ultimate and serviceability limit states to be considered in the various design situations and load cases;
- verifying that the limit states are not exceeded when *design* values for actions, material properties and geometrical data are used in the models.

Design values are generally obtained by using the *characteristic* values in combination with partial safety factors.

A characteristic value F_k of an *action* is the principal representative value of an action. If this characteristic value can be fixed on a statistical basis, it is chosen so as to correspond to a prescribed probability of not being exceeded on the unfavourable side during a reference period, e.g., design working life (EC 1-1, sec. 1.5.3.14)

A characteristic value X_k of a *material property* (or *resistant* force) is a value having a prescribed probability of not being attained in a test series. This value generally corresponds to a specified fractile of the statistical distribution of the particular property of the material (EC 1-1, sec. 1.5.4.1).

The design value F_d of an action is expressed in general terms as:

$$F_d = \gamma_F \cdot F_k$$

with γ_F: partial factor for actions (see Table 3)

The design value X_d of a material property (e.g., soil friction or yield point of steel) is generally defined as:

$$X_d = X_k / \gamma_M$$

with γ_M: partial factor for material property, given in EC 2 to EC 9 (for ground properties see Table 4 in ch. 5.2 of this lecture).

Table 3: Partial safety factors for actions in limit states (excerpt from EC 1-1, Table 9.2)

	Partial safety factors
Case B (Failure of structure or structural element governed by strength of material) *Permanent* actions - unfavourable - favourable *Variable* actions - unfavourable	 γ_{Gsup} = [1.35] γ_{Ginf} = [1.00] γ_Q = [1.50]
Case C (Failure in the ground) *Permanent* actions - unfavourable - favourable *Variable* actions - unfavourable	 γ_{Gsup} = [1.00] γ_{Ginf} = [1.00] γ_Q = [1.30]

One has to state that in the currently available set of Eurocodes, the partial safety factors are partly based on probabalistic considerations (very few on Level II approach calculations; cf. ch. 4.3), partly on a historical or empirical derivation (the latter seems to be predominant in geotechnical design).

Concerning partial factor for actions, one has to differentiate three cases of ultimate limit states:

Case 1: Loss of static equilibrium of a structure as a whole (e. g., hydrostatic uplift)

Case 2: Failure of structure or structural elements, including those of the footing, piles, basement walls etc., governed by the strength of structural material

Case 3: Failure in the ground

As can be seen in Table 3, the partial safety factors are different in these cases. (As Case 1 is less relevant for soil nailed structures, the respective partial factors are not given in the excerpted Table 3.) There are good reasons for dividing limit states into three cases. The main reason for introducing Case 3 is the fact that the partial safety factor for permanent unfavourable actions γ_{Gsup} = 1.35, used in structural engineering (EC 2, EC 3), is absolutely unacceptable in geotechnical design. The factor 1.35 is too high and not justified, because the unit weight of soil does not scatter to this extent. Hence, it would not make any sense to apply this factor 1.35 to the unit soil weight (in other words: to the dead weight of destabilizing earth blocks).

The essential point of the new safety concept in the ECs is the following (cf. EC 1-1, sec. 9.4):

When considering a limit state of static equilibrium of the structure as a rigid body, it shall be verified that:

$$E_{d,dst} \leq E_{d,stb}$$

where: $E_{d,dst}$: design value of the total effect of destabilizing actions (or forces)

$E_{d,stb}$: design value of the total effect of stabilizing (or resistant) forces.

This equation is a symbolic expression, which is to be replaced by an interaction formula or limit state equation.

When considering a limit state of rupture or excessive deformation of a section, member or connection, it shall be verified that:

$$E_d \leq R_d$$

where: E_d: design value of the effect of actions, such as internal force or moment

R_d: design value of the resistance associating all relevant structural properties.

5.2 *Eurocode 7, Part 1 (ENV 1997-1:1994)*

This prestandard applies to the geotechnical aspects of the design of buildings and civil engineering works. It is subdivided into the following sections:

1 General
2 Basis of geotechnical design
3 Geotechnical data
4 Supervision of construction and maintenance
5 Fill, ground improvement and reinforcement
6 Spread foundations
7 Pile foundations
8 Retaining structures
9 Embankments and slopes.

This prestandard is intended for experimental practical application, and this is why certain safety elements in this code have been assigned indicative values which are identified by [ ]. Thus, the partial safety factors in Table 4 are given as so-called boxed values. The authorities in each member country of the European Community are expected to assign definitive values to these safety elements.

Hence each member state has issued a National Application Document (NAD) giving definitive values for safety elements and referencing compatible supporting standards.

The sections 2 and 8 are the most important for the design examples in the next chapter. However, this lecture can only refer to one essential point, namely the selection of characteristic values of soils. EC 7 says that this selection shall take account the geological and other background information (data from previous projects), the variabilities of the property values and the extent of the zone of the ground governing the behaviour of the geotechnical structure at limit state. This is rather general and it seems that, as with the conventional *calculation* value, the assessment of the *characteristic* value will depend on individual engineering judgement and less on statistics.

Table 4: Partial safety factors for ground properties (excerpt from EC 7-1, Table 2.1)

Case C (Failure in the ground)	Partial safety factors
Ground properties:	
Soil friction $\tan\varphi'$	γ_φ = [1.25]
effective cohesion c'	γ_c = [1.60]
undrained cohesion cu	γ_{cu} = [1.40]

6. PRACTICAL DESIGN AFTER EC 7

6.1 *Design example No. 1*

Consider a 10 m high, near vertical slope of an urban excavation pit in sand. The groundwater table is situated very deep beneath the bottom of the excavation. The owner of the neighbouring property will not allow any construction elements to be placed in his ground. Under these conditions a nailed wall will undoubtedly represent an expedient solution. (Note that an anchored wall would require too long anchor rods!)

Thus, a nailed wall could be designed as shown in Fig. 7.

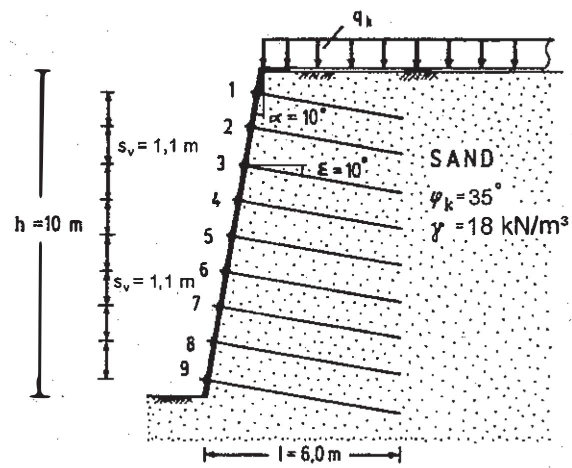


Fig. 7 Design example for a near vertical nailed wall in sand charged with a live load

The following data are given:

characteristic soil parameters: $\varphi'_k = 35°$
$\gamma_k = 18$ kN/m³
characteristic live load: $q_k = 23$ kN/m²
characteristic value of pull-out resistance of a set of test nails: $T^{\#}_{m,k} = 30$ kN/m
(The symbol # detotes $T_{m,k}$ as a test result)

predesigned wall dimensions:
wall inclination $\alpha = 10°$
nail length at the bottom: l = 6,0 m
vertical nail spacing: $s_v = 1.1$ m
line through lower nail ends: vertical

The required *horizontal* nail spacing s_h [m] and the required diameter of the steel bar in the nails are to be determined for the nailed wall in complete state.

1st Calculation step: Derivation of design values:

Design values of soil parameters:
$\varphi'_d = \arctan(\tan\varphi'_k/\gamma_\varphi) = \arctan(\tan 35°/1.25) = 25°$

Comment: For limit state case 1C: $\gamma_\varphi = 1.25$ (see Table 2.1 in EC 7-1, or Table 4 in ch. 5 of this lecture)
$\gamma_k = \gamma_d = 18$ kN/m³

Comment: For permanent actions in case 1C, the partial factors γ_{Gsup} and γ_{Ginf} are 1.00. (see Table 9.2 in EC 1-1, or Table 3 in ch. 5 of this lecture). Hence, for unit soil weight, the design value is identical with the characteristic value.

Design value of load:
$q_d = \gamma_Q \cdot q_k = 1.30 \cdot 23 = 30$ kN/m²

Comment: In case 1C, $\gamma_Q = 1.30$ for unfavourable variable actions (see Table 9.2 in EC 1-1)

2nd Calculation step: Failure mode

Now, one has to select the failure mechanism which the calculation should be based on. In this example the two-part wedge mechanism (TRA-II) will be the basis of the limit state design.

Comment: Considering the boundary conditions and soil parameters, a solution based on the slip circle (see Fig. 6) would be practically equivalent and yield the same result (see also ch. 4.2)

3rd Calculation step: Identifying the unsafest two-part wedge mechanism

One has to identify the two-part mechanism which requires the maximum reinforcement force, i. e. the maximum mean shear force per nail meter $T_{m,max}$. This can be found by searching systematically, varying the inclination ϑ of the slip surface of the reinforced earth block.

Comment: Opposite to DOT Advice Note, HA 68/94, U. K. (see ch. 3.6), it is not advisable to vary any other parameters. It is justified by model tests (Gässler & Gudehus, 1983) that the interwedge boundary coincides with the line of lower nail ends, if the nails are not too long, say l/h < 0.7. Thus, for soil nailing, it is not recommended by the author to assume an interwedge boundary within the reinforced zone.

In this case it is expedient to start the variation of the slip surface with $\vartheta^{(1)} = 35°$ and $\Delta\vartheta = 5°$.

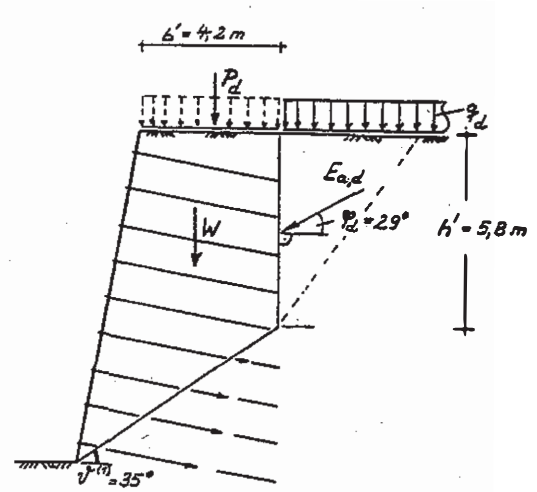


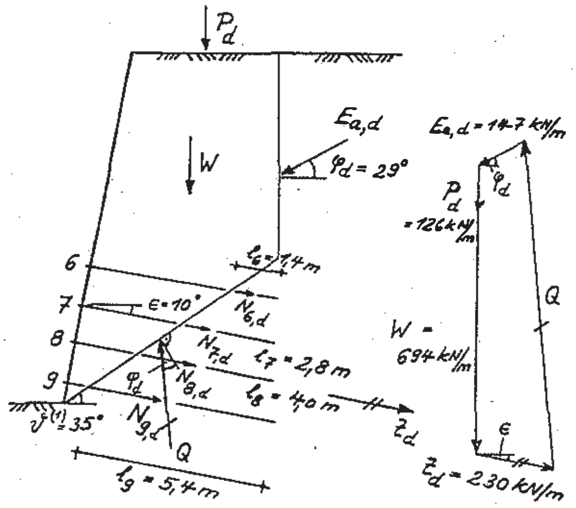
Fig. 8 Two-part wedge mechanism (not reinforced wedge replaced by active earth pressure $E_{a,d}$)

1st variation $\vartheta^{(1)} = 35°$ (see Fig. 8):
Driving forces:

$P_d = q_d \cdot b' = 30 \cdot 4.2 = 126$ kN/m
Comment: P_d is the resultant load on the reinforced wedge
$W_d = V \cdot \gamma_d = 38.55 \cdot 18 = 694$ kN/m
Comment: V is the Volume of the reinforced wedge per unit width of the wall
$E_{a,d} = ½\, h' \cdot (\gamma_d \cdot h' + 2q_d) \cdot K_a(\varphi'_d, \delta_{a,d})$
Comment: In limit state case 1C the design ground properties γ_d, φ_d and the design live load q_d are used to calculate the design earth pressure force (see EC 7-1, ch. 2.4.2, (16)P).
The coefficient of active earth pressure K_a is also a function of the interwedge friction $\delta_{a,d}$, which should, in the author's opinion, be set at φ'_d. The *DOT Advice Note, HA 68/94, U. K.*, safely sets $\delta_{a,d}$ at ½ φ'_d. Of course, details of that kind are not dealt in the EC 7. The coefficient of active earth pressure K_a can be calculated using the well-known formula.
With $K_a(\varphi'_d, \delta_{a,d}) = 0.308$ the design value of the active earth pressure acting on the reinforced wedge is yielded as follows:
$E_{a,d} = ½ \cdot 5.8 \cdot (18 \cdot 5.8 + 2 \cdot 30) \cdot 0.308 = 147$ kN/m.

Resistant forces:

The free body diagram in Fig. 9a shows both driving forces and resistant forces acting on the nailed wedge.
The resultant nail force Z_d (= sum of the intersection nail forces from row No. 6 down to row No. 9) that is required for the equilibrium in the design state is obtained from the force polygon in Fig. 9b.



a) b)

Fig. 9 a) Free body diagram with $\vartheta^{(1)} = 35°$,
b) Force polygon

Referring to Equ. (3) in ch. 4.1, the resultant nail force Z_d = 230 kN/m is equal to the following expression:

$$Z_d = 1/s_h \cdot \sum_{i=6}^{i=9} N_{i,d} = T_{m,d}/s_h \cdot \sum_{i=6}^{i=9} l_i$$

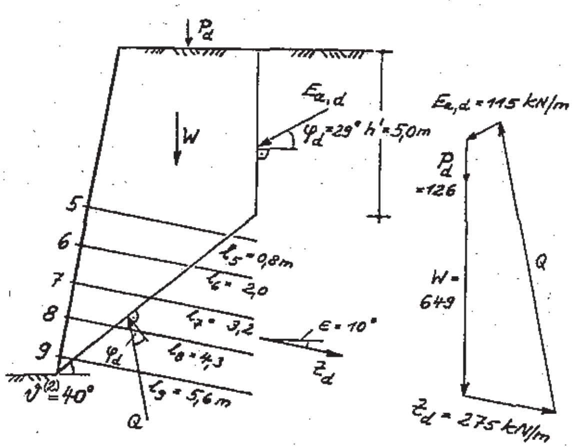
with: $s_h = 1.0$ m (= unit width).

$$230 = T_{m,d} \cdot (1.4 + 2.8 + 4.0 + 5.4)$$

Now, one obtains the mean shear force per nail meter $T_{m,d}$ required for equilibrium at $\vartheta^{(1)} = 35°$:

$$T_{m,d} = 230/13.6 = 16.9 \text{ kN/m/m.}$$

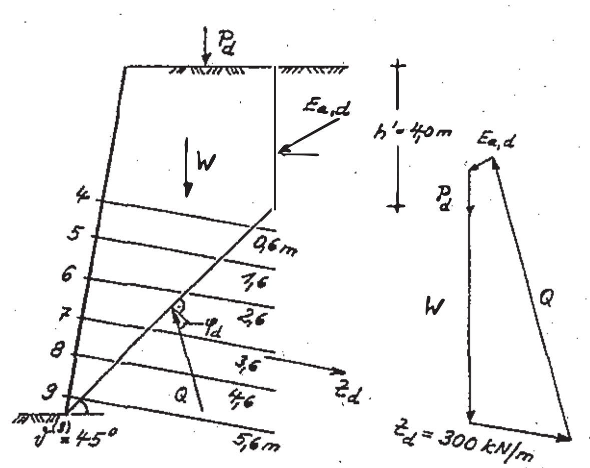
2nd variation $\vartheta^{(2)} = 40°$ (see Fig. 10):



a) b)

Fig. 10 a) Free body diagram with fwith $\vartheta^{(2)} = 40°$,
b) Force polygon

3rd variation $\vartheta^{(3)} = 45°$ (see Fig. 11):



a) b)

Fig. 11 a) Free body diagram with $\vartheta^{(3)} = 45°$,
b) Force polygon

Now, the obtained results from variation are compiled in the following table:

No. of variation	ϑ [°]	Z_d [kN/m]	$\sum l_i$ [m]	$T_{m,d} = Z_d/\sum l_i$ [kN/m/m]
1	35	230	13.6	230/13.6=16.9
2	40	275	15.9	275/15.9=17.3
3	45	300	18.6	300/18.6=16.1

Thus, at $\vartheta^{(2)} = 40°$, the maximum value of the mean shear force per nail meter required for equilibrium in the least safe mechanism is found:

$$T_{m,d,max} = 17.3 \text{ kN/m/m.}$$

4th Calculation step: Determination of $T_{m,k,max}$

As the EC-7-1 does not contain a partial safety factor for nailed walls, the factor γ_N = 1.3 is recommended by the author. This factor is prescribed in the German DIN V 1054-100, which was issued in April, 1996, as a preliminary national standard (cf. ch. 5.2). The factor γ_N = 1.3 is slightly below the value recommended in the *Recommandation Clouterre 1991* (cf. Table 2 in ch. 3.4).
The characteristic mean shear force per nail meter is obtained using Equ. (9):

$$T_{m,k,max} = \gamma_N \cdot T_{m,d,max} = 1.3 \cdot 17.3 = 22.5 \text{ kN/m/m}$$

5th and final calculation step: Evaluation of the required *horizontal* nail spacing

Finally, one obtains the required *horizontal* nail spacing s_h [m] by the following equation:

$$s_h = \frac{T^{\#}_{m,k}\ [kN/m]}{T_{m,k,max}\ [kN/m/m]} = \frac{30}{22.5} = 1.35\ [m] \quad (10)$$

It should be mentioned here, that this $T_{m,d,max}$-method, used by the author since 1982, leads the designer to a unique and optimized solution. It is not a trial-by-error method based on a complete but guessed initial lay-out as many other methods available. In this sense, the British DOT Advice Note, HA 68/94, (Love, 1995) is very similar to the author's method.

The same solution, s_h = 1.35 m, can be easily obtained from design charts (see Appendix) using the specific nailing density μ. The design charts in the Appendix are valid for the limit equilibrium or design state. This means the input *and* output values are design values. All parameters in the design charts are normalized. The use of normalized, or dimensionless, values, say l/h (l: horizontal nail length at the bottom, h: slope height), or $q/(\gamma \cdot h)$ (q: load, γ: unit soil weight), helps considerably to reduce the number of required design charts.

The geometric parameters of the nailed wall, normalized nail length l/h = 0.6, inclination α = 10° and line through lower nail ends vertical (i.e. ρ = 0) lead the designer to the chart in the left column and second row (see Appendix).

With the design values, soil friction φ'_d = 29°, and normalized design load

$$q^*_d = q_d/(\gamma_d \cdot h) = 30/(18\ 10) = 0.17,$$

one obtains the design value of the specific nailing density μ_d = 0.87 from the selected chart. (The asterisk symbolizes q^*_d as a normalized magnitude). It is obvious that μ_d represents the maximum value required for equilibrium in the design state. Thus:

$$\mu_{d,max} \leftarrow \mu_d$$

Now, using the partial safety factor γ_N = 1.3 yields the respective characteristic value:

$$\mu_{k,max} = \gamma_N \cdot \mu_{d,max} = 1.3 \cdot 0.87 = 1.13$$

Finally, application of Equ. (4) leads to the required *horizontal* nail spacing s_h:

$$s_h = T^{\#}_{m,k}/(\gamma_d \cdot s_v \cdot \mu_{k,max}) = 30/(18 \cdot 1.1 \cdot 1.13) = 1.34\ m$$

One can see that the solution is identical to the one in Equ. (10). The difference is meaningless and comes from a slight inaccuracy due to the use of the design chart.

Design charts can be very expedient in practice. Very often a quick pre-design of a nailed slope project is needed to estimate the costs. A complete set of 36 design charts can be found in Gässler (1987) covering frequent geometric and static boundary conditions of nailed walls.

The next check required is the check of the steel bar in the nails. It is a check of a material property or dimension and is, according to EC 1, to associate with case 1B. The check shall fulfill the general requirement of all ECs (see ch. 5.1 of this lecture):

$$E_d \leq R_d$$

Generally, a set of partial safety factors different from Case 1C shall be used now for the *design* action in Case 1B (cf. Table 3, ch. 5). However, in this example the *design* nail force is directly given from the stability calculation in Case 1C, so that the partial safety factors in Table 3 need not be applied.

The maximum design value of the intersection nail force $N_{d,max}$ is found in the free body diagram of the least safe failure mechanism ($T_{m,d,max}$ mechanism) in Fig. 10a. One obtains $N_{d,max}$ according to Equ. (2) as follows:

$$N_{d,max} = T_{m,d,max} \cdot s_h \cdot l_{max} \qquad (2a)$$

with l_{max}: longest nail section beyond the slip plane.

Normally, as it is here, the longest nail section is to be found in the row at the bottom. Thus, one yields:

$$N_{d,max} = 17.3 \cdot 1.35 \cdot 5.6 = 131\ kN \qquad (11)$$

It is important to see now, that the nail force, having been a *resistant* force, becomes an *acting* force, when the material property (or the cross area) of the steel is checked:

$$E_d \leftarrow N_{d,max} \qquad (12)$$

The resistant force is determined by the material property, i. e. the tensile strength of the steel, and the cross section area of the bar. Commonly used nail diameters are given by Bruce & Jewell (1986/87), by Gässler (1990b) or by the French recommendations *Clouterre*.

In this case a steel bar of diameter d = 20 mm, and of normal quality is chosen with the characteristic tensile strength at the yield point:

f_{yk}= 500 N/mm².

The design value is derived from:

$$f_{yd} = \frac{f_{yk}}{\gamma_M} \quad (13)$$

with γ_M: partial safety factor for the material steel (here: concrete steel), given in EC 2

With γ_M = 1.15 the design value of the tensile strength is calculated:

$$f_{yd} = \frac{500}{1.15} = 435 \text{ N/mm}^2 \quad (14)$$

The material resistance R_d is obtained by:

$$R_d = f_{yd} \cdot A_S \quad (15)$$

with A_S: cross section area of the steel bar (A_s = 314 mm² for d = 20 mm)

In the given case R_d is calculated:

$$R_d = 435 \cdot 314 = 137000 \text{ N} = 137 \text{ kN}$$

Finally, the check of the steel bar in the nails can be verified as follows:

$$E_d = 131 \text{ kN} \leq R_d = 137 ✓$$

6.2 *Design example No. 2*

Now, consider a 10 m high nailed cutting slope in cohesive soil. There is no surcharge acting on the crest of the slope. On the uphill side, the natural slope rises with the inclination β = 10° (Fig. 12). The following data are given:

characteristic soil parameters: φ'_k = 29°
γ_k = 19 kN/m³
cohesion c'_k = 15 kN/m²

predesigned wall dimensions:
- wall inclination α = 10°
- constant nail length l = 6.0 m
- vertical nail spacing: s_v = 1.10 m
- horizontal nail spacing: s_h = 1.25 m
- line through lower nail ends: ρ = 10°

The required characteristic value of the mean shear force $T_{m,k}$ [30 kN/m] is to be determined for a quick pre-design (estimated design working life of the structure: > 50 years; cf. EC 1, Table 2.1: Design working life classification). The solution is to be achieved by means of design charts.

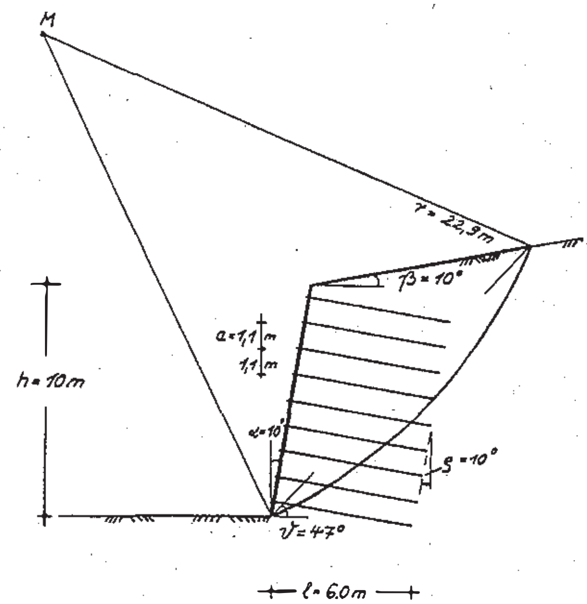


Fig. 12 Design example for a nailed cutting slope in cohesive soil (with unsafest slip circle)

1st Calculation step: Derivation of design values:

Design values of soil parameters:
$\varphi'_d = \arctan(\tan\varphi'_k/\gamma_\varphi) = \arctan(\tan 29°/1.25) = 24°$
$\gamma_k = \gamma_d$ = 19 kN/m³
$c'_d = c'_k / \gamma_{c'}$ = 15/1.6 = 9.4 kN/m²
Comment: For limit state case 1C: $\gamma_{c'}$ = 1.6 (see Table 2.1 in EC 7-1, or Table 4 in ch. 5.2 of this lecture)

2nd Calculation step: Normalizing of data for the design chart

Normalized cohesion is to be found by dividing c' by the unit soil weight and by the height of the slope (cf. Gässler, 1987):

$$c'^*_d = c'_d / (\gamma_d \cdot h) = 9.4/(19 \cdot 10) = 0.05 \text{ [-]}$$

3rd Calculation step: Application of the design chart

For the given geometric parameters the adequate design chart in Fig. 13 was selected from a set of charts developed by the author for cohesive soils (Gässler, 1987).

The basis of the limit state design chart in Fig. 13 is the slip circle (rotation mechanism of one rigid block, ROT-I) as shown in Fig. 6b. Very quickly the specific nailing density $\mu_d = \mu_{d,max}$ = 0.58 is obtained from the chart.

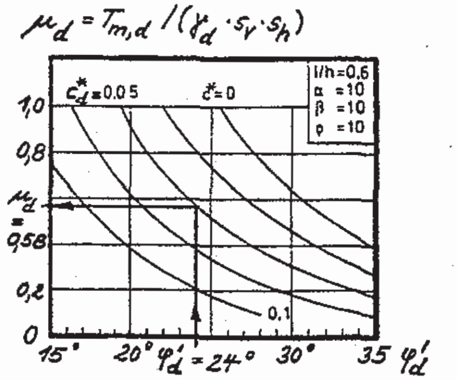


Fig. 13 Design chart for a nailed slope with l/h=0.6, $\alpha = 10°$, $\beta = 10°$, $\rho = 10°$ in cohesive soil

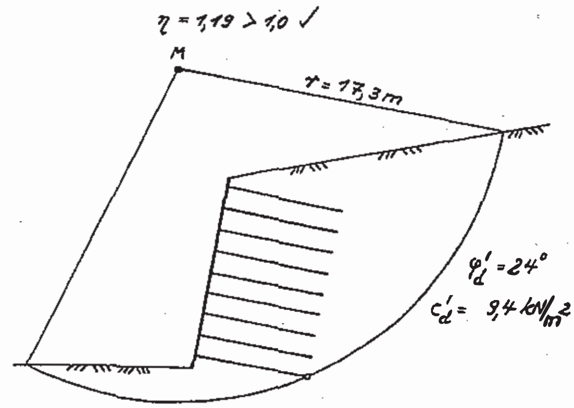


Fig. 14 External stability check: unsafest slip circle

Now, using the partial safety factor $\gamma_N = 1.3$ yields the respective characteristic value:

$$\mu_{k,max} = \gamma_N \cdot \mu_{d,max} = 1.3 \cdot 0.58 = 0.75$$

4th Calculation step: Determination of $T_{m,k,max}$

The characteristic mean shear force per nail meter is obtained using Equ. (4):

$$T_{m,k,max} = \mu_{k,max} \cdot \gamma_k \cdot s_v \cdot s_h$$

$$T_{m,k,max} = 0.75 \cdot 19 \cdot 1.1 \cdot 1.25 = 20 \text{ kN/m/m.}$$

Herewith, the solution is found. However, the design chart does not give any information on the unsafest slip circle. Its geometric parameters can only be achieved from the computer program which the design charts for cohesive soils have been based on (cf. ch. 4.1 and Fig. 6b). Fig. 12 shows the unsafest clip circle with the chord angle $\vartheta = 47°$ and the radius r = 22.9 m.

In the given case it is also recommended to check the external stability. Fig. 14 shows the least safe external slip circle that was found using a conventional computer program. Naturally, following EC 7, the input data were *design* values, say, $\varphi'_d = 24°$ and $c'_d = 9.4$ kN/m². (As no nails are intersected by the check of external stability, no design value for the nails was put in the program.)

Fulfilling the requirements of EC 1 and EC 7, the difference between the resisting moments M_{Rd} and the driving moments M_{Fd} has to be greater or equal zero in the *design* state. The same reqirement can be formulated as the following ratio:

$$\frac{M_{Rd}}{M_{Fd}} \geq 1 \qquad (12)$$

The ratio, resisting moments over driving moments, is 1.19 in Fig. 14. Consequently the respective slip circle in Fig. 14 is safer than the slip circle in Fig. 12, of which the ratio M_{Rd}/M_{Fd} is exactly 1. As the external stability is checked using *design* values, the minimal ratio in Equ. (12) does not need to reach values in the range of 1.5, as it is usually required for *overall* safety factors applied to conventional *calculation* values φ' and c'.

Further stability checks need not be carried out. For example, the external stability checks of sliding or bearing capacity do not govern the design in normal cases. Of course, the justification of the facing is required. However, this cannot not be considered in the scope of this lecture.

7. CONCLUSIONS

The use of the new European Codes, *EC 1 and EC 7* in particular, yields a safe and practicable design of nailed walls and cutting slopes. Thinking in terms of the design limit state and application of several partial safety factors instead of only one global safety factor will be new and unusual for many practising engineers, but in the author's opinion, the Eurocodes can be easily applied to soil nailing without major problems. However, one must not forget that *EC 7- Part 1* in particular is in a pre-liminary state and is not yet completely applicable in practice to all types of retaining structures. Concerning soil nailing, two examples have been given which demonstrate that the partial safety factors approach adopted by the Eurocodes provides an easy and expedient design for soil nailed structures.

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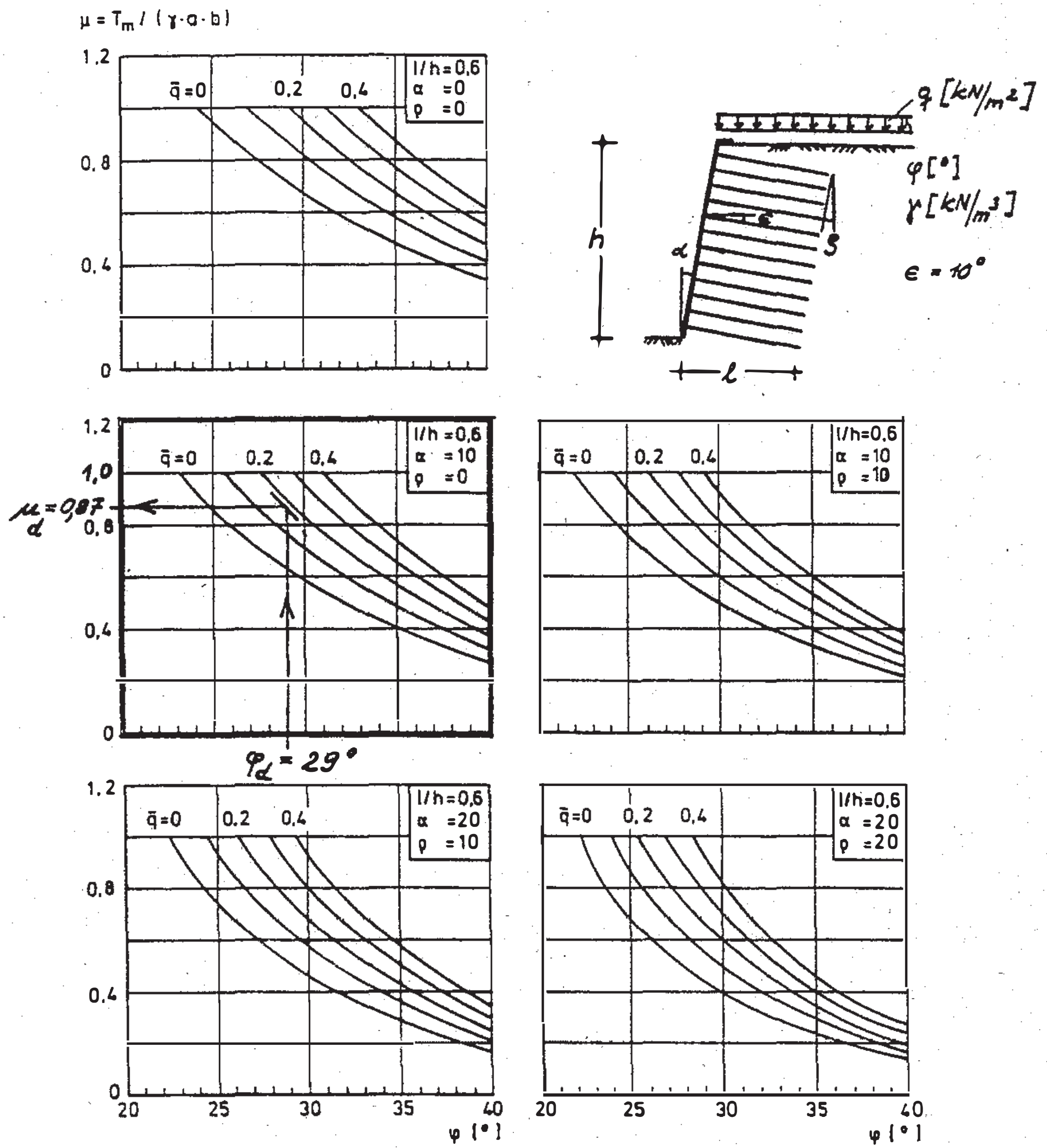

Design charts for nailed walls and slopes in non-cohesive soil with nail length l/h = 0.6 and with constant surcharge q* = q/(γ ·h) acting on the horizontal crest