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Soil-nailing – Design and application to modern and ancient retaining walls

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ABSTRACT: Two practical projects of nailed retaining walls performed in West–Germany are presented. The first one is a steep cut; forces at the nail heads and displacements of the wall surface were measured during a period of about three years to investigate the influence of freezing. The second one is an ancient masonry retaining wall. A simplified limit state equation, based upon a two-body translatory mechanism and ground bearing capacity is formulated. By means of soil nailing the safety and reliability of the old structure increases to a sufficient level. A kind of observational method is applied.

1 INTRODUCTION

The technique of soil nailing has a wide range of applications for temporary and permanent structures. It is mainly used for constructing excavations and stabilizing slopes (Gässler & Gudehus 1981; Stocker et. al. 1979). Numerous projects have been carried out (Bruce &Jewell 1986). The theoretical investigations of the relevant failure mechanisms are summarized by Gässler (1987).

This paper deals first with the influence of frost. Measurement results of nail forces and displacements are presented. Second, the stability analysis of nailed masonry retaining walls on the basis of a consistent failure model is employed. The relevant failure mechanism is confirmed by model tests. By means of probability theory the risk of failure of the structure can be calculated.

2 STEEP CUT

2.1 General description

In the course of the construction of a new road a cut with a height of about 7m and an average inclination of 70° had to be executed. To stabilize the cut, soil-nailing was used with shotcrete and earth terraces. (Fig.1). The terraces were filled up with top soil and planted to cover the shotcrete face.

In the area of the cut weathered debris of Keuper marl (Gipskeuper) is covered by a layer of loam up to several meters thickness. Ground water was not detected. Following our experience, an angle of internal friction of $\varphi'=35°$ could be assumed. The unit weight amounts to 20 kN/m^3. For the stability analysis, a two-body translation mechanism with partial safety factors for the relevant parameters was used, without further investigations (Gässler 1987). The partial safety factors are: $\gamma_\varphi=1.20$ for the angle of internal friction, $\gamma_P=1.30$ for the live load and $\gamma_T=1.6$ for the pull-out resistance applied to the respective mean values. With these parameters, a failure probability of $p_f < 10^{-6}$ is obtained following the Level II approach (Gässler & Gudehus 1983, Hasofer & Lind 1974).

Loading tests were carried starting after the end of the construction in order to determine the pull-out resistance of the soil-nail system. The necessary values to prove the safety of the wall were reached. Loading was halted before the limit state was reached. Therefore a prediction of the precise limits value can not be made.

2.2 Measurements

The main attention was on the developement of forces and displacement after execution of the cut. Especially, the effects of repeated freeze-thaw changes were to be observed.

In a part of the nailed wall, the following instruments

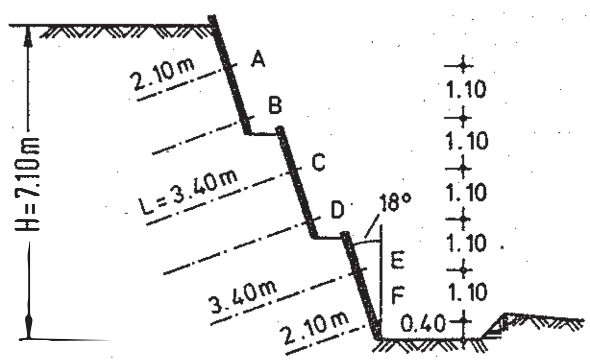
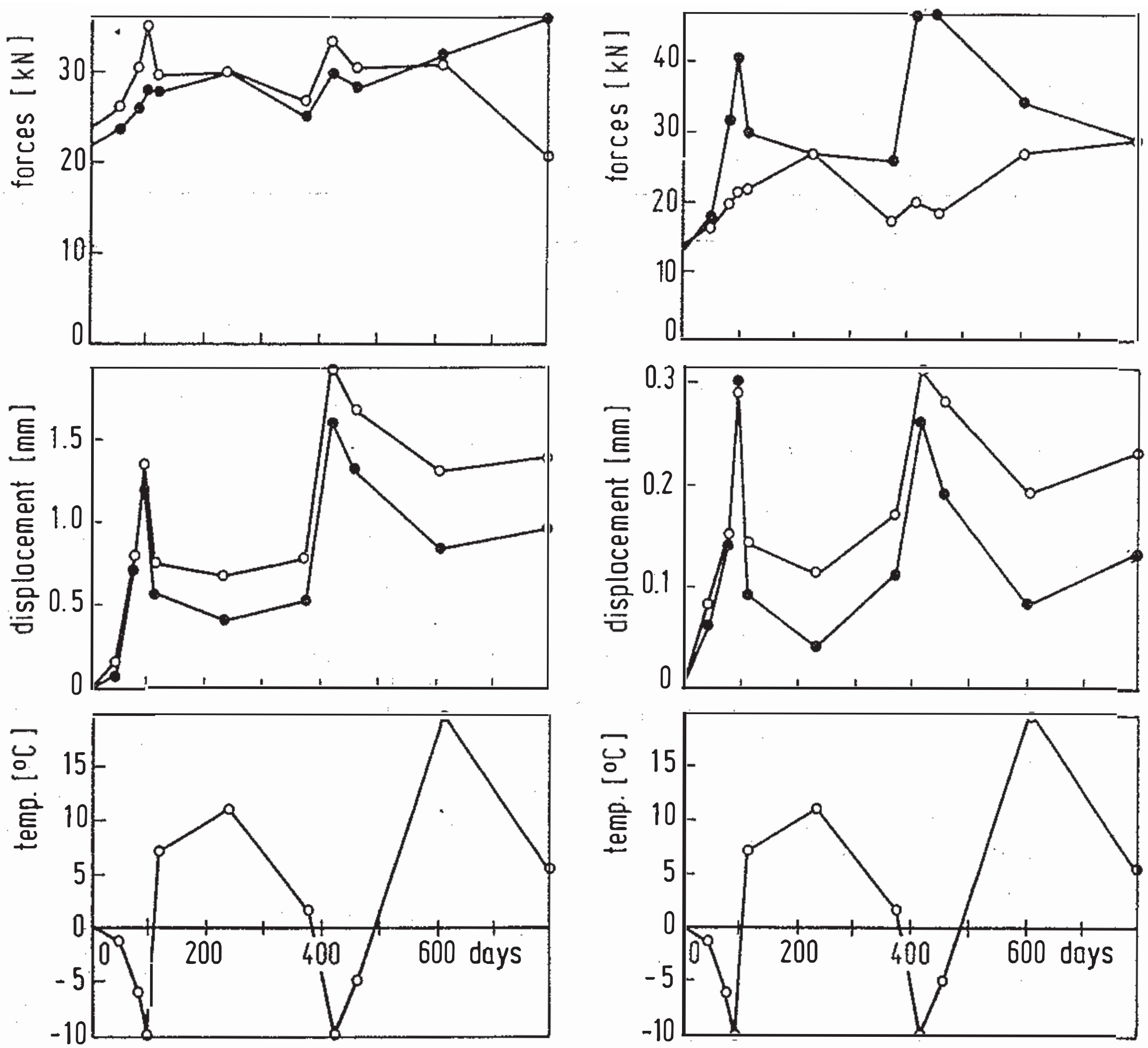


Fig.1 Cross section of cut



a) layer D

b) layer E

Fig.2: Temperatures, forces and displacements

were installed:

- 4 force meters to measure the nail-head forces,
- 4 extensiometers (length: 1.5m) to measure the displacement of the shotcrete wall (Fig.1); two of them in layer D and two in layer E with a horizontal distance of about 2m.

After execution of the cut the first (zero) measurement was made. During a time span of about 2.5 years 11 measurements were taken. Figs.2a,b contain the results.

2.3 Interpretation of the results

It can be seen that frost has a remarkable influence on the nail-head forces and on the displacement of the wall; both increase with frost-duration.

The nail-head forces amount, with low temperatures, to about twice the values of the zero measurement, on both the upper and the lower row of points of measurement. The scatter of results of measurements in the upper layer is remarkably bigger than in the upper layers, which indicates a heterogeneous structure of the soil in this area.

The forces built up in the first frost period decreased to a value greater than the one of the zero measurement. During the second frost period, with about the same low temperatures,they increased again to the values reached before.

The displacement of the wall showed the same changes with temperature; they also increase with frost duration.

A final evaluation of the development of forces and displacements is not jet possible. It is planned to make more measurements, twice a year to the minimum, once during the frost period and once in summer.

The measurements taken at the foot of the wall (layer E) show greater values for displacement than the ones taken in the upper part. As a result of the construction technique, the nails take up smaller tensile forces. Because of the creep and settlement behaviour of the soil they asymtotically reach the values calculated in the final state.

3 ANCIENT RETAINING WALLS

3.1 General description

A lot of old masonry gravity-retaining walls in West–Germany , made out of natural stone with a height of 2m up to more than 8m, are not safe enough. Cracks and bulging witness the inadequate bearing capacity of the wall foundation. The removal of the unsafe constructions would not be adequate. A stabilisation was repeatedly carried out using soil nailing in order to reduce the active earth pressure.

In contrast to conventional soil nailing the ground under the wall can -according to its bearing capacity- still carry an amount of the earth pressure. A mechanism was found to describe this nail-gravity-wall system in the limit state. A statically and kinematically consistent failure model, correctly describing the system in limit state is employed.

A concrete face of the wall could not be applied out of esthetical reasons (preservation of a historical monument). The nail heads, therefore, had to be hidden. This paper shows how to hide the load bearing nail-heads behind the wall.

The stability analysis of the nail-retaining wall-system was completed by means of statistical methods.

3.2 Stability analysis

The basic failure mechanisms for nailed walls are presented in detail by Gässler (1987). The limit state equation can be simplified using a two-body translation mechanism. In our case, the necessary outer concrete face is replaced by the existing gravity wall. Depending on the width b at the foot, the wall can carry a certain part of the earth pressure. Recent investigations of more than 20 ancient retaining walls has shown that they have very shallow foundations, that their back side is nearly vertical, and that they have an almost constant thickness. Only walls of this type are investigated here. Fig. 3 shows the failure mechanism relevant after nailing.

Equation of limit state

The global system can be subdivieded into two bodies. The forces acting upon the retainig wall and the nailed body are shown in Fig.4 a,b. After some algebraic transformations an expression for the earth pressure is

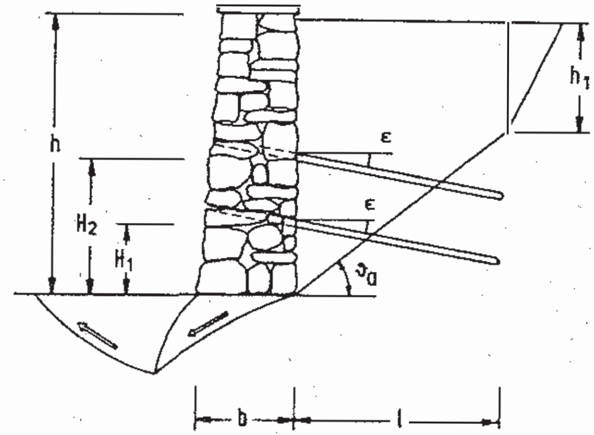


Fig.3 Nailed retaining wall with failure mechanism

obtained:

$$E = \frac{\tan(\vartheta_a - \varphi)}{\sin\delta + \cos\delta} \cdot \Big[\frac{\sum T}{a} \sin\epsilon + E_a \sin\varphi + W_1 - C \frac{\sin\vartheta_a l}{\tan(\vartheta_a - \varphi)} \big(\sum T \cot\epsilon + E_a \cos\varphi - C\cos\vartheta_a\big)\Big] \tag{1}$$

$\delta - \varphi$ can be assumed because the back side of the wall is rough and a slip plane developes there.
Herein $W_1 = 1/2\gamma l(2h - l\tan\vartheta_a)$ denotes the weight of the nailed body, E_a the earth pressure acting on the vertical intermediate slip surface with $h_1 = h - l\tan\vartheta_a$ (for $E_a \leq 0$, it is substituted by $E_a = 0$), and the resultant cohesion $C = cl/\cos\vartheta_a$. ϑ_a is the inclination of the slip surface with the maximum of E, found by variation of the slip surface inclination.

$\sum T = \sum_{i=1}^{2} \frac{l - H_i}{\tan\epsilon + \tan\vartheta_a} \frac{1}{a} \frac{1}{\cos\vartheta_a}$ the resultant nail forces per unit length (with a: horizontal distance of nails; ε: inclination of the nails).

The vertical forces at the bottom of the wall are limited by the bearing capacity of the soil.

$$V_b = b' \cdot (c \cdot N_c \cdot i_c + \gamma \cdot b' \cdot N_\gamma \cdot i_\gamma) \tag{2}$$

with the bearing capacity factors values (DIN 4017 (1979))

$$N_q = e^{\pi \cdot \tan\varphi} \cdot \tan^2(45 + \varphi/2)$$

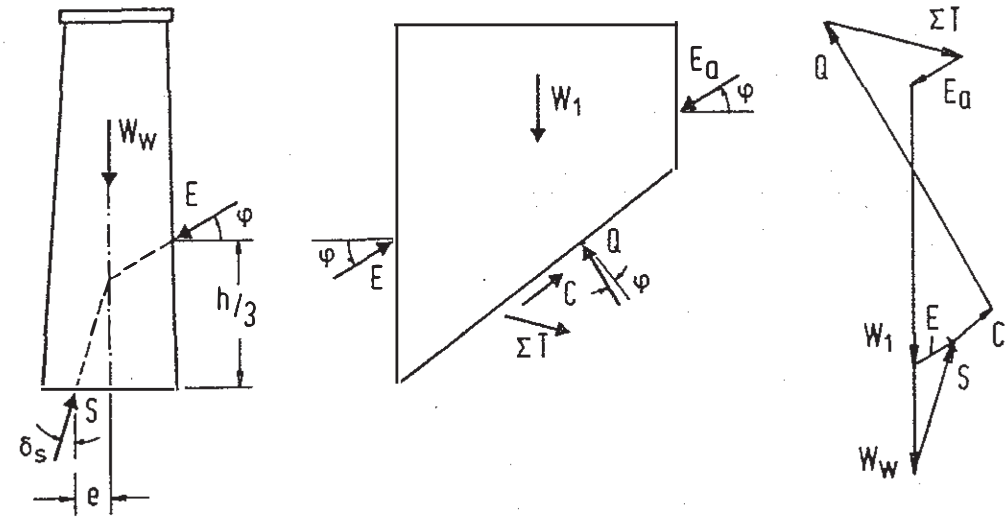
$$N_\gamma = (N_q - 1)\tan\varphi \qquad N_c = (N_q - 1) \cdot \cot\varphi$$

$$i_\gamma = \left(1 - \frac{\sum H}{\sum V + b' \cdot c \cot\varphi}\right)^3$$

$$i_q = \left(1 - 0.7 \cdot \frac{\sum H}{\sum V + b' \cdot c \cot\varphi}\right)^3$$

$$i_c = i_q - \frac{1 - i_q}{N_q - 1}$$

Herein, $\sum H = E\cos\varphi$: the resultant horizontal forces $\sum V = W_W + E\sin\varphi$: the resultant vertical forces and $W_W = \gamma_W bh$,the weight of the wall.



a) acting forces (wall) b) acting forces (nailed body) c) force polygon

Fig.4 System

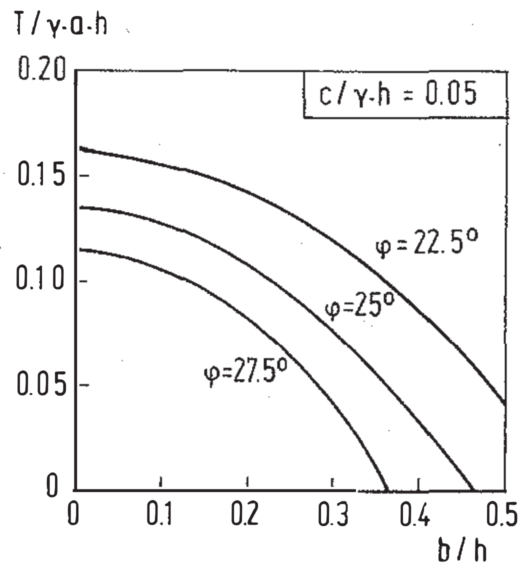


Fig.5 Design chart for $H_1/h=0.25$; $H_2/h=0.45$; $\gamma_W/\gamma=1.1$; $c/\gamma h=0.05$; $l/h=0.8$

The reduced width is calculated as

$b' = b - 2 \cdot e$

with the eccentricity $e = \frac{\sum M}{\sum V} = \frac{\sum H \cdot h/3 - E \cdot \sin\varphi \cdot b/2}{\sum V}$.

The earth-pressure resultant is assumed to have the height of h/3 above the level of foundation

Based on this failure mechanism, the limit state equation can now be formulated as:

$$V_b - \sum V = 0 \tag{3}$$

It can be represented in diagrams with dimensionless parameters l/h, b/h, $c/\gamma h$, H_1/h, H_2/h , $T/\gamma ah$, γ_W/γ and φ and can be used for stability analysis.

3.3 Construction of nail heads

Nail heads can be anchored in front or behind the wall. Because of optical reasons and monument preservation, nail heads in front of the wall are not to be used. Anchoring in the body of the wall is problematic. The quality of masonry is not known and cavities are -in spite of injection- still present.

Thus a construction detail was designed to fit between wall and backfill. Using a water-jet injection the soil behind the wall is liquified within a diameter of 0.7 m, and sucked off. The cavity thus created is filled with cement grout after the placement of the nail. Experiments with this detail show a sufficient bearing capacity, much higher than the ones obtained with anchoring in the masonry of the wall. Moreover, the resulting displacements are much smaller.

3.4 Model Tests

Model tests were carried out according to the laws of similarity mechanics. Model walls were built out of hewn sandstone without mortar. The pull-out resistance of the model nails and their bending rigidity were correctly represented by using 1mm thick pipe cleaners. The length of the model nails was 10cm, and they were built in two layers. The nail head details were represented by thin rectangular sheets of wood. A contact to the wall was prevented. Dense dry sand (γ=16.3 kN/m^3), φ ~39°) was pluviated with thin black marker layers of 4cm distance.

Other tests showed that the failure mechanisms are disturbed along the side walls of the model bin. Therefore, the earth bodies were cut vertically in the center after failure. This procedure was enabled by submerging and draining and thus producing sufficient capillary cohesion.

Model tests for unreinforced gravity walls showed that the wall should be safe up to a height of about 20cm. Knowing the pull-out resistance of the model nails and in consideration of the friction along the side walls the limit height of the reinforced wall at the point of collapse was predicted to 26cm using the limit state equation(3).

The sandstone wall was built up to the limit hight; collapse was triggered by pluviating sand behind the wall. Sufficient kinematic liberty was guaranteed, and no possible mechanism was favored beforehand. A combined mechanism as in Fig.6 with $\vartheta = 54°$ was observed. The calculated limit state mechanism is the one of Fig.4a with $\vartheta = 52.5°$ and is nearly the same as the one observed. Therefore the limit state function is sufficiently verified; especially the assumptions for the inclination and the acting height of the resultant earth pressure are confirmed.

3.5 Probability of failure

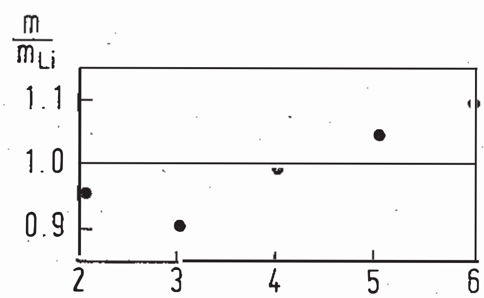
In the state limit function (equ.3) the quantities φ, c and T are scattering; they are so-called basic variables in the sense of the statistic safety theory (Hashofer and Lind 1974). The distributions of the angle of internal friction and the cohesion c are taken here from Walz and Genske (1987).They are for φ: log–normal,truncated at X_{φ_L}, coefficient of variation V_φ=0.05–0.075 and for c log–normal, coefficient of variation V_c=0.2. As shown by Gässler and Gudehus(1983), the unit weight of soil γ can be considered as non-scattering; likewise the unit weight of the wall γ_W. Pull–out tests to determine the distribution, the mean values and the coefficient of variation of the static friction nail/soil were carried out in different fields. The distribution can be fitted with a log–normal distribution. Typical test results of one field in silty soil are given in Fig. 7 a,b. It was found that the mean values were nearly independent of the nail length (Fig.7a). The coefficient of variation decreases with increasing length (Fig. 7b) as Gudehus (1987) proposes. The mean value of 5 fields (more than 100 tests) in silty soil is $T_0 = 34$kN/m with a standard deviation $\sigma_{T_0} = 2.7$ kN/m and, therefore, a coefficient of variation $V_{T_0} \approx 8\%$. In the sense of the Bayesian approach this can be taken as prior information. This prior information can be combined with n observed data of pull–out test in a real project(sample mean $\bar{T}_x$ and sample standard deviation σ_{T_x}) to estimate the mean value m_T(Ang and Tang(1975)):

$$m_T = \frac{(\sigma^2_{T_x}/n)T_0 + \sigma^2_{T_0}\bar{T}_x}{(\sigma^2_{T_x}/n) + \sigma_{T_0}} \qquad (4).$$

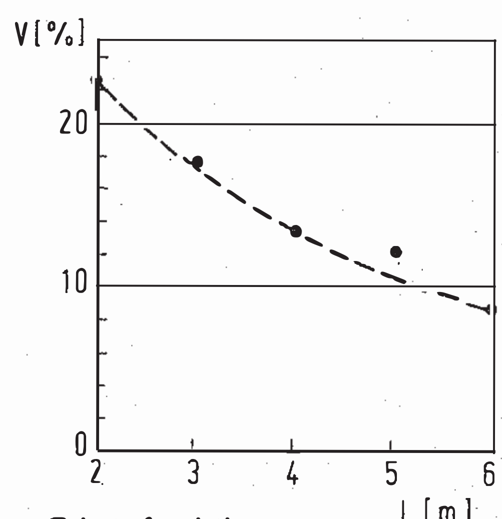
m_T is an average of the prior mean T_0 and the sample mean $\bar{T}_x$, weighted inversily by the respective variances.

The probability of failure, p_f, is estimated via the safety index β. In the Level II approach (Hasofer and Lind 1974) probability distributions are replaced by Gaussian ones. The limit state equation is linearized in the vicinity of the so-called design point with the

Fig.6 : Model test failure mechanism



a) mean values



b) coefficient of variation

Fig.7 Results of pull-out tests

design values X_i^*. β is the minimal distance of this point from the limit state function (Smith 1981).

The design values depend on the safety index β, the respective distributions, coefficients of variation, weight coefficient α_i and the limit state function. With the simplifying assumption that the weight coefficients α_i for the three basic variables φ, c and T are equal to $1/\sqrt{3}$ (Gudehus 1987), the design values can be derived for log–normal distributions as $X_i^* = m_i/\gamma_i$ with the partial safety factors

$$\gamma_i = \frac{\sqrt{1+V_i^2}}{\exp(-\beta\sqrt{3}\sqrt{\ln(1+V_i^2)})} \tag{5}$$

respectively for at X_{L_i} truncated log-normal basic variables

$$\tilde{V} = \frac{V_i}{1 - X_{L_i}/m_i} \tag{6}$$

$$\eta_i = \frac{\sqrt{1+\tilde{V}_i^{\,2}}}{\exp(-\beta/\sqrt{3}\sqrt{\ln(1+\tilde{V}_i^2)})} \tag{7}$$

$$\gamma_i = \frac{\eta_i}{1 + X_{L_i}/m_i(\eta_i - 1)} \tag{8}$$

3.6 Design example

A more than 200 year old masonry retaining wall with a height of 8m and a nearly constant width of 1.6m showed a bulged profile. Cracks and forward movements were observed. Bearing capacity calculations by the Level II approach showed a safety index β of 0.5 (i.e. probability of failure $p_f \approx 0.3$), whereas a safety index β =4.7 (i.e. $p_f \approx 10^{-6}$) is required. In order to increase the safety to the sufficient level, the soil behind the wall was reinforced by nails. Three pull-out tests were performed.

Data:
walls: h=8m, b=1.6m, b/h=0.2, $\gamma_W = 22\text{kN/m}^3$
silty soils: $m_\varphi = 30.5°, V_\varphi = 0.05, \varphi_L = 20°$,
$m_c = 16kN/m^2, V_c = 0.20, \gamma = 19kN/m^3$
nails: sample mean $\bar{T}_x = 30$ kN/m, standard deviation $\sigma_{T_x} = 3.6$ kN/m, n=3, L=6m, $\epsilon = 10°$, H_1=2m, H_2=3.6m

The horizontal nail distance a is to be determined. The design value of the cohesion is obtained (Eqn.5) as $c^* = 16/1.75 = 9.2\text{ kN/m}^2$, for the angle of internal friction it can be found (Eqn.6–8) $\tilde{V}_\varphi = 0.145, \eta_\varphi = 1.5, \gamma_\varphi = 1.13$ and $\varphi^* = 30.5/1.13 = 27°$.

With the prior information $T_0 = 34$ kN/m, $\sigma_{T_0} = 2.7$kN/m and the observed data one obtained from Eqn.4 and 5 $m_T = 31.5$ kN/m, $\gamma_T = 1.4$ and $T^* = 31.5/1.4 = 22.5$ kN/m (mean length behind the slip surface ca. 4m, $V_T = 0.12$). The limit state function (Eqn. 3) requires a pull-out resistance of $T_1 = 13.3$kN/m per unit wall length. With the existing allowable value of $T^* = 22.5$ kN/m, the horizontal distance a has to be smaller than $T^*/T_1 = 22.5/13.3 = 1.67$ m. By means of the design chart (Fig. 5) one easily obtains, with $c^*/(\gamma h)$=0.06 and $\varphi = 27°$, $T/(\gamma ah) \approx 0.09$; a is given immediately by $a = T^*/(0.09 \cdot 20 \cdot 8) = 1.56$ m. The results of a Level II approach calculation for a choosen horizontal distance a=1.50m yield the sensitivity coefficient α_φ=0.545, α_c=0.694, φ_T=0.479 and the safety index β=5.2.

4 CONCLUSIONS

The conventional method of soil nailing requires a closed shotcrete face. The arrangement of berms in a steep cut leads to a stepped wall with open planes. Taking the frost action into account this wall is as safe as a conventional permanent nailed wall; the outer concrete face can be hidden by planting the earth terraces.

A failure model based on the kinematic failure mechanisme of rigid bodies can be developed for nailed masonry retaining walls. The translation mechanism of two bodies combined with the ground bearing capacity is found as the relevant failure mode. It is found by variation of slip surfaces depending on all input data. The results of theoretical investigations can be verified by small model tests. By means of the new statistic–probabilistic theory partial safety factors for the relevant basic variables can be derived. Observed data of field tests combined with prior information yield a reliable structure. The proposed design procedure is simple and supported by more than a dozen successful applications.

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