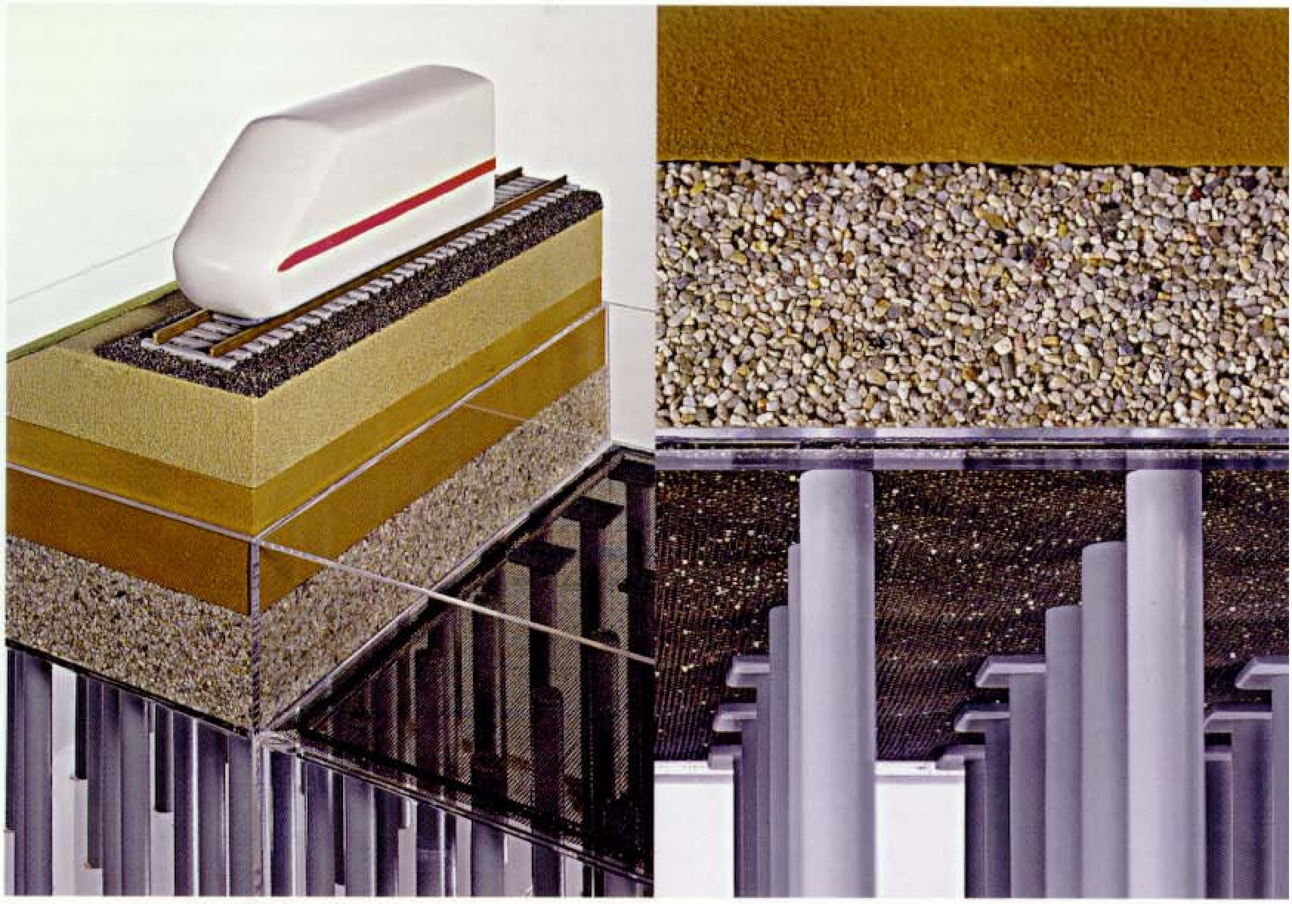
Bautechnik

12

The General Civil-Engineering Magazine



Vol. 74
December 1997
No. 12
ISSN 0932-8351

Ernst & Sohn

Design of geosynthetic-reinforced bearing layers over piles

Embankment using high-strength geogrids over piles (model and photos: HUESKER Synthetic, Gescher)

Hans-Georg Kempfert, Michael Stadel and Dirk Zaeske

Hans-Georg Kempfert
Michael Stadel
Dirk Zaeske

Design of geosynthetic-reinforced bearing layers over piles

Special Topic

Design of geosynthetic-reinforced bearing layers over piles.
The construction of geosynthetic-reinforced bearing layers over piles is a new method of foundation over soils with low-bearing capacity. This type of construction is primarily used for transportation routes, where an economical basal solution is required. The basis for the analytical calculation and design of the composite construction is explained.

1. Introduction

Geosynthetic-reinforced mineral bearing layers are finding increasing use in foundations for transportation routes constructed on low-strength soil, transferring the load from the structure such as the embankment and incident traffic via arch support onto pile elements of various types (e.g. driven piles, grout injected stone columns, etc.). The piles penetrate the underlying settlement-prone weak soil layers in a grid arrangement to reach the lower load-bearing stratum. A geosynthetic reinforcement, generally comprising one or more layers of a high strength geogrid, is placed over the pile caps, and below the base of the embankment and the overlying structure. The basic principle of this foundation system is shown in fig. 1.

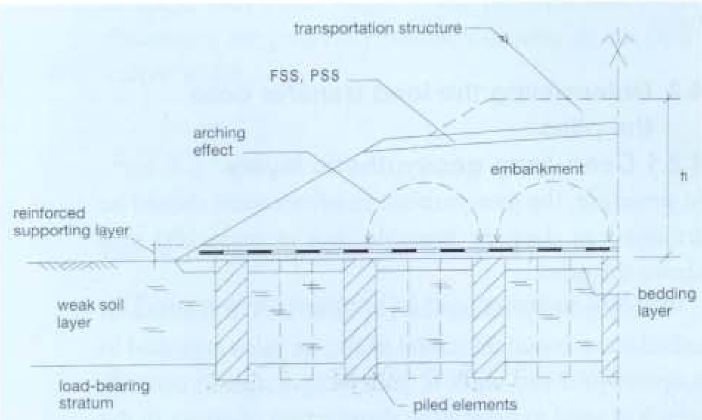


Fig. 1. Geosynthetic-reinforced mineral bearing layer over piles

Compared with other methods of foundation improvement (e.g. soil replacement), the construction of geosynthetic-reinforced bearing layers over piles is in many cases not only an economical alternative, but also an environmentally compatible solution, avoiding the need to disturb the underlying soil or alter its natural composition. As this foundation method allows considerable flexibility, construction methods can also be readily adapted to meet changing subsoil conditions. Initial design conditions for this composite method of construction were given in [1], which has since been updated, taking into account further practical knowledge.

This foundation method may in principle also be used for conventional structures (tank foundations, buildings, etc.), which mainly involve passive loading.

2. Load-bearing behaviour of the system

The vertical stresses made up of dead weight and traffic loading are transferred at the base of the embankment onto the piles and the underlying soil. Since the piles are more rigid than the surrounding material, the vertical loads are concentrated above the piles. Conditions of equilibrium reduce the vertical load on the intervening spaces and the load on the soil is reduced. The degree of load transfer will depend on the difference in rigidity between the piles and the soil, and on the shear strength of the embankment fill.

Settlement differences at the base of the embankment between the piles and the resilient subsoil produce strain and tensile forces in the installed geosynthetic material. The geosynthetic extends like a membrane over the largely rigid piles, resulting in a further transfer of the vertical loading onto the piles. The membrane effect of the geosynthetic material is shown diagramatically in fig. 2. In the diagram, the stress above the piles is indicated as σ_p, the stress above the intervening spaces as σ_s and the reaction stress of the soil as σ_r.

However, the limited rigidity of the geosynthetic material does not allow the load to be completely removed from the soil between the piles, which must still provide a somewhat reduced bearing function.

With increased strain in the geosynthetic material, the transfer of load onto the piles increases with a simultaneous increase in the supporting effect of the soil. This leads in turn to a reduction in stress in the geosynthetic material and with that a reduction in the strain required to provide equilibrium. A balance is reached between imposed and resisting stresses; this self regulation of forces being characteristic of the system's stable state of equilibrium.

In addition to the membrane effect geosynthetics can sometimes absorb horizontally aligned forces, which may be produced by the distribution of traffic load or the horizontal components of soil pressure in the embankment. These generally outward-directed forces are called lateral thrust [2]. They must, if the soil is not laterally supported in the region of a slope, be transferred by shear stress to the subsoil or the geosynthetic material, see fig. 3.

3. Forces

Forces include dead weight and traffic load, the former being determined according to the usual rules of structural engineering.

Applicable traffic loading should be determined in accordance with the specifications of the particular application. Traffic load is usually taken to be a static load, and dynamic or cyclical load influences are covered by an additional load-increasing factor. When calculating railway traffic load, for instance, a load-increasing factor k_S should be included as described in [3] to allow for dynamic load effects as a function of design speed.

4. Verification and calculation

4.1 Stability check

The greater rigidity of the piles increases resistance to deep-reaching failures in the lower strength soil layers. Sufficient safeguard against failure can be verified by established methods of calculation, either by taking into account the effect of the piles and their proportional shearing forces or by generally increasing the shear strength of the weak load bearing layers. The forces in an intersected geosynthetic reinforcement should also be included in the calculation. The tensile forces included in the calculation may be based on the strength or the pull-out resistance of the reinforcement.

In detail, the following verification of the system's stability should be undertaken:

a) Overall stability in terms of resistance to shear failure or basal failure with allowance for the shear strength of the piles, to DIN 4017 or DIN 4084.

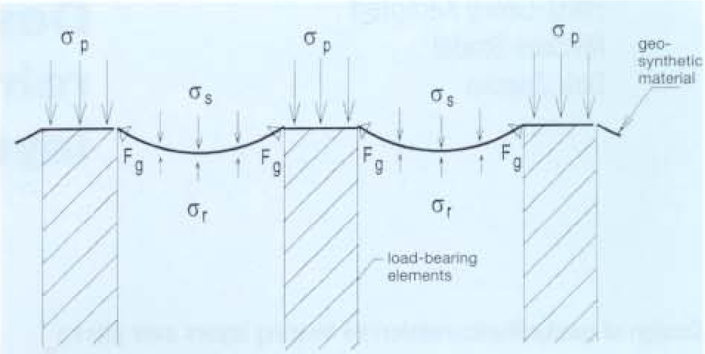


Fig. 2. Distribution of the vertical stresses in the pile region in the deformed system

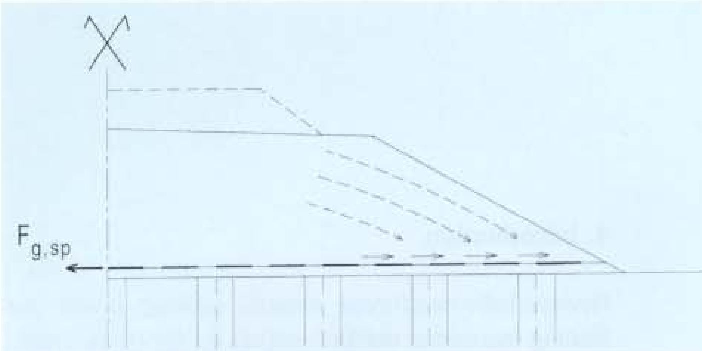


Fig. 3 Take-up of the lateral thrust by shear stresses in the geosynthetic reinforcement

b) Stability of the slope at the edge (or toe) to DIN 4017 or 4084.

c) Verification of the outer and inner load-bearing capacity of the piles.

d) Verification of the geosynthetic reinforcement's ability to withstand rupture taking into account the known tensile strength value.

e) Verification of joint strength of the reinforcement in the direction of principal tensile forces.

f) Verification of anchorage of the geosynthetic materials.

4.2 Determining the load transfer onto the piles

4.2.1 Deep-lying geosynthetic layers

In principle, the geosynthetic reinforcement should be arranged as deep as possible, but at least 200 mm above the piles.

Bibliography reference [4] describes a method of calculation, based on model trials, for piles arranged in a square grid and without geosynthetic reinforcement, which is used to determine distribution of stress in the embankment footprint as a function of the depth of surcharge, the axial spacing and diameter of the piles and the angle of internal friction of the embankment fill material. This method can be applied to the piles

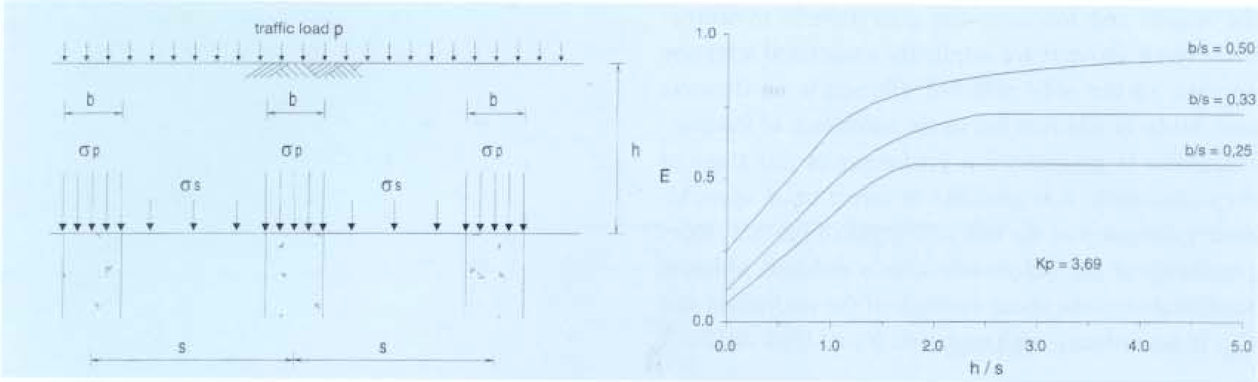


Fig. 4. Load distribution on the piles

discussed in this paper and is based on the formation of three-dimensional arches (hemispheres) above the load-bearing members. The method has been extended in [1] to the system with a geosynthetic-reinforced bearing layer above the piles, which is under discussion here. Accordingly, using the terms in figs. 5 and 6, the relationship between the vertical load on the piles and the vertical load on the surface of influence associated with a load-bearing element can be expressed as the non-dimensional ratio-

$$E = \frac{\text{load on a load-bearing element}}{\text{total load on the associated surface of influence}}$$

$$E = \frac{\sigma_p \cdot b^2}{(\gamma \cdot h + p) \cdot s^2} \tag{1}$$

E describes the degree of load transfer onto the pile caps, which is to take place, see fig. 4. Two boundary conditions, namely failure of fill material at the crown of the arch and failure of fill material at the bearing point of thearch, are examined in order to determine E.

Since it is assumed in [4] that the piles are square in shape with a side length b, and the piles now under discussion are generally round (diameter d), an equivalent width

$$b_{ers.} = \sqrt{\frac{d^2 \cdot \pi}{4}} \tag{2}$$

can be taken as an approximation for b.

1) Failure of fill material at the crown of the arch

$$E = 1 - \left[1 - \left(\frac{b}{s}\right)^2\right] \cdot (A - A \cdot B + C) \tag{3}$$

$$A = \left[1 - \left(\frac{b}{s}\right)\right]^{2 \cdot (K_p - 1)} \tag{3a}$$

$$B = \frac{s}{\sqrt{2} \cdot h} \cdot \left(\frac{2 \cdot K_p - 2}{2 \cdot K_p - 3}\right) \tag{3b}$$

$$C = \frac{s - b}{\sqrt{2} \cdot h} \cdot \left(\frac{2 \cdot K_p - 2}{2 \cdot K_p - 3}\right) \tag{3c}$$

$$K_p = \frac{1 + \sin \varphi'}{1 - \sin \varphi'} \tag{3d}$$

2) Failure at bearing point of the arch

$$E = \frac{\beta}{1 + \beta} \tag{4}$$

$$\beta = \frac{2 \cdot K_p}{K_p + 1} \cdot \frac{1}{1 + \frac{b}{s}} \cdot \left[\left(1 - \frac{b}{s}\right)^{-K_p} - \left(1 + K_p \cdot \frac{b}{s}\right)\right] \tag{4a}$$

Equation (3) should be used for light surcharges and wide pile spacing, while with increasing surcharge h and similar spacing s equation (4), failure of fill material at the bearing point of arch is more suitable.

This method is valid as long as the depth of surcharge h is not less than the axial pile spacing s. Should $h < s$, the load transfer value E can be determined in simplified form by linear interpolation between $h = s$ and $h = 0$. The load distribution for $h \rightarrow 0$ is then derived from the ratio of the cross-sectional area of a pile to the area of its surface of influence; in other words there is no additional transfer of load to the piles. By way of example, fig. 4 illustrates the variation of E as a function of h/s and b/s given an angle of internal friction of the fill material $\varphi' = 35°$ ($K_p = 3{,}69$).

Stress σ_s on the weak layer, taken to be uniformly distributed, can be calculated as follows:

$$\sigma_s = \frac{(\gamma \cdot h + p)}{(s^2 - b^2)} \cdot (1 - E) \cdot s^2 \tag{5}$$

The geosynthetic material is subject to the stress σ_s on the soil minus a vertical reaction stress produced by the supporting effect of the soil. This supporting effect can be determined by including a bedding modulus for the soil dependant on the vertical deformation [5]. However, the deformation of the soil and the strains and forces in the geosynthetic reinforcement lying above it are implicitly associated with one another, so the only possible solution is an iterative one. Since in addition the strain resistance of the reinforcement is generally not yet known at this stage of the calculation, it is possible to arrive at an approximate estimation of the reaction stress of the soil, independantly of the deformation, as a reduced ultimate load related to the shear strength of the undrained soil C_u, in accordance with equation (6) of DIN 4017.

$$\sigma_0 = \sigma_{0f}/\eta = \frac{(2+\pi)\cdot c_u}{\eta} \tag{6}$$

The safety factor used to reduce σ_{0f} should be included here as at least $\eta = 2$. If there are stratified soils below the geosynthetic layer, averaged c_u values may be used, or an equivalent shear strength may be estimated from cohesive and non-cohesive layers. Comparative calculations have shown that this method gives somewhat higher stresses in the geosynthetic material than are indicated for normal soil characteristics by the use of the bedding modulus.

The vertical load on the geosynthetic layer spanning between the pile caps, which acts on the span (s'-b), can be calculated for the load bearing surfaces of a square or triangular grid of piles as shown in fig. 5 by using the following equation:

$$q_G = \frac{\sigma_s \cdot (s^2 - b^2)}{2 \cdot (s' - b)} - \frac{\sigma_0 \cdot (s^2 - b^2)}{2 \cdot (s' - b)} \tag{7}$$

Where there are several layers of geosynthetic reinforcement above the pile-type load bearing elements, the load q_G can be distributed between the geosynthetic materials approximately in proportion to their strain resistances.

4.2.2 High-lying geosynthetic layers

The membrane effect of the reinforcement decreases with increasing vertical distance from the pile caps, since the transfer of shear forces in the soil layer between the synthetic material and pile cap causes settlement differences to be reduced. This effect is produced when a mineral intermediate layer is arranged under the reinforcement, or when, as often has been the case in practice to date, so-called partially grouted stone columns are used as load-bearing elements. With partially grouted stone columns a compacted gravel cap is placed above a stiff (grouted) piled element and the reinforcement is placed on the working plane over the relatively yielding gravel caps. Advice on installation and experience with this system are given in [6]. The load bearing behaviour, which results when reinforcement is placed at a high level can be understood by considering two extremes. One

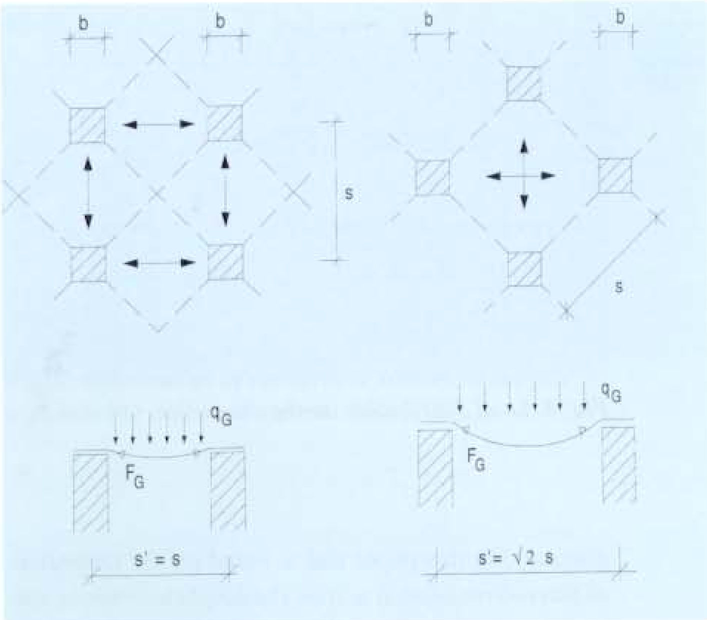


Fig. 5. Load distribution on the geosynthetic layers for piles in rectangular and triangular grid spacing

extreme assumes that the reinforcement has no influence on the transfer of load onto the piles and that the arching effect develops mainly below the reinforcement. In this case proof must be provided that the vertical soil stress σ_s in the plane of the top edge of the rigid part of the pile element, which has been calculated with equation (5) for height h in fig. 6 (on the left), can be absorbed by the subsoil without inclusion of the reinforcement.

The other extreme assumes that the arch supports are loaded onto the subsoil in the plane of the reinforcement. The load transfer should then be determined for height h' in fig. 6 (on the right). The influenece of the transfer of load onto the piles, which has already taken place below the reinforcement layer, can be represented in non-cohesive soil by conical load application regions inclined at angle of 60° to the horizontal. To calculate the transfer value E in accordance with equation (3) and equation (4), the diameter of the pile is therefore increased from d to d' as in fig. 6 (on the right). If the soil between the reinforcement and pile contains a significant level of fine-grained fractions (fractions with $d_{15} < 0.06$ mm of more than 15%), the increase in diameter should be limited to $d' \leq 1.2 \cdot d$. However, this theoretical increase in diameter should only be used when the distance between pile cap and the reinforcement is > 0.5 m. Depending on the space between the reinforcement layer and the rigid part of the pile element, and on the rigidity of the gravel cap compared with the surrounding embankment material, the load-bearing behaviour will approach either one of the two extremes presented. Mathematically to ensure the stability of the system, it is recommended that, if the distance between the

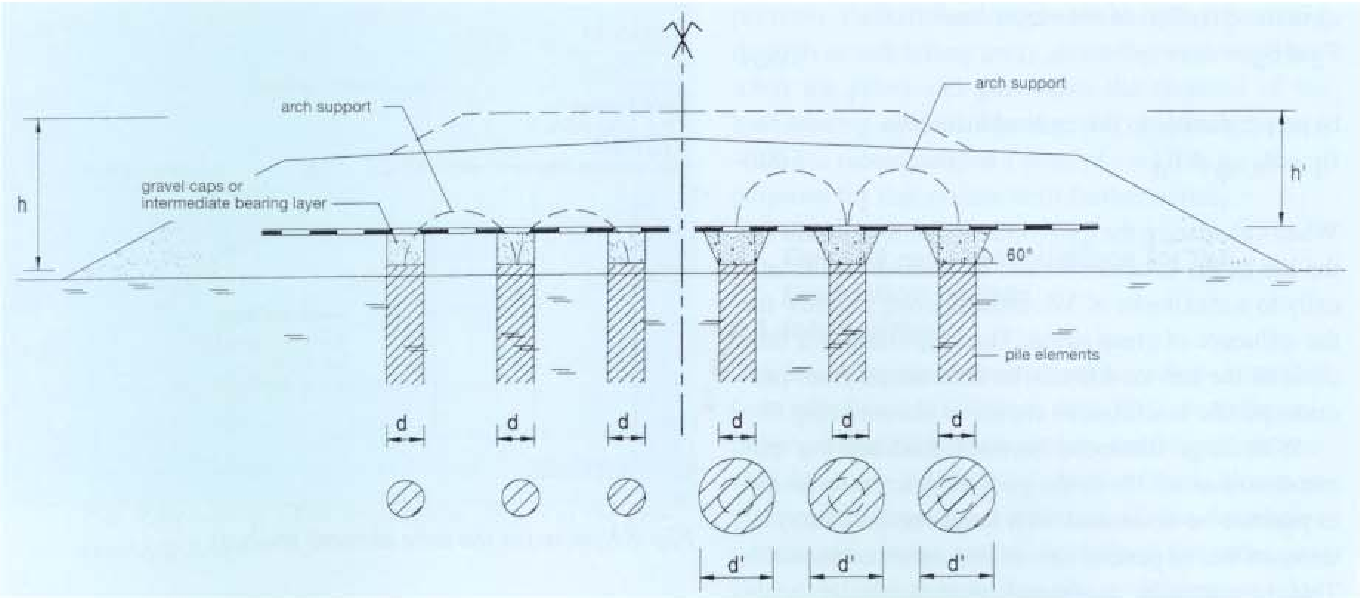


Fig. 6 High-level geosynthetic over the piles without (on left) and with (on right) mathmatical allowance for reinforcement

reinforcement layer and the rigid section of the pile is > 0.5 m, both load-bearing models should be examined. However, it is recommended that any distance > 1.0 m should be avoided.

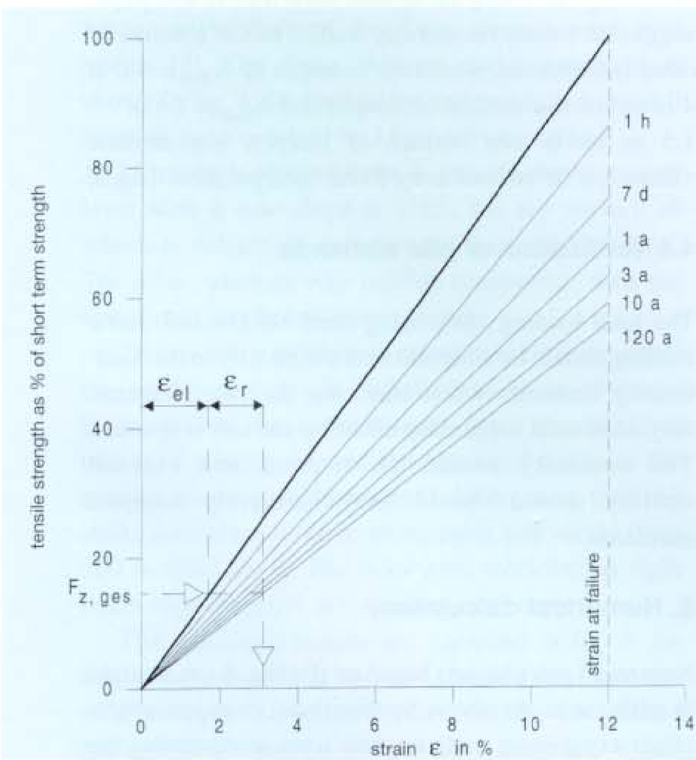


Fig 7. Isochronous stress - strain properties of a geosynthetic, qualitative

4.3 Design of geosynthetic materials in a mineral bearing layer

The tensile force F_Z present in the geosynthetic reinforcement can be calculated in simplified form by using static catenary formula by considering a geogrid strip of width b under uniform load q_G over the span (s'- b).

The tensile force as a function of strain ε in a two-dimensional catenary can be calculated in a simplified manner from the relationship

$$F_z = \frac{q_G \cdot (s' - b)}{2 \cdot b} \cdot \sqrt{1 + \frac{1}{6 \cdot \varepsilon}} \qquad (7)$$

F_Z is the active load on the geosynthetic reinforcement, to be determined iteratively, because in the above strain ε is included as an additional unknown quantity. The relationship between the assumed strain ε and the calculated force F_Z is to be determined by referring to the characterisitic curve (stress-strain curve) of the geosynthetic material employed. In embankment systems the geosynthetic material must also absorb horizontal spreading forces $F_{Z,sp}$ which are directed to the base of the slope and transferred by shear stress to the geosynthetlc material. To be on the safe side these spreading forces can either be calculated from an assumed active soil pressure, resulting from the passive weight and traffic load, which will have built up between the reinforcement layer and the crown of the embankment, or can be calculated as described in [2], and applied to the geosynthetic material in a transverse direction. As a result there are two calculation directions for embankment systems, namely

a) in the direction of the embankment axis

$$F_Z \leq F_d \quad (8a)$$

b) perpendicular to the embankment axis

$$F_Z + F_{Z,\,sp} \leq F_d \quad (8b)$$

When calculating the geosynthetic the total strain for the life of the structure should be limited mathematically to a maximum of 3%, remembering to allow for the influence of creep strain. The total strain as a function of the service life can be determined from product-specific isochronous curves as shown in fig. 7.

With large distances between load-bearing elements strains of 3% in the geosynthetic material can in practice be associated with large vertical deformations, so that in general low strains are recommended. This is normally confirmed mathematically when equation (7) is used.

With current knowledge it is not possible to provide data on the resistance of the geosynthetic material to mainly non-passive loading. Strain measurements available to date and conducted on geogrids made of polyester, installed in a railway embankment, approx. 2.5 m high, at a short distance above the piles, indicate a slight influence of dynamic traffic loading on the strain behaviour of the reinforcement [7]. Based on the passage of trains at speeds of 160 km/h the dynamic strain component after two years in operation is only about 0.01%, while the total strain generated by the static loading is up to 1 %. But since the influences of train speed, the distance between the reinforcement and the top of the rail, the relative rigidities of the soil layers and the strain resistance of the reinforcement on the stress in the geosynthetic material over long periods in use cannot yet be described with sufficient reliability, it is recommended (provisionally) to introduce a partial reduction factor A_{dyn} when determining admissible geosynthetic material stresses. The reduction factor A_{dyn} can be included in determining the design value for the tensile strength of the geosynthetic material by using the following relation as described in [8]:

$$F_d = \frac{F_k}{A_1 \cdot A_1 \cdot A_1 \cdot A_1 \cdot A_{dyn} \cdot \gamma} \quad (9)$$

F_d design value for long-term strength of geosynthetic material

F_k short-term strength of geosynthetic material

A_1 reduction factor for creep

A_2 reduction factor for installation damage

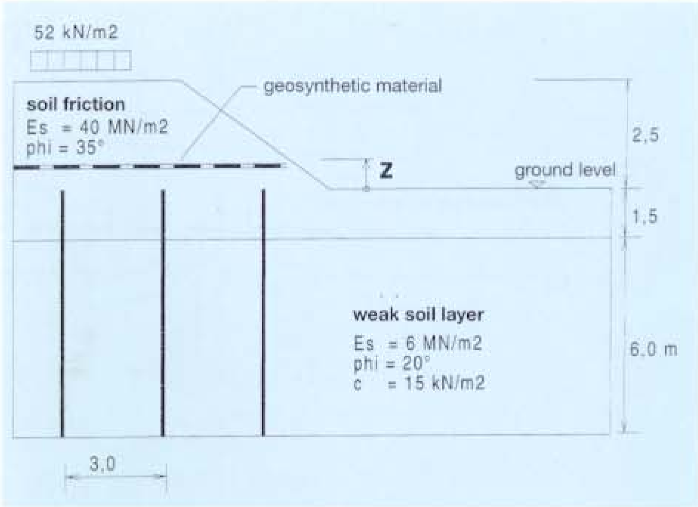


Fig. 8 System for the inite element analysis

A_3 reduction factor for manufacturing consistency

A_4 reduction factor for environmental influences

A_{dyn} provisional reduction factor to allow for dynamic traffic load

γ safety factor

The values of the reduction factors A_1 to A_4 should be verified by the geosynthetic manufacturer. The reduction factor A_{dyn} is to be used only when the effect of traffic load is high (e.g. on railway lines) and should be defined as a function of depth below the load application region. Using the system described in this paper suggested values for railway traffic, in the absence of other information, would for example be A_{dyn} = 1.0 at 4 m below top surface of sleeper and A_{dyn} = 1.5 at 1.5 m below top surface of sleeper. Intermediate values can be arrived at by linear interpolation. [fig.8]

4.4 Verification of pile elements

The total loading comprising dead weight and traffic loading should be assigned mathematically to the load-bearing elements when checking the pile elements. Any additional supporting effect by the soil is ignored. The necessary checks on internal and external stability should should follow currently accepted standards.

5. Numerical calculation

Numerical calculations based on the Fig. 8 can be used in addition to the above verifications, or given appropriate experience, even on their own, in designing the load-bearing system and thus determining the live loads on the geosynthetic material.At present calculations are economically feasible only when based on a two-dimensional deflection model; so the

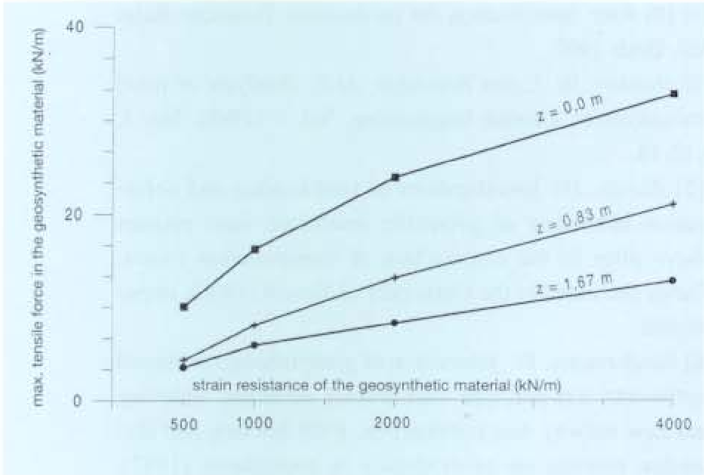


Fig. 9 Results of FEM calculation, forces in the geosynthetic layer

three-dimensional set-out of piles must be transferred to a two dimensional system in keeping with the load distribution [9].

The calculations shown were produced from the finite element system satisfying the following requirements:
- geosynthetic elements
- friction elements between the geosynthetic material and soil (interfaces)
- elasto-plastic material behaviour

The effect of a geosynthetic reinforcement at variable levels within the embankment is described below in the light of the finite element analytical results [5]. The finite element model employed is shown in fig. 8, the foundation soil consisting of a 6.0 m weak zone overlaid by 1.5m of frictional soil.
A 2.5 m high embankment is placed above ground level with a side slope at 1:1.5, the top surface of which is subject to a distributed load of 52 kN/m². The piles, taken as very rigid in comparison with the soil, finish just below ground level.

To provide a number of calculations a geosynthetic reinforcement was installed at three different levels within the embankment, from ground level (distance z in fig. 8), and the measured forces in the reinforcement were determined. At the same time the strain resistance of the reinforcement was varied from 500 to 4000 kN/m. The piles were modelled as rigid beam-type elements at a spacing s = 3.0 m.

The calculated results are compiled in fig. 9. As might be expected a shorter distance between the reinforcement layer and piles results in greater applied forces on the geosynthetic material, which can be further enhanced by an increase in the strain resistance.

The main purpose of the reinforcement is to secure a stable zone of equilibrium above the piles, which prevents local failure mechanisms (piles punching through or soil falling away above the weak zone between the piles) and guarantees the function of the load-bearing system. Bearing that in mind, Section 4.2.2 has recommended a max. of z = 1.0 m for design purposes for this system until further notice.

6. General recommendations for the foundation system

6.1 Introduction

The following recommendations have been compiled from experience available to date. They cover the definition of several basic boundary conditions and provide advice on practical construction work. However, the precise boundary conditions of the foundation system will always need to be defined for each particular case, which may give rise to different or additional requirements.

6.2 Soil mass

To ensure a sufficient bond, selected coarse-graind types from the soil groups GW, GI, GE, SW and SI (DIN 18 196, Soil Classification / construction) are recommended for use in contact with the geosynthetic material. These soils must be weather resistant and not include fractions capable of swelling, prone to disintegration or creating construction difficulties. It is particularly important to ensure the soil is compatible with the geosynthetic material.

There should be an intermediate base course between the top surface of the rigid piles and the lowest geosynthetic layer to prevent the geosynthetic lying directly on the pile caps.

The load-bearing capacity of the soil immediately below the lowest geosynthetic layer should be verified and must include a deformation modulus of at least $E_{V2} \geq 20$ MN/m². If larger on going settlement in the weak formation zone is anticipated due to groundwater fluctuations, special investigations will be necessary or otherwise this method should not be used.

6.3 Geosynthetic materials

Irrespective of the mathematical requirements described in Section 4, the geosynthetic materials used should have an ultimate tensile strength of at least 60 kN/m with a max. elastic strain component of 3% as a minimum requirement. With multi-layer reinforcement the vertical space between the layers should not be less than 0.15 m.
The geosnthetic layers should be laid horizontally and should be straightened with slight tension before placing the next layer of fill to ensure the material is

should be laid horizontally and should be straightened with slight tension before placing the next layer of fill to ensure the material is crease-free. Any direct traffic over the laid out geosynthetic should be avoided. No traffic is recommended until a compacted layer (0.15 m) of fill covers the geosynthetic.

Apart from this the manufacturer's installation instructions are to be followed or checked out.

In principle, the geosynthetic material should be rolled out in the direction of the greatest tensile forces (on transportation routes perpendicular to the direction of travel). Adequate transfer of force is to be ensured at overlapped joints, a minimum of 0.5m overlap being recommended. For anchorage the geosynthetic material should extend at least 1.0 m beyond the axis of the outermost row of piles.

6.4 Piles

It is important to ensure a regular grid is maintained when installing the piles. A working platform will be necessary to allow use of heavy equipment for placing the load-bearing elements and for this reason a depth of subgrade may need replacing, thus providing a bedding or levelling course in the region of the pile heads after their installation.

The axial distance between adjacent piles should be no greater than s = 1.75 m. and when constructing railtrack at least three rows of columns should be provided for each line of track.

Bibliography:

[1] *Kempfert, H.-G and Stadel, M:* The load-bearing behaviour of geosynthetic-reinforced earthworks above pile-type load-bearing elements. Fourth briefing and discussion meeting on geosynthetics in geotechnics (1995). Munich. Deutsche Gesellschaft fuer Geotechnik e.V., Geotechnik special issue, published 1995, p. 146-152.

[2] *Brauns, J.:* Slope stability on inclined ground. Bauingineur 55 (1980), p. 433-436.

[3] DS 836: Specification for earthworks. Deutsche Bahn AG. Draft 1997.

[4] *Hewlett, W. J. and Randolph, M.F.:* Analysis of piled embankments. Ground Engineering, Vol. 1 (1988), No. 3, p.12-18.

[5] *Zaeske, D.:* Investigations of load-bearing and deformation behaviuor of geotextile reinforced base courses above piles in the construction of transportation routes. Thesis presented to the University of Kassel (1996), unpublished.

[6] *Sondermann, W.:* Interaction of geosynthetic-reinforced earthworks and pile-type load bearing elements, widening and new railway line construction. Fifth briefing and discussion meeting on geosynthetics in geotechnics (1997). Munich. Deutsche Gesellschaft fuer Geotechnik e.V., Geotechnik special issue, published 1997, p. 47-51.

[7] *Gartung, E., Verspohl, J., Alexiew,D . and Bergmair,F.:* Geogrid-reinforced railway embankment on piles - Monitoring. Proceedings of the first European Geosynthetics Conference, EUROGEO 1, Maastricht. A.A. Balkema / Rotterdam / Brookfield (1996). p. 251-258.

[8] FGSV: Code of practice for the use of geotextiles and geogrids in road construction earthworks. Forschungsgesellschaft fuer Strassen- und Verkehrswesen e.V., Cologne (1994).

[9] *Bergado, D. and Long, P. V.:* Numerical analysis of embankment on subsiding ground improved by vertical drains and granular piles. Geotechnical Institute of Technology Bangkok, XIII ICSMFE (1994), New Delhi, India, p. 1361-1366.

Authors of this paper:

Uni.-Prof. Dr.Ing. Hans-Georg Kempfert,, University of Kassel

Dipl.-Ing. Dirk Zaeske, University of Kassel, Geotechnics Faculty, Moenchebergstrasse 7, 34125 Kassel

Dipl.-Ing. Michael Stadel, Kempfert & Ptnr GmbH, Untere Koenigsstrasse 86, 34117 Kassel

"Special Print by Bautechnik 74(1997), H.12, S.818-825"