HENRY

Hydraulic Engineering Repository

Ein Service der Bundesanstalt für Wasserbau

Conference Paper, Published Version

Schuppener, Bernd

Living reinforced soil - an ecological method to stabilise slopes

Verfügbar unter/Available at: https://hdl.handle.net/20.500.11970/100753

LIVING REINFORCED SOIL - AN ECOLOGICAL METHOD TO STABILISE SLOPES

Bernd Schuppener

Abstract: *Living Reinforced Soil* is a method of stabilising slopes by means of hardwood cuttings and/or hardwood whips by creating a retaining structure using plant material to reinforce soil. Such retaining structures ensure that steep slopes remain stable. The twigs and branches that act as a reinforcement are taken from plants capable of growing adventive roots – usually willows; they do not rot, but remain alive due to development of the roots, thus ensuring the durability of the structure. In spring, the parts of the plants growing above ground produce new foliage that not only protects the slope against erosion due to wind and precipitation but also prevents desiccation of the soil. Model tests and test slopes have been carried out to determine the structural performance of plants. These investigations have demonstrated that it is not the strength of the plant material, but the pull-out resistance of the plants and the strength of the bond between the plants and the soil that govern slope design, for the design of slope stabilisation using plants. Two calculation models are being investigated: a rigid body failure mechanism with a straight failure surface and a failure mechanism with two sliding wedges. The partial safety factor concept is applied when deriving the design formulae used to determine the required number, length and thickness of the plants to be installed in slope stabilisation. The result is a soil mechanical design method verified by tests that enables the stabilising effect of plants to be taken into account in slope design.

1. Introduction

The method of stabilising slopes by means of hardwood cuttings and/or hardwood whips (SCHIECHTL, 1987), referred to below as *Living Reinforced Earth*, consists of creating a retaining structure by using plant material to bind soil together (see Fig. 1). Such retaining structures ensure that steep slopes remain stable. The twigs and branches placed in the soil are taken from plants capable of growing adventive roots – usually willows; they do not rot, but remain alive due to development of the roots, thus ensuring the durability of the structure. In spring, the parts of the plants growing above ground produce new foliage that not only protects the slope against erosion due to wind and precipitation but also prevents desiccation of the soil.

The first approach to taking the role of plants into account in slope stabilisation used Coulomb's friction law to describe how plants reinforce soil (SCHUPPENER, 1994). Since then, a number of model tests and field trials have been carried out to determine structural performance. The tests and trials, which are described below, demonstrate that the bond strength between the plants and the soil can be described by a constant value with a sufficient degree of accuracy. This formed the basis of a design model that has now been developed. It enables stable slopes to be designed by calculating the number, length and thickness of the plants required and by applying the partial safety factor concept set out in the latest version of DIN 1054 (2005).

Dr.-Ing. Bernd Schuppener, and Research Institute, Karlsruhe, Germany. Phone: 004972197264830, E-mail:

Federal Waterways Engineering Kussmaulstrasse 17, 76187 004972197263800, Fax: bernd.schuppener@baw.de.
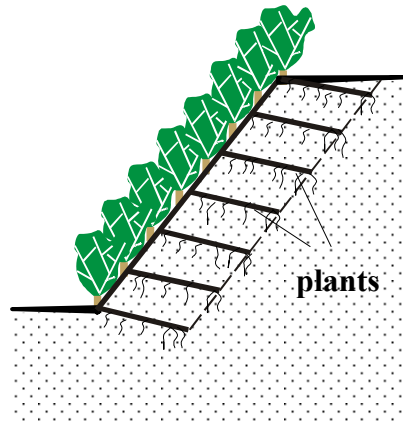
Figure 1: Schematic drawing of a slope stabilisation by Living Reinforced Earth.

2. Structural performance of slopes stabilised by plants

The following stability analyses are required for retaining structures:
- verification of external stability and
- verification of internal stability.

Verification of external stability involves demonstrating that the entire slope incorporating the retaining structure is stable, i.e., that overall stability is ensured. The aim of the internal stability analysis is to verify that the retaining structure possesses the degree of strength required for it to resist the internal stresses occurring as a result of the action of external forces and own self weight. Such analyses form the basis of the design of *Living Reinforced Earth* retaining structures and enable the number, length and thickness of the plants required to be determined.

The loss of internal stability will result in one of two different types of failure (see Figure 2). In the first of these, the tensile strength of the plants is no longer sufficient. As a result, the plants tear at or near the failure surface and are shorn off. This type of failure can occur when the plants have strong root growth, and there is therefore a very good bond strength between the plants and the soil. However, the plants may also be pulled out of the soil when a slope fails. The latter type of failure can occur shortly after the completion of slope stabilisation measures when the plants have not yet had the chance to grow roots, and a good bond strength between the plants and the soil has therefore not yet developed. It is this situation after construction of the slope that governs its stability. The most important parameter for a design model is therefore reliable data on the pull-out resistance of the plants.
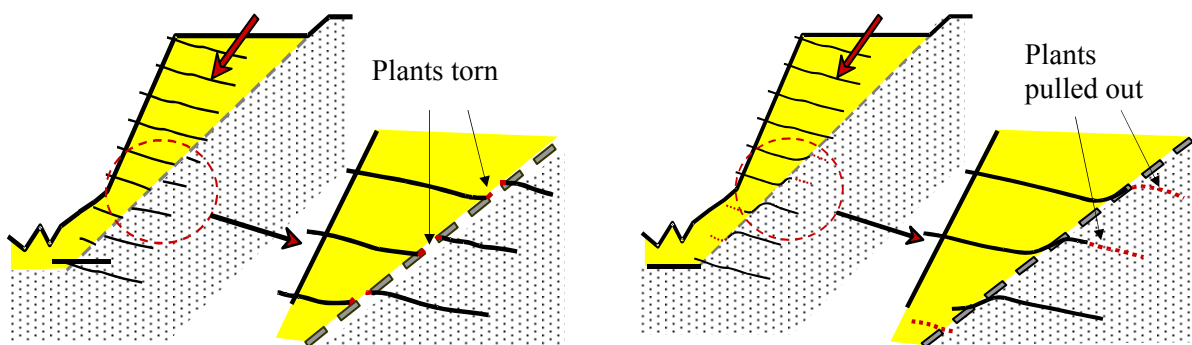
Figure 2: Failure mechanisms in a slope stabilised by plants
when the plants are torn or pulled out

3. Tests to check the design model

Since early 1995, research aimed specifically at determining pull-out resistance has been carried out in Berlin by the Federal Waterways and Engineering and Research Institute (BAW). The investigations covered pull-out tests in a test box in a laboratory and on trial slopes in order to study the growth of the plants and the development of their strength when used as reinforcing elements.

The test set-up used in the pull-out tests consists of a steel box filled with layers of sand in which a plant is placed (see Figure 3). There is a pressure pillow under the steel lid of the box that enables a precisely defined normal stress to be exerted by air pressure on the soil and thus also on the upper surface area of the plant. The ends of the plant protrude from the box. One end of the plant is subjected to a tensile force by means of a system of gears via a clamping device and a wire cable. The displacement and force are then measured. This test set-up corresponds more or less to that specified in the draft standard DIN EN 13738 (2000) which sets out a laboratory method of determining the resistance of geotextiles to being pulled out of soil using a pull-out box.
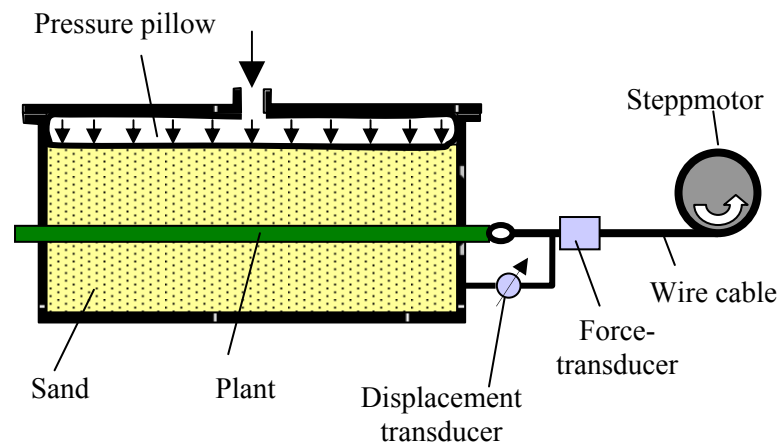
Figure 3: Test set-up for pull-out tests

In addition to the pull-out tests performed in the laboratory, trial slopes incorporating plants were constructed on the BAW´s site in Berlin, where the soil consists of a slightly silty sand with fine gravel with a Proctor density (D_{Pr}) of around 92%. The slopes were between 2.50 m and 3.0 m in height, with inclinations β between 45° and 60°. Plants with a diameter of around 2 cm and around 2.0 m in length were placed at an inclination $\alpha = 10°$ in horizontal rows 0.5 m apart. In addition to exploring matters relating to the vegetation, the aim of constructing the trial slopes was to investigate the following soil mechanical questions:

–　How close together and how long should the plants be so that they do not lose their vitality or even die over the long-term?
–　How does the resistance of the plants develop over a number of years?

Pull-out tests on the plants placed in the soil were therefore performed immediately after the construction of the first trial slope in May 1995. The tests were very similar to those

carried out on piles or anchors in foundation engineering, except that the equipment used was easier to handle and smaller due to the considerably lower forces involved. In the tensile tests conducted on the trial slope it was also possible to pull the 2 m long plants out of the soil. Prior to development of the roots the governing factor is therefore the strength of the bond between the soil and the plants, not the tensile strength of the wood.

The results of the first pull-out tests and those of the model tests are shown in Figure 4, where the maximum available bond stress τ_f, which is the bond strength between the soil and the plants, is plotted against the mean normal stress σ acting on the surface area of a plant. There are three main results:

– The increase in the bond strength τ_f from its initial value is proportional to that of the normal stress σ.

– An increase in the Proctor density (D_{Pr}) of the soil is accompanied by an increase in the bond strength τ_f

– There is a good degree of correlation between the bond strength τ_f of plants without root development immediately after construction of the trial slope (open circles) and the results of the model tests in the sand box.

Further pull-out tests were carried out after the first vegetation period. It can be seen very clearly that the bond strength of the plants τ_f had increased due to root development. Figure 4 shows only the results of the 6 tests in which it was possible to pull the plant out of the soil. In four of the tests, the wood tore directly behind the point at which the force was applied. Due to root development, the bond strength that could be resisted in this case had increased to a point at which the tensile strength of the wood became the factor governing the bearing capacity.
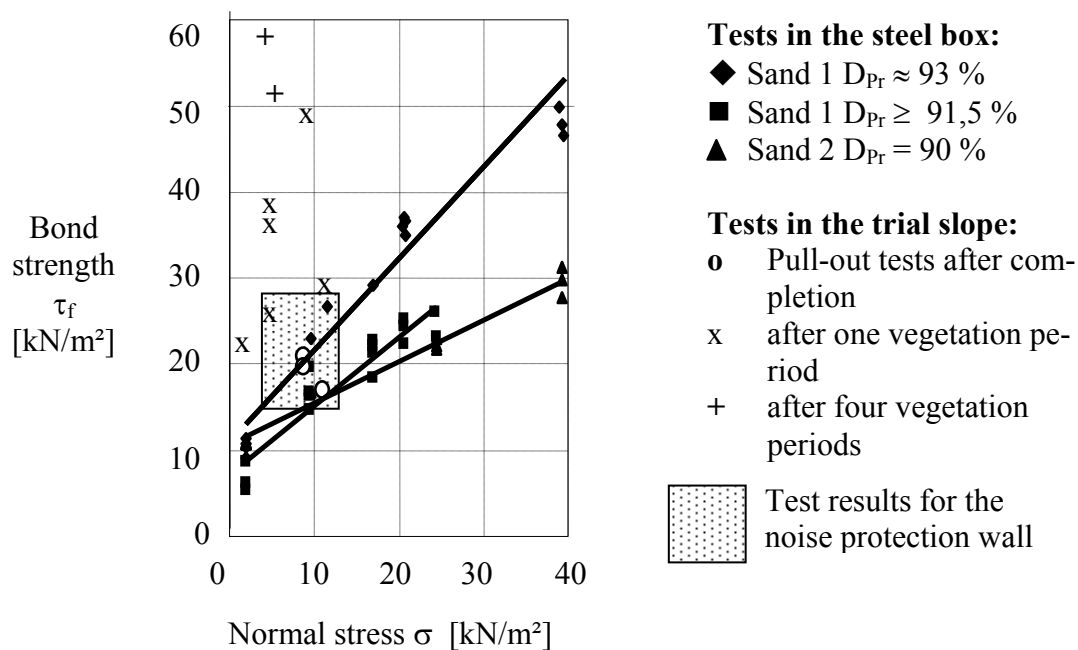

Figure 4: Results of the tests in the sand box and those on the trial slope

A further 7 pull-out tests were carried out in the summer of 1998 after 4 vegetation periods. It was possible to pull out only the two plants located at the top edge of the slope with a soil covering of only 0.5 m. Thus it can be seen that the maximum available bond stress had again increased quite considerably.

The increase in the bearing capacity of the plants as the result of root development during the first vegetation periods is an extremely useful reserve in case some of the plants that are installed should die over the course of time, as it enables the remaining plants to compensate for the loss of the dead ones.

Systematic botanical investigations were carried out in addition to the pull-out tests. Less than 10% of the plants died in spite of the fact that they were not tended. A significant result as regards the function of *Living Reinforced Earth* as a retaining structure was the root development over the entire length of the plants, i.e., over a length of around 2.0 m (SCHUP-PENER, B. & HOFFMANN, J., 1999). This ensures that the entire length of each plant survives.

Since then, the *Living Reinforced Earth* method has been applied in soil engineering projects in practice, for instance, on the A 113 motorway near Berlin, where it was used to stabilise the steep slopes of a noise protection wall. 4 to 6 cm thick and 3 m long willow rods were placed on berms and covered with a slightly clayey, silty sand with fine gravel. The soil was subsequently compacted to a Proctor density (D_{Pr}) of around 97% using a vibrating roller (Figure 5).

Figure 5: Placing plants on a berm, covering them with soil and compacting it

Tensile tests were also carried out on newly placed, unrooted plants with a soil covering of between 0.4 and 0.7 m, corresponding to a mean normal stress σ of between 5 kN/m² and

10 kN/m². A mean bond strength, τ_f, of 24 kN/m² was obtained in eight tensile tests. This result was slightly higher than that obtained for the trial slope owing to the higher Proctor density in the noise protection wall. The results obtained in the tests to date are summarised in Figure 4. Due to the irregular geometry which varies from plant to plant, there is a relatively high degree of dispersion of the values of the bond strength. A quantitative evaluation of the tests shows that the dispersion of the test results masks the effect of the normal stresses σ on the bond strength τ_f. Given slope inclinations of 1 : 1 and plant lengths of 2 m, the normal stresses σ acting on the surface area of a plant are always less than 25 kN/m². In view of the high degree of dispersion of the bond strength values, it would therefore seem appropriate to apply a constant value of the bond strength τ_f when designing soil bioengineered slopes. Based on the tests conducted so far, the characteristic bond strength $\tau_{f,k}$ can be assumed to be 15 kN/m² at a Proctor density (D_{Pr}) greater than or equal to 93% to obtain a conservative design.

4. Design model

The design model for slopes stabilised by *Living Reinforced Earth* as described in detail by SCHUPPENER (2001) is based on the partial safety factor concept of Eurocode 7 (2004) and DIN 1054 (2005), according to which the design values W_d of the degree of resistance must be shown to be greater than the design values E_d of the actions in the ultimate limit state:

$$W_d \geq E_d \tag{1}$$

The following simplified and conservative assumptions are made when determining the actions and degree of resistance (see Figure 6):

- The failure condition relevant to the design is characterised by a straight failure surface, which is similar to the calculation of the active earth pressure according to Coulomb.

- When considering the equilibrium condition of the sliding wedge, the actions are the self weight of the sliding wedge, G, and the variable load, q, or, alternatively, their components $T_{G,d}$ and $T_{Q,d}$ acting parallel to the failure surface.

- The resistances are composed of the design value of the bearing resistance Z_d of the plants and design values of friction R_d, and the cohesion K_d of the soil.

The limit state equation (1) for the degree of resistance and action in the failure surface is therefore as follows:

$$R_d + K_d + Z_d \geq T_{G,d} + T_{Q,d} \tag{2}$$

The design bearing resistance Z_d of the plants required to ensure that the slope has an adequate level of internal stability is obtained by resolving equation (2) after Z_d.

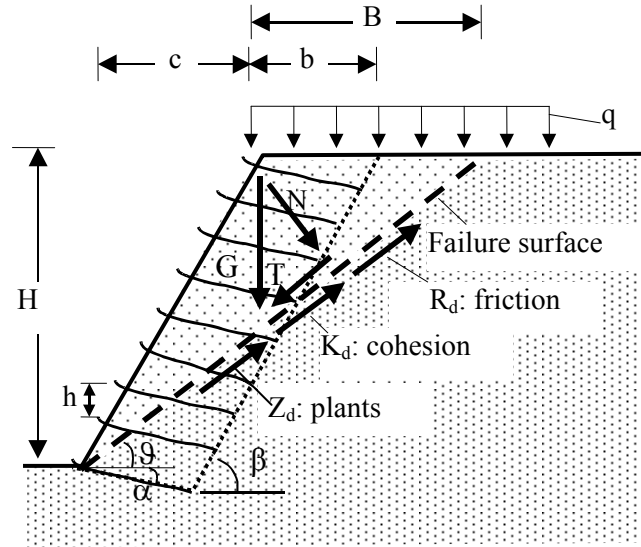$$Z_d \geq T_{G,d} + T_{Q,d} - (R_d + K_d) \tag{3}$$

Figure 6: Stabilised slope with sliding wedge and forces

The design values $T_{G,d}$ and $T_{Q,d}$ of the actions are determined by first obtaining the characteristic values $T_{G,k}$ and $T_{Q,k}$ from the self weight G of the sliding wedge and the variable load q

$$G = H \cdot B/2 \cdot \gamma = H^2 \cdot \gamma \cdot (\text{ctg } \vartheta - \text{ctg } \beta) / 2$$

with the unit weight γ of the soil

$$T_{G,k} = G \cdot \sin \vartheta$$
$$T_{Q,k} = B \cdot q \cdot \sin \vartheta = H \cdot (\text{ctg } \vartheta - \text{ctg } \beta) \cdot q \cdot \sin \vartheta$$

and then multiplying them by the appropriate partial safety factors for the permanent and variable actions γ_G and γ_Q:

$$T_{G,d} = T_{G,k} \cdot \gamma_G$$
$$T_{Q,d} = T_{Q,k} \cdot \gamma_Q$$

The values of the partial factors can be taken from Eurocode 7 (2004) or DIN 1054 (2005). The design values of the soil resistances R_d and K_d are calculated from the geometry of the slope and the self weight G of the sliding wedge using the design values of the shear parameters φ_d and c_d. These are obtained from the characteristic values of the shear parameters φ_k and c_k by applying the partial safety factors γ_φ and γ_c:

$$\tan \varphi_d = (\tan \varphi_k) / \gamma_\varphi \quad \text{and} \quad c_d = c_k / \gamma_c$$

Thus the design values of the soil resistances R_d and K_d are:

$$R_d = N \cdot \tan \varphi_d = G \cdot \cos \vartheta \cdot \tan \varphi_d$$
$$K_d = c_d \cdot H / \sin \vartheta$$

The pull-out tests in the laboratory and on the trial slope demonstrated that the characteristic bond strength $\tau_{f,k}$ between a plant and the soil can be described by a constant value in a sufficiently accurate approximation. The characteristic value P_k of the pull-out resistance of each plant is therefore:

$$P_k = \pi \cdot D \cdot l \cdot \tau_{f,k}$$

where D is the diameter of the plant and l its anchorage length in the soil (see Figure 7). Only the component Z_k of the pull-out resistance parallel to the failure surface is used to verify the internal stability of the slope. Thus,

$$Z_k = \Sigma P_{k,i} \cdot \cos (\alpha + \vartheta)$$
$$Z_k = \pi \cdot D \cdot L \cdot \tau_{f,k} \cdot \cos (\alpha + \vartheta)$$

where ϑ is the inclination of the failure surface, α the inclination of the plant and L the total anchorage length of all the plants subjected to the actions.

The design bearing capacity Z_d of the plants is then obtained by applying the partial factor γ_P for the bearing capacity of the plants:

$$Z_d = Z_k / \gamma_P = \pi \cdot D \cdot L \cdot \tau_{f,k} \cdot \cos (\alpha + \vartheta) / \gamma_P \tag{4}$$
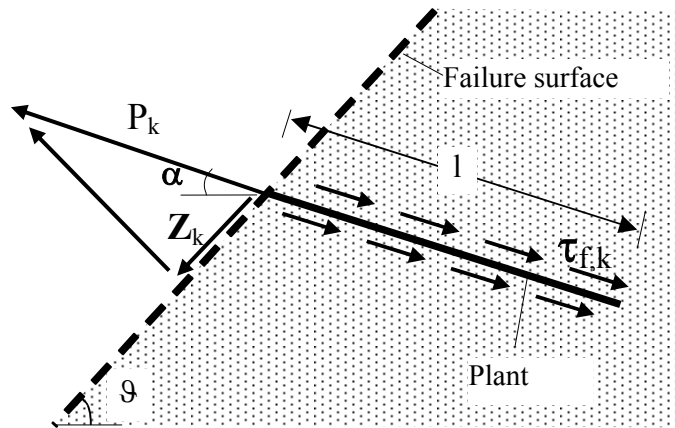
Figure 7: Diagram showing the structural performance of plants

When the failure surface under investigation protrudes at a distance of $B \le b/2$ from the upper edge of the slope (see Figure 6), the anchorage length of each plant on the air-side face of the failure surface is less than that on the opposite side thereof. A failure occurring with this inclination of failure surface will therefore result in the plants being pulled out of the sliding wedge. Given that, generally speaking, $\alpha < 10°$, the mean anchorage length of each plant obtained in a sufficiently accurate approximation is therefore:

$$l_m = B/2.$$

If N is the number of plants per metre placed in the slope, the overall anchorage length is then:

$$L = N \cdot B/2$$

so that the design bearing capacity of all the plants calculated using equation (4) is:

$$Z_d = \pi \cdot D \cdot N \cdot B/2 \cdot \tau_{f,k} \cdot \cos(\alpha + \vartheta) / \gamma_P \tag{5}$$

The design value of the statically required bearing capacity Z_d is calculated from the equilibrium condition of the potential sliding wedge using equation (3) so that equation (5) can be resolved as follows after N, the number of plants required, for failure surfaces where $B \leq b/2$:

$$N = \frac{2 \cdot Z_d \cdot \gamma_P}{\pi \cdot D \cdot B \cdot \tau_{f,k} \cdot \cos(\vartheta + \alpha)} \tag{6}$$

Similarly, the formulae can be developed for failure surfaces with other inclinations (see SCHUPPENER, 2001). Where the distance between the point at which the failure surface protrudes and the upper edge of the slope is such that $b/2 < B \leq b$:

$$N = \frac{Z_d \cdot \gamma_P}{(z_w \cdot l_o/H + (H - z_w) \cdot l_u /H) \cdot \pi \cdot D \cdot \tau_{f,k} \cdot \cos(\vartheta + \alpha)} \tag{7}$$

with

$z_w = H \cdot (1 - b/(2 \cdot B))$
$l_o = (b/2 + (b - B))/2$ and
$l_u = b/4.$

For a situation in which the failure surface is located partly outside the retaining structure $(B > b)$, the number of plants required is:

$$N = \frac{2 \cdot Z_d \cdot \gamma_P \cdot H}{(H - z_w) \cdot b \cdot \pi \cdot D \cdot \tau_{f,k} \cdot \cos(\vartheta + \alpha)} \tag{8}$$

with

$z_w = H \cdot (1 - b/(2 \cdot B)).$

Where the height and gradient of the slopes increase and the height/width ratio of the retaining structure H/b is greater than 2, the design may also be governed by divided failure surfaces (see Figure 8). In this case, the upper failure surface runs along the interface between the stabilised slope and the underlying soil behind the ends of the plants placed in the slope. Only friction and cohesion will be mobilised at this interface in the failure condition to hold the slope in place, but they may not be adequate to preserve the equilibrium of the section of the retaining structure in question. The upper sliding wedge therefore exerts an additional shearing stress on the lower one.

The derivation of a design algorithm for the required number of plants is based on a failure mechanism with two sliding wedges (see Figure 8) in which it is assumed that the failure surface between the wedges is vertical. The following design values of the resistances or actions act at the fractures between the sliding wedges and the underlying soil in the ultimate limit state:

- The cohesion force K_d, the shear force P_d of the cut plants and the force due to friction and the force, Q_d, at an angle φ'_d to the normal at the vertical fracture (see Figure 8) acts vertically in the vertical failure surface between the upper and lower sliding wedges. The conservative value of the shear force P_d of the cut plants is calculated from the cross-sectional area of the plants and their shear strength across the grain.

- The cohesion force $K_{o,d}$ and the resulting force $Q_{o,d}$ acting on the failure surface at an angle of φ'_d to the normal act between the upper sliding wedge and the underlying soil.
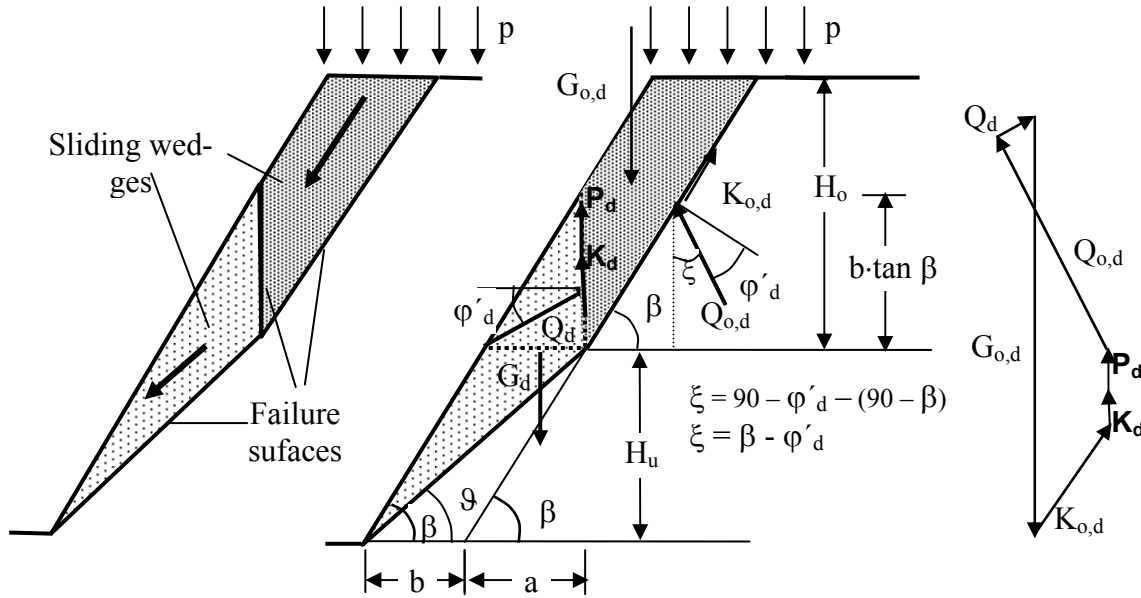

Figure 8: Failure mechanism with two sliding wedges

The quantities of the forces Q_d and $Q_{o,d}$, for which only the directions are known at first, can be calculated from the equilibrium condition at the upper sliding wedge. This can be done either by plotting a triangle of forces or analytically. The following relations are obtained for each geometric quantity:

$$H_u = b \cdot \tan \vartheta /(1 - \tan (90 - \beta) \cdot \tan \vartheta)$$
$$H_o = H \cdot H_u$$

The force $G_{o,d}$ due to the self weight of the upper sliding wedge, which incorporates the live load p to which the partial factor γ_Q has been applied, is obtained as follows:

$$G_{o,d} = (H_o - 0{,}5 \cdot b \cdot \tan \beta) \cdot \gamma \cdot b \cdot \gamma_G + p \cdot b \cdot \gamma_Q$$
$$K_{o,d} = c_{c,d} \cdot H_o / \sin \beta$$
$$K_d = c_{c,d} \cdot b \cdot \tan \beta$$

The following are obtained from the equilibrium conditions using the shear force P_d of the plants:

$$\Sigma V: G_{o,d} - K_d - P_d - Q_d \cdot \sin \varphi'_d - K_{o,d} \cdot \sin\beta - Q_{o,d} \cdot \cos (\beta - \varphi'_d) = 0 \qquad (9)$$
$$\Sigma H: Q_d \cdot \cos \varphi'_d + K_{o,d} \cdot \cos \beta - Q_{o,d} \cdot \sin (\beta - \varphi'_d) = 0 \qquad (10)$$

The following is obtained from (10):

$$Q_{o,d} = (Q_d \cdot \cos \varphi'_d + K_{o,d} \cdot \cos \beta) / \sin (\beta - \varphi'_d)$$

By introducing it in (9):

$$Q_d = \frac{G_{o,d} - K_d - P_d - K_{o,d} \cdot \sin\beta - K_{o,d} \cdot \cos \beta / \tan (\beta - \varphi'_d)}{\sin \varphi'_d + \cos \varphi'_d / \tan (\beta - \varphi'_d)}$$

weight G_d of the lower sliding wedge

$$G_d = 0,5 \cdot b \cdot (b \cdot \tan \beta + H_u) \cdot \gamma \cdot \gamma_G$$

is used to obtain the components of Q_d, G_d, K_d and P_d parallel and perpendicular (index Z and index N respectively) to the failure surface between the lower sliding wedge and the soil as follows:

$$Q_{N,d} = Q_d \cdot \sin (\varphi'_d - \vartheta)$$
$$Q_{Z,d} = Q_d \cdot \cos (\varphi'_d - \vartheta)$$
$$G_{N,d} = G_d \cdot \cos \vartheta$$
$$G_{Z,d} = G_d \cdot \sin \vartheta$$
$$K_{N,d} = c_{c,d} \cdot b \cdot \tan \beta \cdot \cos \vartheta$$
$$K_{Z,d} = c_{c,d} \cdot b \cdot \tan \beta \cdot \sin \vartheta$$
$$P_{N,d} = P_d \cdot \cos \vartheta$$
$$P_{Z,d} = P_d \cdot \sin \vartheta$$

The design values of the forces acting in the failure surface between the lower sliding wedge and the underlying soil, i.e. :

– cohesion force $K_{u,d} = c_{c,d} \cdot H_u / \sin \vartheta$,
– friction force $R_{u,d} = (Q_{N,d} + G_{N,d} + K_{N,d} + P_{N,d}) \cdot \tan \varphi'_d$ and
– bearing capacity $Z_{u,d}$ of the plants,

are used to obtain the design value W_d of the resistance in the lower failure surface as follows:

$$W_d = (Q_{N,d} + G_{N,d} + K_{N,d} + P_{N,d}) \cdot \tan \varphi'_d + K_{u,d} + Z_{u,d}$$

The design value E_d of the actions taken as the sum of the destabilising forces is obtained by:

$$E_d = Q_{Z,d} + G_{Z,d} + K_{Z,d} + P_{Z,d}$$

The design value of the required bearing capacity of the plants is obtained by introducing R_d and E_d into the limit state equation (2) and resolving the equation after $Z_{u,d}$ as follows:

$$Z_{u,d} \geq Q_{Z,d} + G_{Z,d} + K_{Z,d} + P_{Z,d} - ((Q_{N,d} + G_{N,d} + K_{N,d} + P_{N,d}) \cdot \tan \varphi_d + c_{c,d} \cdot H_u / \sin\vartheta) \quad (11)$$

When determining the required number of plants N for the lower sliding wedge, the mean anchorage length of the plants can be assumed to be

$$l = b/4$$

in a sufficiently accurate approximation so that the number of plants N required can be calculated using equation (6):

$$N = \frac{Z_{u,d} \cdot \gamma_{Pf}}{\pi \cdot D \cdot b/4 \cdot \tau_{f,k} \cdot \cos(\vartheta + \alpha)} \qquad (12)$$

As neither the governing failure mechanism nor the inclination of the failure surface ϑ is known at the beginning of the design process, a variation calculation must be performed to determine the required number of plants. This involves studying both failure mechanisms, varying the inclination of the failure surface ϑ and determining the number of plants required for each failure surface inclination. The failure surface inclination relevant for the design is that for which the largest number of plants is obtained.

5. Summary and outlook

Investigations into the way in which plants contribute to the stability of slopes stabilised by the *Living Reinforced Earth* method have shown that the governing parameter is not the strength of the plant material, but the pull-out resistance of the plants or the bond strength between the plants and the soil, τ_f. The bond strength increases with the density of the soil. Root development leads to a four- to five-fold increase in the bond strength over several years. This increase in bearing capacity is a useful reserve in case some of the installed plants die over the course of time. The bond strength varies to a relatively high degree owing to the irregular geometry of the plants, thus cancelling out the effect of the normal stress thereon. It is for this reason that the design model has now been simplified by assuming a constant bond strength τ_f instead of applying the friction law originally used (SCHUPPENER, 1994).

Two design models are being investigated: a rigid body failure mechanism with a straight failure surface and a failure mechanism with two sliding wedges. The partial safety factor concept is applied when deriving the formulae for the design of biologically stabilised slopes that are used to determine the required number, length and thickness of the plants to be installed. The result is a soil mechanical design method verified by tests that enables the stabilising effect of plants to be taken into account in the design of slopes.

In addition to extending the database to enable the bond strength τ_f to be applied in the design to be specified with certainty and realistically, further field investigations are required to establish what type of physical conditions the plants require so that geotechnical engineers are able to rely on their being effective in the long term.

6. Example of a design of a slope stabilised by soil bioengineering

The *Living Reinforced Earth* method is to be used to stabilise a 4 m high slope with an inclination, β, of 50° . The horizontal terrain above the slope is subjected to a live load, p, of 5 kN/m². It is planned to install plants with a mean diameter, D, of 0.02 m on berms at a vertical distance apart, h, of 0.5 m at an inclination, α, to the horizontal of 5° and over a length sufficient to produce a retaining structure with a width, b, of 2.0 m.

The soil of the retaining structure must be compacted to a Proctor density (D_{Pr}) of 93%, so that an angle of friction, φ'_k, of 32,5° and a capillary cohesion, $c'_{c,k}$, of 2 kN/m² can be assumed in the stability calculations. The soil and the retaining structure each have a unit weight, γ, of 18 kN/m². It is assumed that the characteristic bond strength between the plants and the soil, $\tau_{f,k}$, is 15 kN/m² immediately after slope stabilisation using the *Living Reinforced Earth* method. But the strength should be verified by pull-out tests during construction where appropriate. The stabilised slope is designed using the partial safety factors specified in DIN 1054 (2005): γ_G= 1.0 for the permanent actions, γ_Q =1.30 for unfavourable variable actions, γ_φ = 1.25 for the tangent of the angle of shearing resistance tan φ'_k, γ_c= 1.25 for the cohesion $c'_{c,k}$ and γ_P =1.40 for the pull-out resistance of the plants.

A variation calculation is performed using equation (3) to determine the design value of the bearing capacity Z_d of the plants, assuming straight failure surfaces (see Table 1) and reducing the inclination ϑ of the failure surfaces in increments, $\Delta\vartheta$, of 2° from ϑ = 42° to ϑ = 30°. The number of plants required per metre slope, N, or the number of plants required per metre berm, n, is then determined using the equations (6) (7) and (8) depending on the inclination of the failure surface.

Table 1: Design results for straight failure surfaces

Inclination of failure surface ϑ	Z_d kN/m	B m	N	n	Remark
42	3,8	1,1	16	1,9	
40	5,2	1,4	19	2,3	
38	5,6	1,8	21	2,6	
36	**5,1**	**2,2**	**22**	**2,7**	
34	3,5	2.6	17	2,2	
32	0,6	3,0	3	0,4	
30	-3,9	3,6	0	0	

An analogous variation on the calculations is performed assuming a failure mechanism with two sliding wedges to determine which inclination ϑ of the lower sliding wedge requires the highest number of plants (see Table 2). First, the required bearing capacity $Z_{u,d}$ of the plants is determined using equation (11). When determining the shear force of the plants in the vertical fracture between the two sliding wedges, it is assumed in this case that five plants per metre berm are required. Five rows of plants are cut for a slope with an inclination, β, of 50° and with a vertical distance, h, of 0.5 m between the rows, so that a total of 25 plants contribute to the shear force. Assuming that the shear strength of the wood across the grain, σ_{zul}, is approximately equal to 1 N/mm², or 1.000 kN/m², in accordance with NIEMZ (1993) and already includes a safety factor, the design shear force of 25 plants is therefore: $P_d = 25 \cdot \pi \cdot D^2/4 \cdot \sigma_{zul} = 7,9$ kN/m. The final step is to determine the number of plants needed to achieve the required bearing capacity of the plants using equation (12).

Table 2: Design results for a failure mechanism with two sliding wedges

Inclination of failure surface ϑ	Z_d [kN/m]	H_U [m]	N	n	Remark
19	3,1	0,97	14,2	7	
21	**3,8**	**1,13**	**17,8**	**9**	Relevant for design
23	4,4	1,31	20,6	7	
25	4,6	1,53	22,3	7	
27	4,6	1,77	22,7	7	
29	4,2	2,07	21.1	6	

The variation calculations show that a failure mechanism with two sliding wedges and a fracture inclination, ϑ, of 21° requires a greater number of plants than when straight failure surfaces are assumed. In the latter, the highest number of plants is required when the inclination of the failure surface, ϑ, is 36°. The design of stabilised slopes is based on the failure surface for which the highest number of plants is required. Accordingly, the design of the slope under consideration here must be based on the failure mechanism with two sliding wedges.

7. Bibliography

DIN 1054 (2005)   *Ground – Verification of the safety of earthworks and foundations*, Beuth Verlag, Berlin

DIN EN 13738 (2000)   *Geotextiles and geotextile-related products – Determination of pullout resistance in soil*, Draft February, 2000

Eurocode 7 (2004)   *EN 1997-1: Geotechnical design - Part 1: General rules* October 2004

Niemz, P. (1993)   *Physik des Holzes und der Holzwerkstoffe*, DRW-Verlag

Schiechtl, H. M. (1987)   *Böschungssicherung mit ingenieurbiologischen Bauweisen, Grundbautaschenbuch*, 3rd edition, Part 3, pp. 217 – 315

Schuppener, B. (1994):   *Die statische Berechnung der Bauweise Lebend Bewehrte Erde*, geotechnik 4

Schuppener, B. & Hoffmann, J. (1999)   *Living Reinforced Earth – an ecological method of stabilizing steep slopes*, Proc. of 12th ECSMGE, Vol. 1, Amsterdam, 1999

B. Schuppener (2001)   *Bemessung von Böschungssicherungen mit Pflanzen*, geotechnik 24 (2001), No. 3